Ashley Valley

Storm Water Master Plan

June 2008

Final Issued December 2008

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Uintah county

People Serving People
City of Naples

Vernal City

Storm Water System
Master Plan

June 2008

Final Issued December 2008

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EXECUTIVE SUMMARY

This report is intended to provide the necessary guidance to the local governing bodies within the Ashley Valley to ensure that the Valley will be protected from future large flooding events. The project was commissioned in 2006 by the governmental entities within the Ashley Valley: Vernal City, Naples City, and Uintah County. The region is currently experiencing rapid growth. This growth is continually encroaching upon natural stream channels and other previously undeveloped portions of the Valley. This study is intended to provide guidelines to ensure that development is regulated in a manner that will provide adequate protection from large storm events.

The purpose of the study is to:

1) Evaluate the major components of the existing storm water network based on existing conditions as well as determine how the existing network will behave with future planned development;

2) Determine deficiencies within the system and portions of the Valley that are at risk of flooding currently as well as areas that may be at risk in the future;

3) Provide a comprehensive plan to control storm water now and in the future.

EVALUATIONS

The first step in the evaluation process is to determine the major components that comprise the storm water management system and determine the adequacy of the existing system. Through meeting with the local staff, field investigations and research of previous studies, the major components of the existing storm water conveyance system were determined to consist of natural drainage channels throughout the basin, a series of irrigation canals, roadside swales, culverts, and a few storm drain pipes throughout the highly developed regions of the basin.

Using advanced modeling techniques, the existing system was modeled under the 10-, 25-, 50-, 100-, and 500-year storm events. The evaluation of the existing system indicated:

1) Natural flood channels have been modified and/or filled throughout the basin;

2) The irrigation canals could not safely convey storm water during large storm events;

3) Portions of the Valley are at risk of flooding during a 25-year or larger storm event;

4) The capacity of Upper Ashley Creek and the major bridges were inadequate above the 50-year event;

5) Existing regulations were not sufficient to provide adequate flooding and water quality protection with the growing population;

6) Peak velocities in many of the stream channels are likely to cause erosion, degraded water quality, and potential migration of the stream channels.

Once the existing system evaluation was complete and deficiencies noted, the modeling process was repeated assuming the Valley continues to grow in accordance with the current zoning and building standards. Results of the future conditions evaluation indicated that the existing problems would be exacerbated by additional development and some portions of the new anticipated growth would also be at risk of flooding.

RECOMMENDED IMPROVEMENTS

After evaluating the existing system and determining a number of deficiencies, improvement methodologies were evaluated and compared to the existing standards throughout the Ashley Valley. Three possible methodologies were identified: 1) do nothing, 2) preserve the drainages, and 3) divert and protect. Through numerous discussions with the elected officials and staff, a hybrid improvement methodology was identified. The hybrid methodology focused on preserving the natural drainages wherever possible, and diverting water around existing highly developed regions only when natural drainages could not be restored.

Using the selected improvement methodology, a series of potential improvements were input into the model and evaluated for potential benefit, cost, and risk. Through an iterative trial and error process a total of 100 recommendations were developed. The recommendations consist of preserving natural drainages, which are identified in this report, converting existing irrigation canals into storm water channels, building new storm water channels, upgrading stream crossings, as well as constructing a series of detention and debris basins.

OPINION OF PROBABLE COSTS

Each recommendation provided in this report includes an estimated cost to design and construct the improvement. The total improvement costs to correct the deficiencies along Ashley Creek are estimated to be $189,658,000 which includes the construction of two dams to regulate the flow. The proposed improvements to Ashley Creek will not only provide flood protection but will also provide many acres of wetlands, as well as valuable open space for the community to enjoy. An additional $15,366,813 will be required to construct the debris / detention basins as well as new channels to divert storm water. Finally, this report recommends that over 60 crossings be upgraded to ensure that critical transportation corridors remain passable during large storm events. Crossing upgrades are estimated to cost $4,418,063 for an average protection to the 25-year storm event. These costs are based on 2008 construction cost estimates at an Engineering News Record construction cost index of 8,184.94.

Chapter 1 INTRODUCTION

This study provides a comprehensive evaluation of current and future storm water conditions of the Ashley Valley (Valley), including a short history of the area, potential areas of development, and historical, current and projected storm water flows. This study includes a review of the current storm water management practices as well as recommendations for future storm water management strategies. The report also identifies the major capital improvements within the Valley that will be required to manage future storm water runoff effectively.

Each of the recommended capital improvement projects has been identified to provide flood protection for certain hydrologic conditions that are anticipated to only occur once every 10, 25, 50, 100, or 500 years depending on the structure. The recommended improvements will not protect the Valley from all flood damage during all flood events. Rather, the recommendations are intended to greatly minimize flooding during typical large precipitation events and lessen the damage that will occur during the most extreme precipitation events. Additionally, this report focuses on the large-scale flooding concerns throughout the Valley. This study does not examine, evaluate, or provide recommendations to prevent or minimize localized flooding. In summary, the recommendations in this report are intended to manage the risks associated with large precipitation events and reduce flooding damage, but will not protect the entire Valley from all precipitation events.

1.1 HISTORY

Valley is located in north-central Uintah County in eastern Utah, approximately 175 miles east of Salt Lake City and in close proximity to the Colorado state line. It is bordered on the north by the Uintah Mountains, one of the few mountain ranges in the world which lies in an east-west, rather than the more common north-south, direction. The Book Cliff Mountains lie to the south and Blue Mountain to the east. The Valley, and Ashley Creek, a major water course in the Valley, are named after William H. Ashley, an early fur trader who entered the area in 1825 via the Green River. In 1861, President Abraham Lincoln set the area aside as the Uintah Indian Reservation, and appointed Captain Dodds as Indian agent for this reservation.

When Dodds retired, he moved to Valley to raise livestock, along with other agency workers. They arrived on February 14, 1873 and settled on the banks of Ashley Creek. Dodds built the first cabin in the Valley, located about four miles northwest of present day Vernal. Many trappers, prospectors, and home seekers moved in and out of the Valley until 1878. Alva Hatch came to the Valley looking for a place to homestead in May 1978. He returned later with his family and his father, Jeremiah Hatch. The fall of 1879 brought many settlers to the Valley.

As the Valley was settled, large portions of the basin developed into crop lands. The arid climate severely limited the type and quantity of crops that could be grown. To increase the agricultural productivity of the area, a series of irrigation canals were constructed to provide water to the crops. Today, modern irrigation channels traverse the entire basin providing water to the large majority of crops. Most of these canals capture large volumes of storm water during rain and snow events. The current irrigation system within the Valley has a dual purpose: to convey essential water to the crops and to safely route storm water through the basin.

The first town in the basin, Vernal, was incorporated in 1897. In 1948, Vernal had its first oil boom, and from that time on it has been a boom and bust town. Naples was the second incorporated area in the Valley, named after the prominent city in Italy. A thriving tourist business located near the popular Dinosaur National Monument, combined with livestock and agriculture production, have helped to diversify the local economy and in turn keep Vernal, Naples and the surrounding area prosperous.

Maeser is an unincorporated community of the Valley, located approximately three miles northwest of Vernal. The community was named after an educator by the name of Karl G. Maeser. The community of Maeser has a total area of approximately 6.5 miles and is located north of State Route 121 on the west side of the Valley.

1.2 THE NEED FOR STORM DRAIN MASTER PLAN

Presently, the majority of land within the Valley is open space or has been developed for agricultural purposes. However, as the local economy continues to diversify, the Valley is becoming increasingly urbanized as agricultural fields and open space are transformed into incorporated towns and cities. This increased development will affect the storm water runoff patterns within the region. Without a master plan, individual developments will be solely responsible for storm water run-on and run-off management strategies. This microscopic approach to storm water management often leads to costly and ineffective management styles. In some cases, different storm water mitigation approaches within the same basin can conflict with one another, creating potentially hazardous results.

1.3 PURPOSE

Currently, the Valley does not have a comprehensive basin-wide master plan. The existing storm water facilities are currently owned by numerous entities, including: Vernal City, Naples, various irrigation companies, and Uintah County. As the region continues to grow, the affects of development will intensify and the need for these networks to work together will increase dramatically. This master plan is intended to identify the existing backbone for the storm water conveyance and detention network throughout the basin and provide a list of the capital facilities that will be required to ensure the networks work together and effectively manage future storm water flows.

1.4 METHODOLOGY

This master plan begins by identifying the study area and defining the drainage boundaries of the Valley. Critical hydrologic parameters such as inflow, rainfall intensity, duration, and frequency ("design storm"), land use, soil type, and contour elevation data were collected as a basis for the analysis. This data was then compiled into a geospatial database, or Geographic Information System (GIS), to perform advanced computations and spatial analysis, described in more detail later in this report.

The design storms used in this study are established based upon the intensity/duration/frequency (IDF) curves that are generated by the National Oceanic and Atmospheric Administration (NOAA) for the Vernal Airport weather station. Land use, soil types, and contour elevation data are gathered from county, state, and federal agencies, as appropriate.

Drainage basins within the study area are identified intelligently to provide sufficient detail, while not over-complicating the modeling process. A hydrologic model, utilizing the defined parameters, is then used to determine the runoff potential from the individual basins by routing the flows through a series of irrigation canals, natural ditches and creeks, pipes and detention facilities.

Areas and types of future development are identified and the modeling process repeated to observe the affects of the anticipated development. Where the model indicates future flooding will occur, flows are re-routed or conveyance capacities increased to alleviate the problems. From the model, a list of the required capital facilities necessary to prevent future flooding is provided as well as the estimated cost of each improvement.

1.5 OBJECTIVES OF THE STUDY

The objectives of this study include the evaluation of the existing storm water facilities and the recommendation of improvements to be made in the existing storm water conveyance network to correct existing deficiencies as well as to convey future flows. These objectives will be accomplished by evaluating the effectiveness of the current faculties through advanced modeling.

1.6 LIMITATIONS OF THE STUDY

This study provides an extensive storm water evaluation of the entire Valley, and is designed to provide details inherent to the storm water system as a whole. As such, this model should be used in conjunction with site-specific hydrology studies; it is not designed to replace such studies.

The Valley encompasses an area of approximately 55 square miles; many of the storm events only affect a portion of the area or affect different regions of the basin uniquely. This study assumes a uniform rainfall distribution over the entire Valley. It is assumed that this form of modeling will provide accurate or slightly conservative estimates of storm water runoff for the large design storms.

The best calibration was achieved with a zero flow from the Steinaker Dam. This study does not attempt to delineate flooding that would result from a breach of the dam and assumes that flows that may result from controlled discharges are properly regulated and controlled. This study does, however, assume that the Steinaker Feeder Canal to the reservoir does not divert any water away from the flood during the storm events which produce flows in Ashley Creek in excess of what gauging stations have recorded to date.

Due to the size of the study area, the majority of the drainage basins were delineated using a two-meter digital elevation model (DEM), or aerial topology, that was provided by Uintah County, instead of traditional ground surveying methodology. Information regarding the location, capacity, and discharge points for major canals within the Valley is based on information obtained from operational personnel. Knowledge from City and County staff was used to determine existing known problem areas and other pertinent information in order to calibrate the model effectively.

Chapter 2 STUDY AREA CHARACTERISTICS

2.1 INTRODUCTION

The purpose of this chapter is to describe the pertinent physical, environmental, hydrologic, and land use characteristics of the study area to provide a basis for storm water flows outlined in this report. This chapter identifies the study area and drainage basin boundaries for the hydrologic analysis. It also describes the land use and soil data used to calculate runoff coefficients, and it outlines the hydrologic patterns that form the basis for the selection of IDF curves.

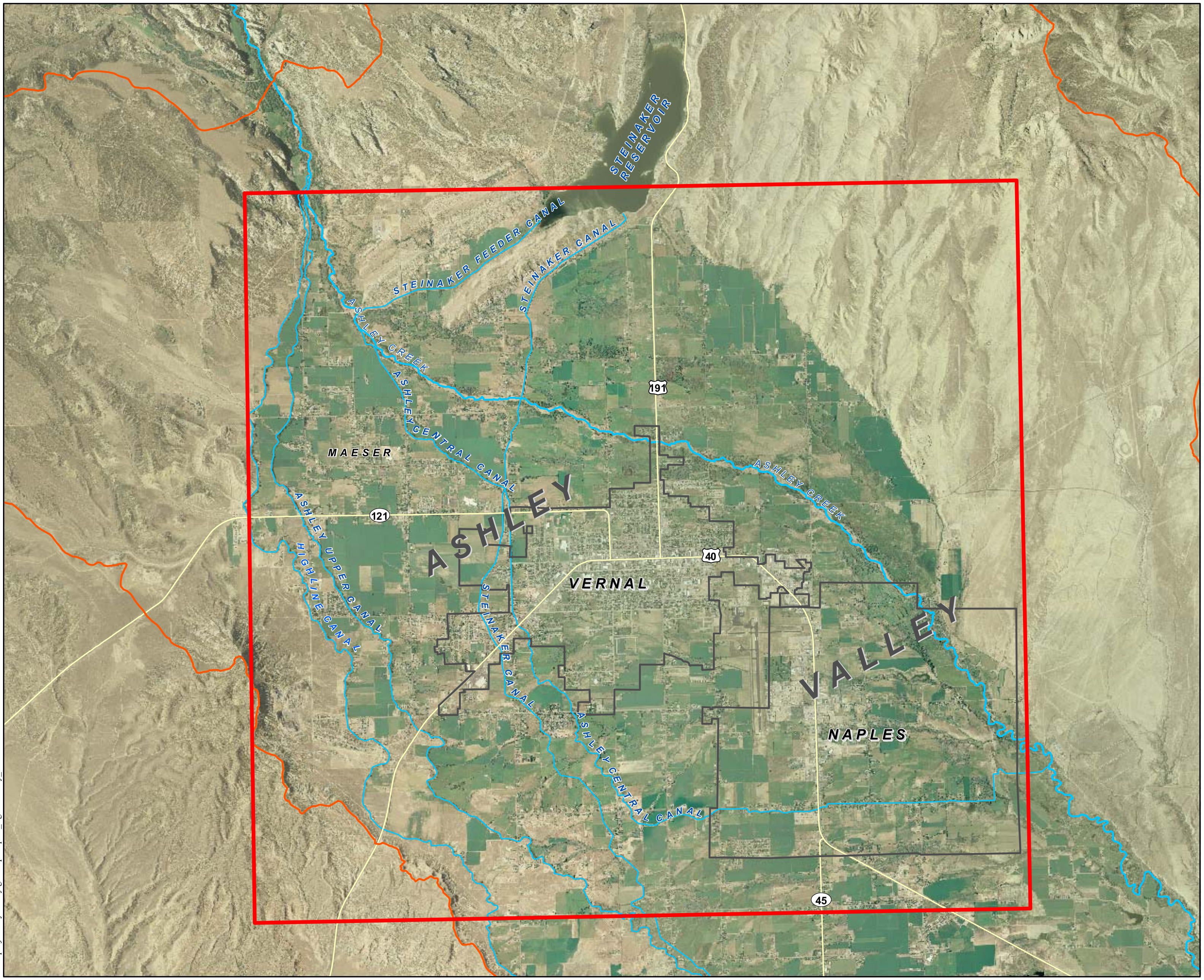
2.2 STUDY AREA BOUNDARIES AND COMPOSITION

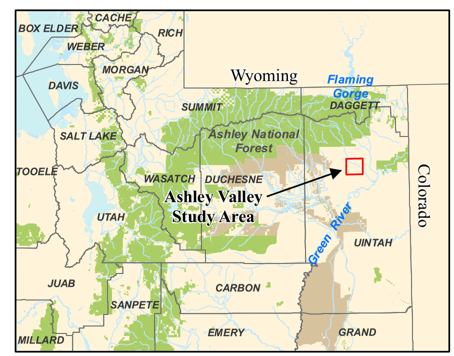
The study area encompasses the city limits of Naples to the east, extends past Vernal to the bench on the west, and includes the Valley area between 3500 South to the south and Steinaker Reservoir to the north, for a total area of approximately 55 square miles. To ensure complete and accurate results, the drainage basins were extended to the ridgelines surrounding the Valley as shown in Figure 2-1. The full drainage area of Ashley Creek was not modeled due to the large contributing areas and numerous control structures along the stream course. Instead, stream gauge data located in the northwest corner of the Valley was utilized to provide accurate inflow data, as described in more detail later in this report.

The majority of the Valley consists of rural undeveloped lands or developed lands used for agricultural purposes. Portions of the central Valley have developed into cities that include commercial and industrial land uses. It is anticipated that the majority of future growth will result from the cities expanding from the center of the Valley into the outlying farmlands.

FIGURE 2-1
STUDY AREA
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008





Source: AGRC NAIP 2006 and Uintah County GIS.

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2.3 Geographical Setting

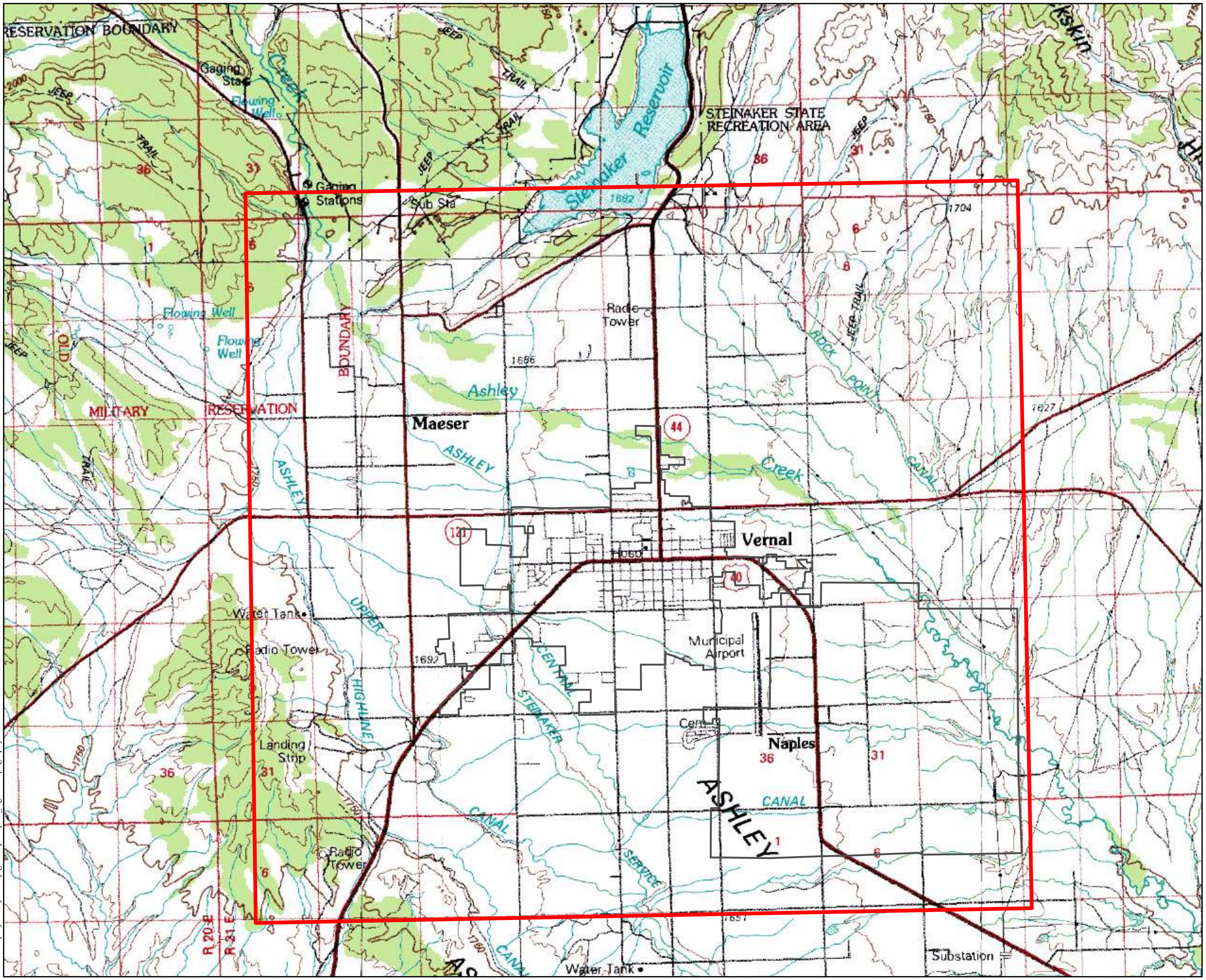
The Valley is located in Uintah County in northeastern Utah near the Colorado border, north of the Green River and south of Flaming Gorge Reservoir. The region is a high-elevation (+5,000 feet) arid basin surrounded by mountains that are part of the larger Uinta Mountain Range to the north and extend over 1,500 feet above the Valley floor.

2.3.1 Topography and Drainage

Ashley Creek provides the major drainage through the Valley, which is located within the greater Lower Ashley Creek watershed in the Ashley Creek/ Steinaker Reservoir/ Coal Mine Basin-Ashley Creek sub-basins. Ashley Creek generally flows in a southeasterly direction from the Ashley National Forest to the northwest, meanders through the Valley and exits at the southeast corner of the study area, eventually reaching the Green River.

Flows from Ashley Creek are diverted at numerous locations along the river for irrigation needs and other purposes. To provide for adequate water supply year-round, the Steinaker Dam and Reservoir were constructed in 1968 to store and distribute the excess spring flows of Ashley Creek. Water from Ashley Creek is diverted by Fort Thornburgh Diversion Dam, located approximately four miles northwest of Vernal and stored by the Steinaker Dam and Reservoir, located off-stream in Steinaker Draw about 3.5 miles north of Vernal. From the diversion dam, the water is conveyed eastward to the reservoir through the 2.8 mile-long Steinaker Feeder Canal. Reservoir water is released to Steinaker Service Canal and conveyed south 11.6 miles to other canals and ditches. Steinaker Reservoir has a total capacity of 38,173 acre-feet, and a surface area of 820 acres.

The Valley floor ranges in elevation from 5,000 feet to 5,600 feet. The basin is surrounded by mountains as high as 7,000 feet. The Valley topography and major drainage features are shown in Figure 2-2.



Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 2-2
VALLEY TOPOGRAPHY
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Study Area Boundary
- Municipal Boundary

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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2.3.2 Soils

The type of soil can have a great affect on the quantity of storm water runoff in an area. Tightly bound clay soils generally have very high runoff potential while loose, well-graded sands generally have very low runoff potential. Based on the USDA Natural Resources Conservation Service (NRCS) soil maps, the Valley contains approximately 80 different soil types. For the purposes of quantifying storm water runoff it is not necessary to treat each soil type individually. Instead, the soils can be grouped with other soils that share similar hydrologic properties. The NRCS, formerly the Soils Conservation Service (SCS), classifies soils into four hydrologic soil groups. This classification system will be used for the purposes of this study, and is based on the soil's runoff potential as defined below:

Group A is sand, loamy sand or sandy loam types of soils. These soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist primarily of deep, well to excessively drained sands or gravels and have a high rate of water transmission.

Group B soils are silt loam or loam. These soils have a moderate infiltration rate when thoroughly wetted and primarily consist of moderately drained soils with moderately fine to moderately coarse textures.

Group C soils are sandy clay loam. These soils have low infiltration rates when thoroughly wetted and primarily consist of soils with a layer that impedes downward movement of water and soils with moderately fine to fine structure.

Group D soils are clay loam, silty clay loam, sandy clay, silty clay or clay. These soils have the highest runoff potential and very low infiltration rates when thoroughly wetted. They primarily consist of clay soils with a high swelling potential and/or soils with a permanent high water table.

Figure 2-3 shows the soil classification groups throughout the Valley. The majority of soils in the Valley are classified as types C and D with moderate to high runoff potential.

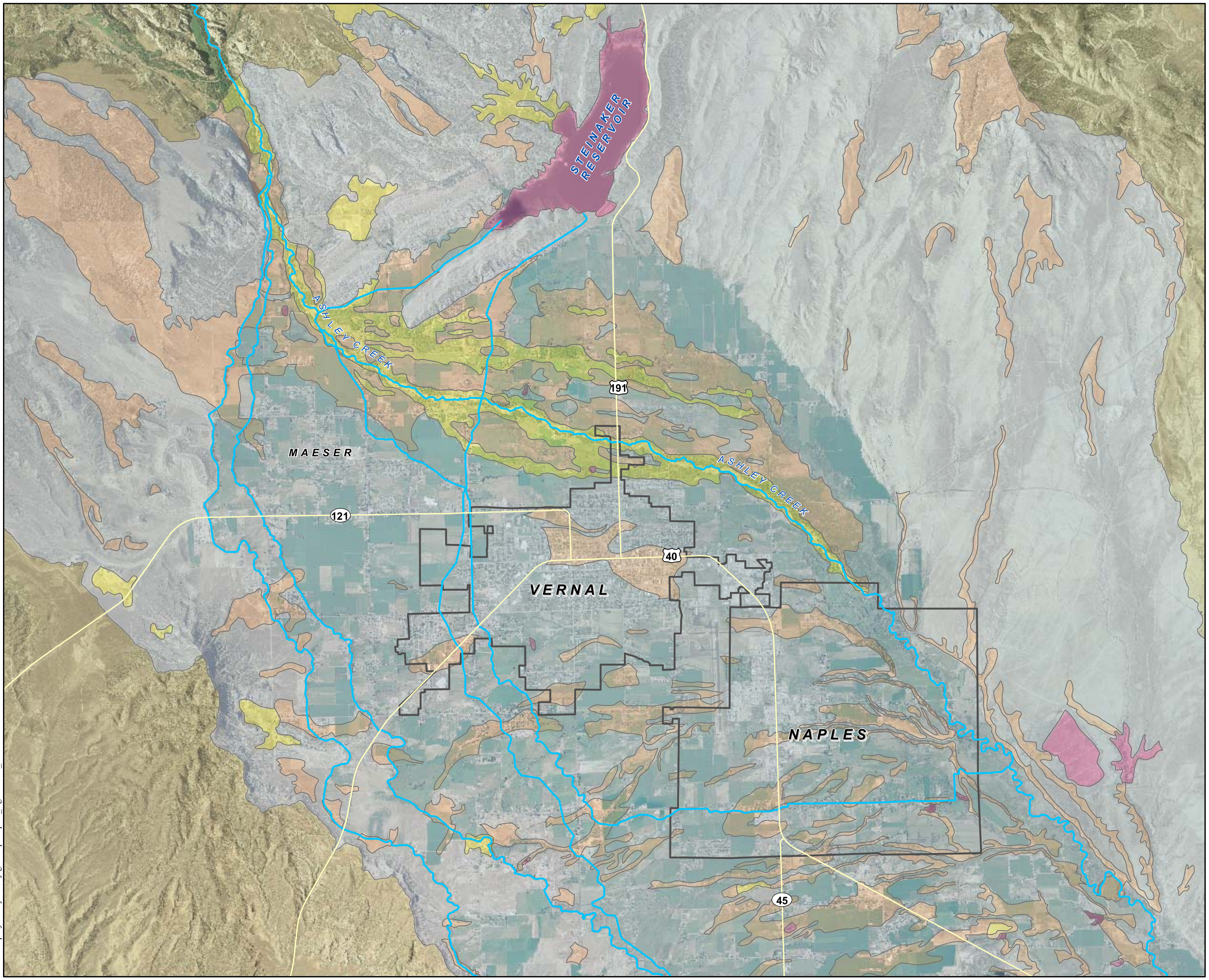

FIGURE 2-3
SOIL CLASSIFICATIONS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Major River or Canal

Municipal Boundary

Hydrologic Soil Group

A - Low runoff potential

B - Moderate runoff potential

C - Moderate to High runoff potential

D - High runoff potential

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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2.3.3 VEGETATION

In addition to soils, the type of vegetation throughout an area can have a large affect on how rainfall is captured and the resulting runoff rates. Dense vegetation will generally trap a portion of rainfall as well as slow the rate at which the water can run off the basin and into channels. Conversely, bare soils or soils with little vegetation will generally hold less water and runoff velocities will be higher.

Vegetation in the Valley is widely varied. Being an arid desert, the region consisted primarily of prairie grasses and brush prior to development, except near the natural water courses where the vegetation is generally dense compared to the rest of the area. As the Valley was settled, however, large sections of the region were developed into irrigated crop lands. Mature crop lands generally provide dense vegetation while new crops or tilled fields between seasons will provide very little, if any, vegetation. The perimeter of the Valley is bounded by mountains with steep slopes. The mountainsides are largely un-vegetated hillsides, and as a result, have a high runoff potential.

2.4 CLIMATE

The Valley is a high desert with an arid climate. On average, the Valley receives less than 9 inches of rainfall annually. The climate is characterized by hot, dry summers, moderate autumns, cold winters with intermittent snow storms, and relatively wet springs during which the majority of rainfall occurs. Table 2-1 shows the average monthly temperature range and average precipitation for the area.

Table 2-1 Climate Data

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	30	37	51	62	73	82	90	87	78	64	46	33	61.2
Average Min. Temperature (F)	4.9	11	22	30	39	45	52	50	41	31	20	9.3	29.5
Average Total Precipitation (in.)	0.5	0.5	0.7	0.8	0.8	0.7	0.5	0.7	0.9	1.1	0.6	0.6	8.31
Average Total Snowfall (in.)	4.7	2.9	1.6	0.2	0	0	0	0	0	0.3	0.9	4.6	15.3
Average Snow Depth (in.)	1	1	0	0	0	0	0	0	0	0	0	0	0

2.5 HYDROLOGY

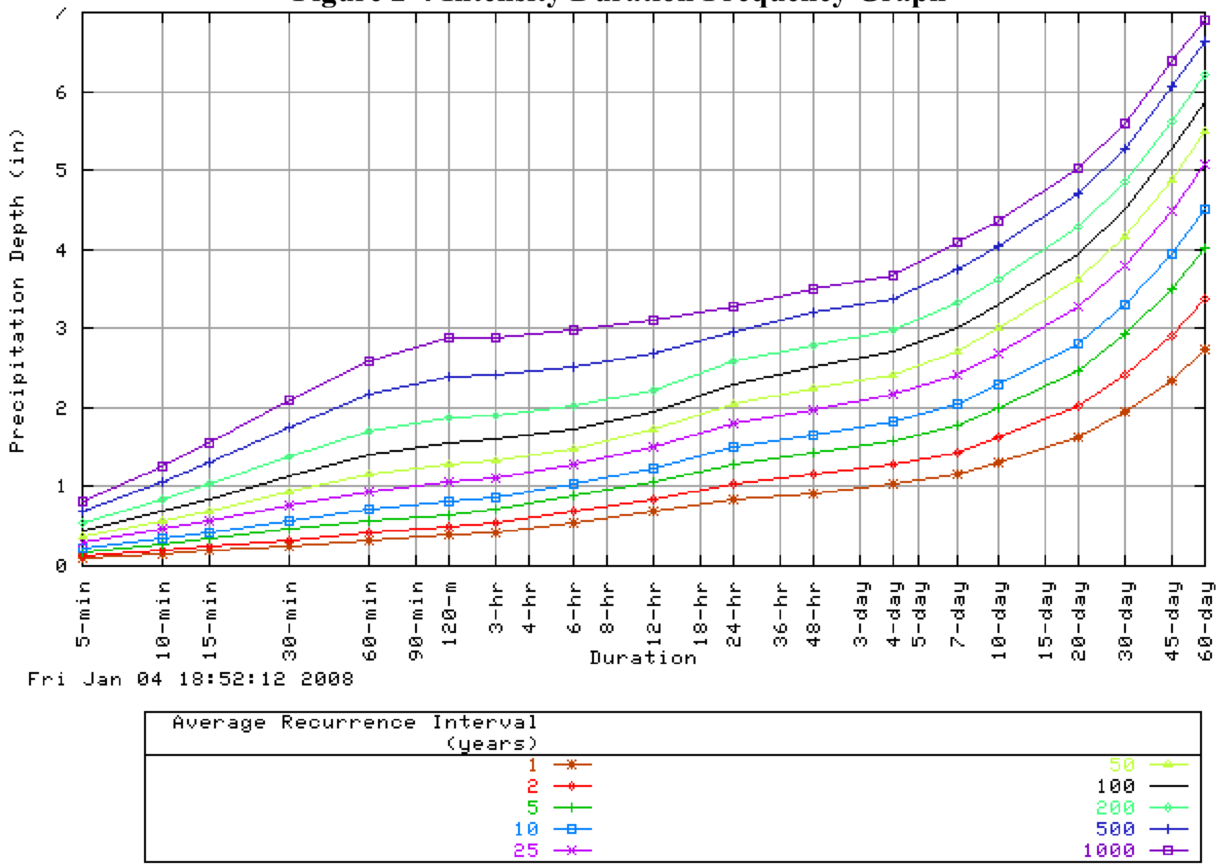
Storm water master planning and the design of drainage facilities are highly dependent upon the selection of the "design storm". This storm, typically expressed in terms of its expected *recurrence interval* (e.g., 10 years), is used to determine rainfall intensity. The recurrence interval, also called a *return period* or *event frequency*, is the length of time expected to elapse between rainfall events of equal or greater magnitude. For example, a 10-year recurrence interval represents a storm event that is expected to occur once every 10 years, on average. This does not imply that two storm events of that same size will not occur in the same year, nor does it mean that the next storm event of that size will not occur for another 10 years. Rather, there is a 10-percent chance of occurrence in any given year. The length of the design storm also affects storm flows and runoff. For the purposes of this study, the 24-hour duration storm has been selected from the intensity/duration/frequency (IDF) data.

The IDF curves are created from precipitation records collected by the National Oceanic and Atmospheric Administration (NOAA). The precipitation station with the longest history, and the greatest amount of data, within the Valley is the Vernal Airport Station (Station 42-9111). The resulting rainfall depths and intensities for a range of durations for each return period are shown in Figure 2-4 and Table 2-2.

Table 2-2 Intensity Duration Frequency Data, Vernal Airport

Precipitation Frequency Estimates (inches)																			
		Storm Duration																	
		5	10	15	30	60	120	3	6	12	24	48	4	7	10	20	30	45	60
		min	min	min	min	min	min	hr	Hr	hr	hr	hr	day	day	day	day	day	day	day
Return Period (years)	1	0.1	0.15	0.19	0.26	0.32	0.38	0.43	0.55	0.68	0.83	0.92	1.03	1.16	1.31	1.63	1.96	2.35	2.74
	2	0.13	0.2	0.24	0.33	0.41	0.48	0.54	0.69	0.85	1.03	1.15	1.28	1.44	1.63	2.02	2.42	2.91	3.38
	5	0.18	0.27	0.34	0.46	0.56	0.65	0.71	0.88	1.06	1.28	1.43	1.58	1.77	2.0	2.47	2.93	3.5	4.03
	10	0.23	0.34	0.43	0.58	0.71	0.8	0.87	1.04	1.24	1.5	1.66	1.82	2.04	2.3	2.82	3.31	3.94	4.5
	25	0.3	0.46	0.57	0.77	0.95	1.05	1.11	1.29	1.5	1.8	1.98	2.16	2.41	2.69	3.27	3.8	4.49	5.08
	50	0.37	0.56	0.7	0.94	1.16	1.28	1.33	1.49	1.72	2.04	2.24	2.43	2.71	3.0	3.61	4.16	4.89	5.49
	100	0.45	0.69	0.85	1.14	1.42	1.55	1.59	1.73	1.95	2.3	2.51	2.71	3.01	3.31	3.95	4.51	5.27	5.87
	200	0.54	0.83	1.02	1.38	1.71	1.88	1.9	2.01	2.21	2.58	2.79	2.99	3.33	3.63	4.28	4.85	5.63	6.22
	500	0.69	1.05	1.3	1.76	2.17	2.4	2.42	2.52	2.67	2.96	3.19	3.37	3.75	4.04	4.72	5.28	6.07	6.62
	1000	0.82	1.25	1.55	2.09	2.59	2.88	2.89	2.99	3.12	3.28	3.51	3.68	4.08	4.37	5.04	5.59	6.38	6.9

Figure 2-4 Intensity Duration Frequency Graph

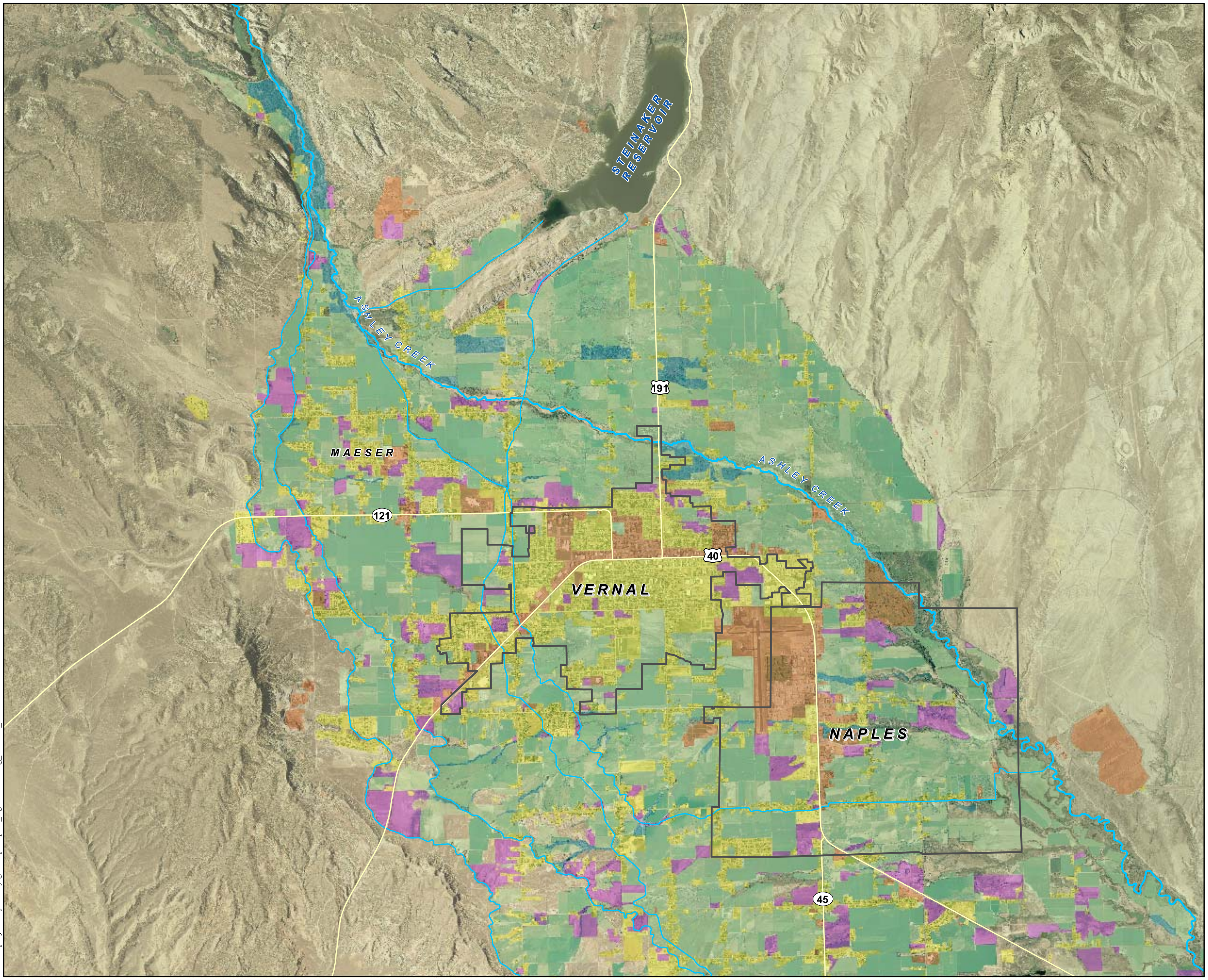


2.6 MAJOR DRAINAGE BASINS

In order to model storm water flows in the Valley, a series of drainage basins are required that accurately reflect the true drainage boundaries of the area. The Valley was delineated intelligently into 200 basins using a high-resolution digital elevation model. Each of the basins contains an outlet which routes the flows from each basin into existing channels, pipes, natural streams or other drainages.

2.7 EXISTING LAND USE

The majority of the Valley is rural and currently used for grazing or agricultural purposes. Approximately 21% of the Valley has been developed into cities including commercial, industrial and other land intensive uses. Figure 2-5 shows the current land uses in the Valley, divided into the following five categories: irrigated/cultivated, residential, riparian, urban, and water. The open land currently used for agricultural purposes currently allows much of the storm water to infiltrate into the soil.



Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 2-5
EXISTING LAND USE
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Major River or Canal

Municipal Boundary

Land Use

Irrigated

Not in Use

Residential

Riparian

Urban

N
W E
S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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2.8 EXISTING SYSTEMS

As development occurred in the Valley, numerous structures were built which altered the historic storm water flow patterns. Irrigation canals have been constructed throughout the Valley which capture and convey storm water runoff and divert them to agricultural fields.

The central portion of the Valley has been developed into the cities of Vernal and Naples. These and other developed regions (i.e. Maeser) have increased the amount of impervious surface and, consequently, the amount of storm water runoff from these areas. To convey and control the increased storm water runoff, Vernal has installed a number of pipes that are networked throughout the city. In the unincorporated areas, development under Uintah County code required the construction of retention basins to retain storm water runoff in most of the large-scale developments within the Valley.

2.8.1 STORM DRAIN PIPE NETWORK

The majority of the drainages in the Valley are natural channels and irrigation canals. Small portions of the Valley have closed-conduit, piped, storm water conveyance to move storm water from the heart of the developed areas to the perimeter. The existing pipe networks generally convey water within the defined basins; the pipe networks do not currently move significant volumes of storm water between defined basins.

2.8.2 STORM DRAIN DETENTION FACILITIES

Uintah County requires complete retention of all storm water up to the 100-year event for all large developments located outside of the incorporated areas (i.e. Vernal or Naples City). This has resulted in a large number of local retention basins that minimize the volume of storm water that exits the site so long as the basins are properly maintained. The existing system also has a number of "natural detention basins" in the form of wetlands along natural channels within the Valley.

2.8.3 IRRIGATION CANALS

Meetings were held with the major irrigation companies to identify canals that affect the storm water runoff. Canal capacity and emergency turnout points were identified to improve the accuracy of the runoff flow rates. For the purposes of determining the worst case flooding potential, the analyses contained in this report assume the irrigation canals are full at the beginning of the storm event. The worst case flooding is then defined as the storm event plus the maximum turn out capacity within each basin.

2.8.4 NATURAL STREAMS

The natural stream channels throughout the basin provide the primary drainage mechanism to move water through the basin toward Ashley Creek. The natural channels vary from small depressions in the upper reaches of the basin to year round streams in the lower portions of the basin. Portions of the streams have been channelized as the basin developed. In places, the streams are routed through culverts or other structures. The minor culverts and ditch constructions were not accounted for in this macro-scale model. The larger structures such as major culverts, raised roadbeds or long sections of channelized stream were incorporated into the model. Figure 2-6 highlights the major natural stream channels throughout the basin.

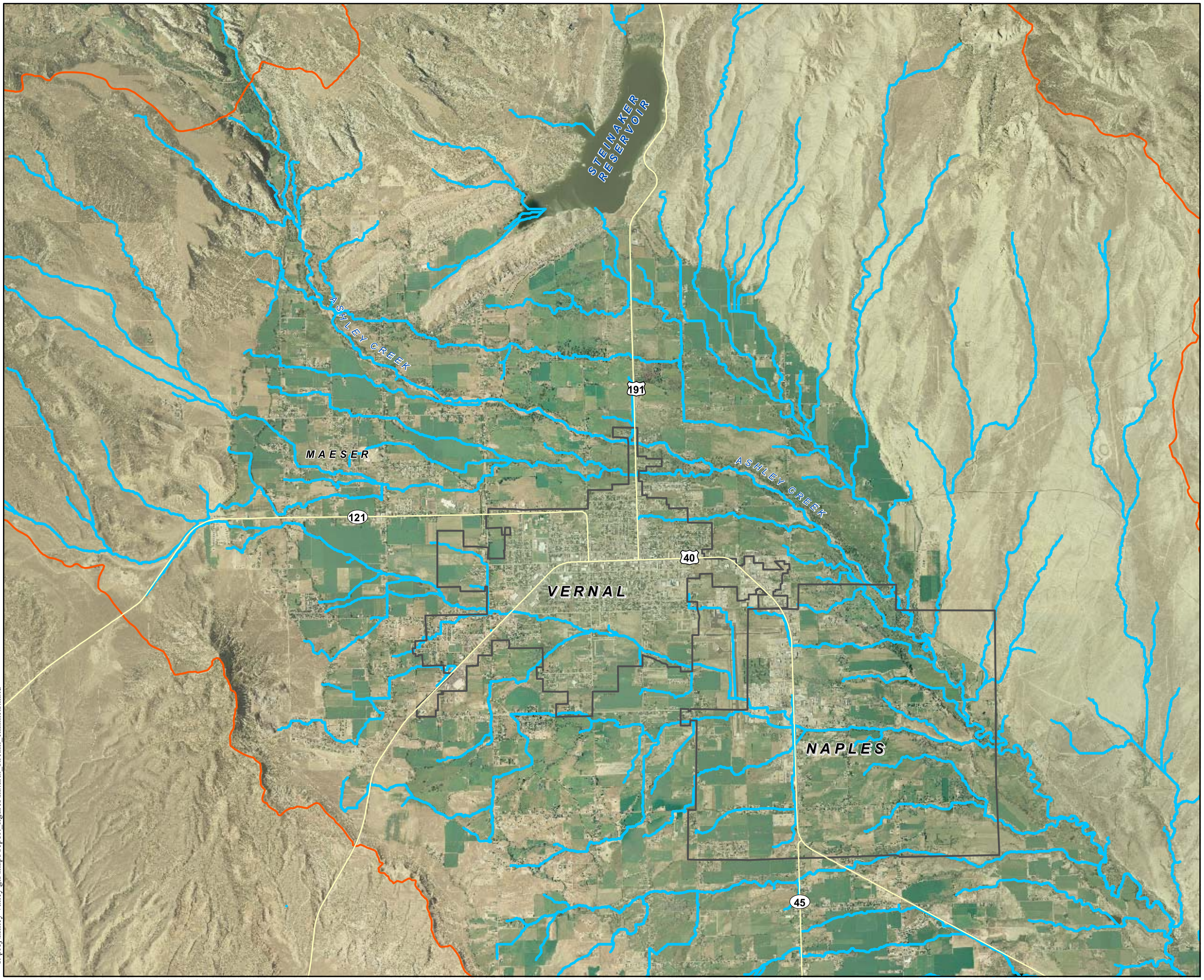

FIGURE 2-6
NATURAL STREAM CHANNELS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Natural Stream Channel
- Major Road
- Ridgeline/Watershed
- Municipal Boundary

N
W E
S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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2.8.4.1 Ashley Creek Inflow

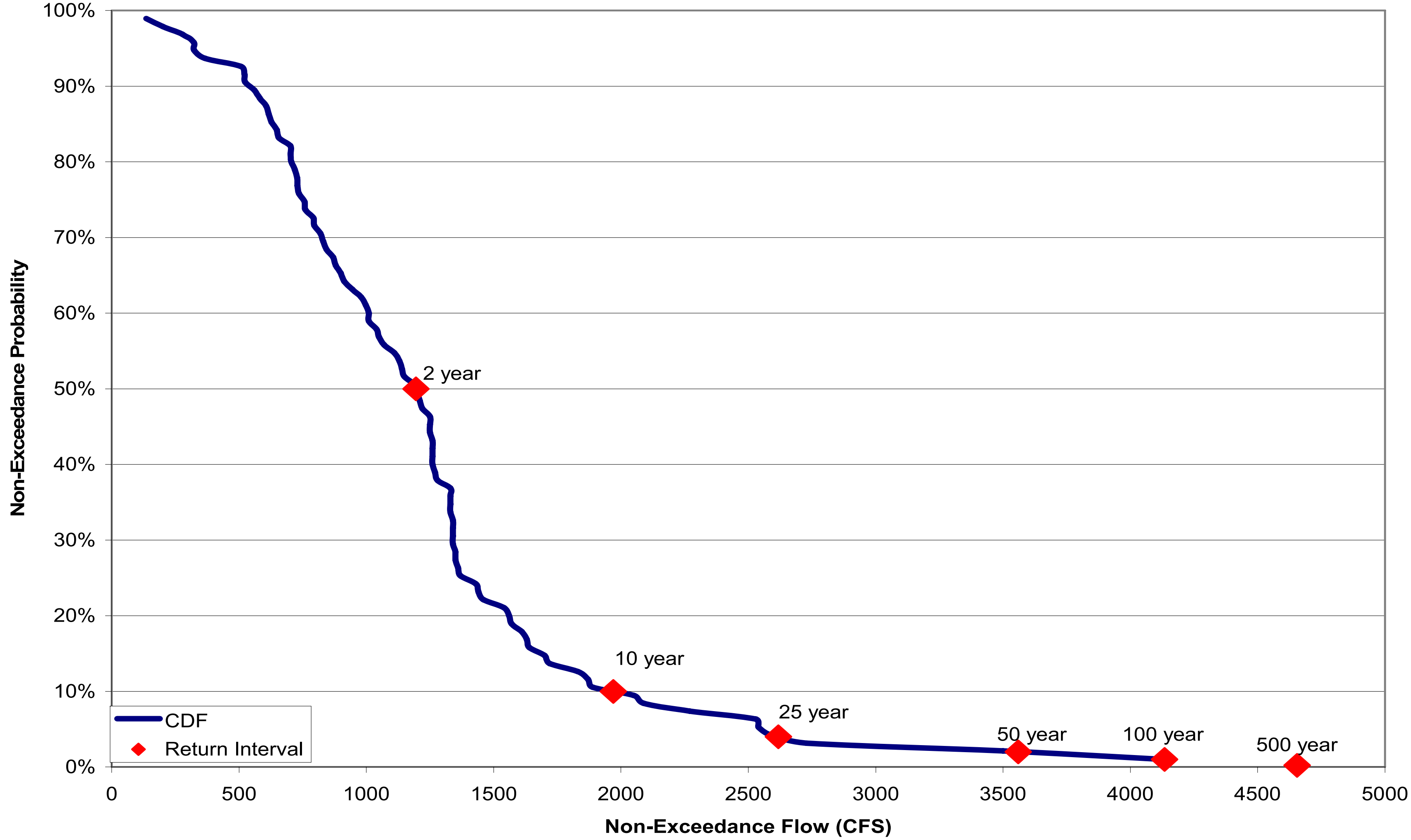
Similar to the irrigation turnouts, the inflow from Ashley Creek into the Valley is modeled as a steady state point flow. The inflow rate was established through statistical analysis of the recorded peak flows at the USGS gauging station (09266500) "sign of the main". The record contains the annual peak flows for approximately 96 years which were used to produce a cumulative distribution curve (CDF) of the flow exceedance probability. Linear interpolation and extrapolation algorithms were then used to determine the peak inflow at the upper reaches of the Valley. The CDF curve is shown in Figure 2-7 and the inflow results for each storm intensity are shown in Table 2-3.

Table 2-3 Ashley Creek Inflow Rate Summary

Probability	Return Interval	CFS	Calculation
0.2%	500	4,655	Extrapolation
1.0%	100	4,134	Extrapolation
2.0%	50	3,560	Interpolation
4.0%	25	2,618	Interpolation
10.0%	10	1,970	Interpolation
50.0%	2	1,195	Interpolation

* Flows were extrapolated when insufficient data was available for interpolation

Figure 2-7 Upper Ashley Creek Non-Exceedance Probability Distribution Function



Chapter 3 STUDY AREA GROWTH

3.1 OVERVIEW

The Valley has been experiencing recent population growth. This growth is expected to continue through the next 50 years as the local economy continues to diversify and local oil production increases. Many portions of the Valley are developing to house and serve this increasing population. This section presents the historic population trends as well as the population projections based on the Utah Governors Office of Planning and Budgets 2005.

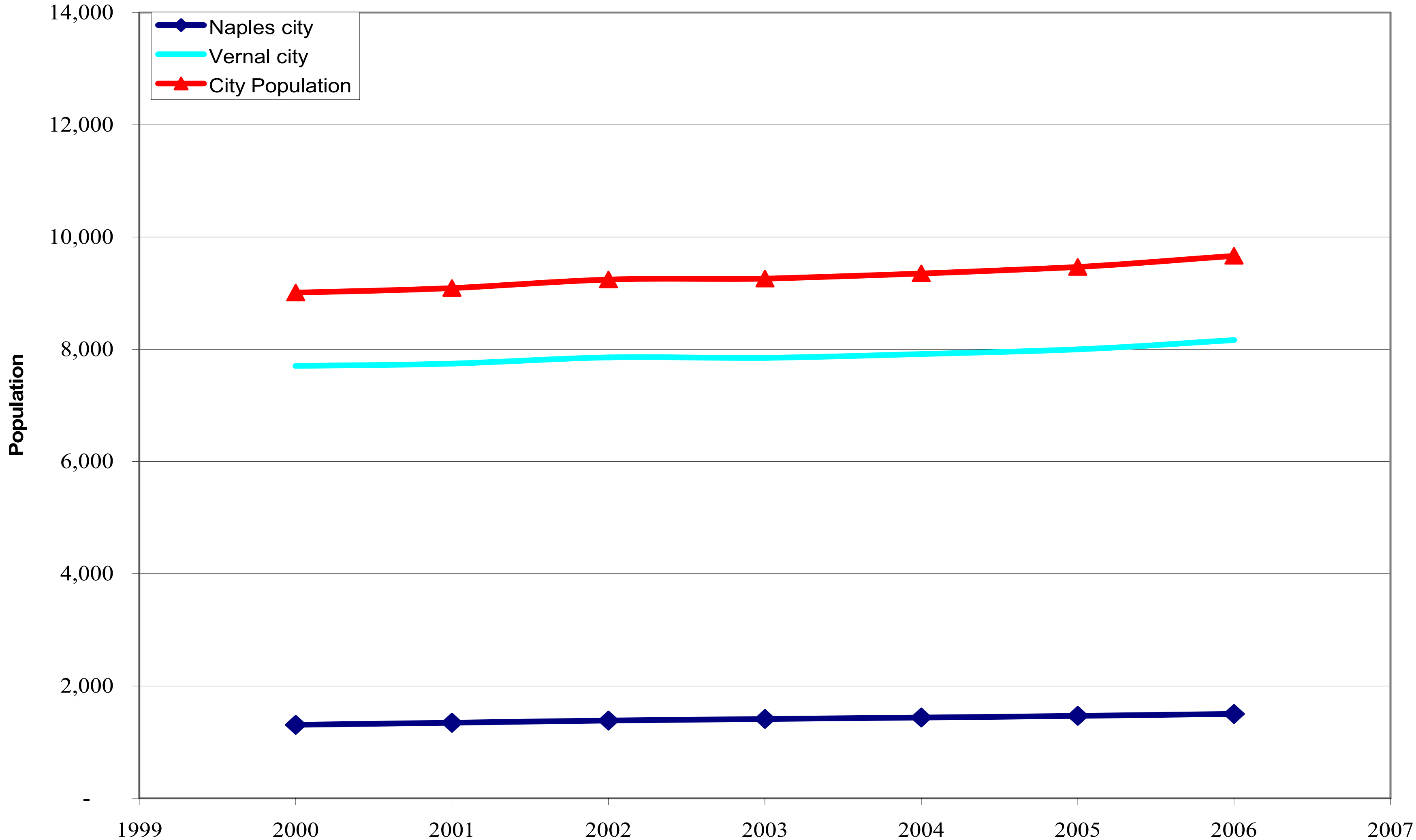
3.2 HISTORIC POPULATION TRENDS

The majority of the Valley's population resides within the cities of Vernal and Naples. Furthermore, the growth projections of the cities are likely indicative of the growth throughout the adjoining unincorporated areas of the Ashley Valley. The population within Vernal and Naples has grown by more then 500 people from 2000 to 2006 according to the State Governors office. Local officials indicate the growth rate has been much higher. Below, Table 3-2 presents the Governors population estimates for the cities of Vernal and Naples.

Table 3-1 Historic Population Growth

	2000	2001	2002	2003	2004	2005	2006
Naples City	1300	1343	1384	1413	1439	1466	1502
Vernal City	7714	7746	7856	7845	7912	7999	8163
City Population	9014	9089	9240	9258	9351	9465	9665
Growth Rate %		0.9%	1.66%	0.19%	1.0%	1.22%	2.11%

Figure 3-1 Historic Population Trend



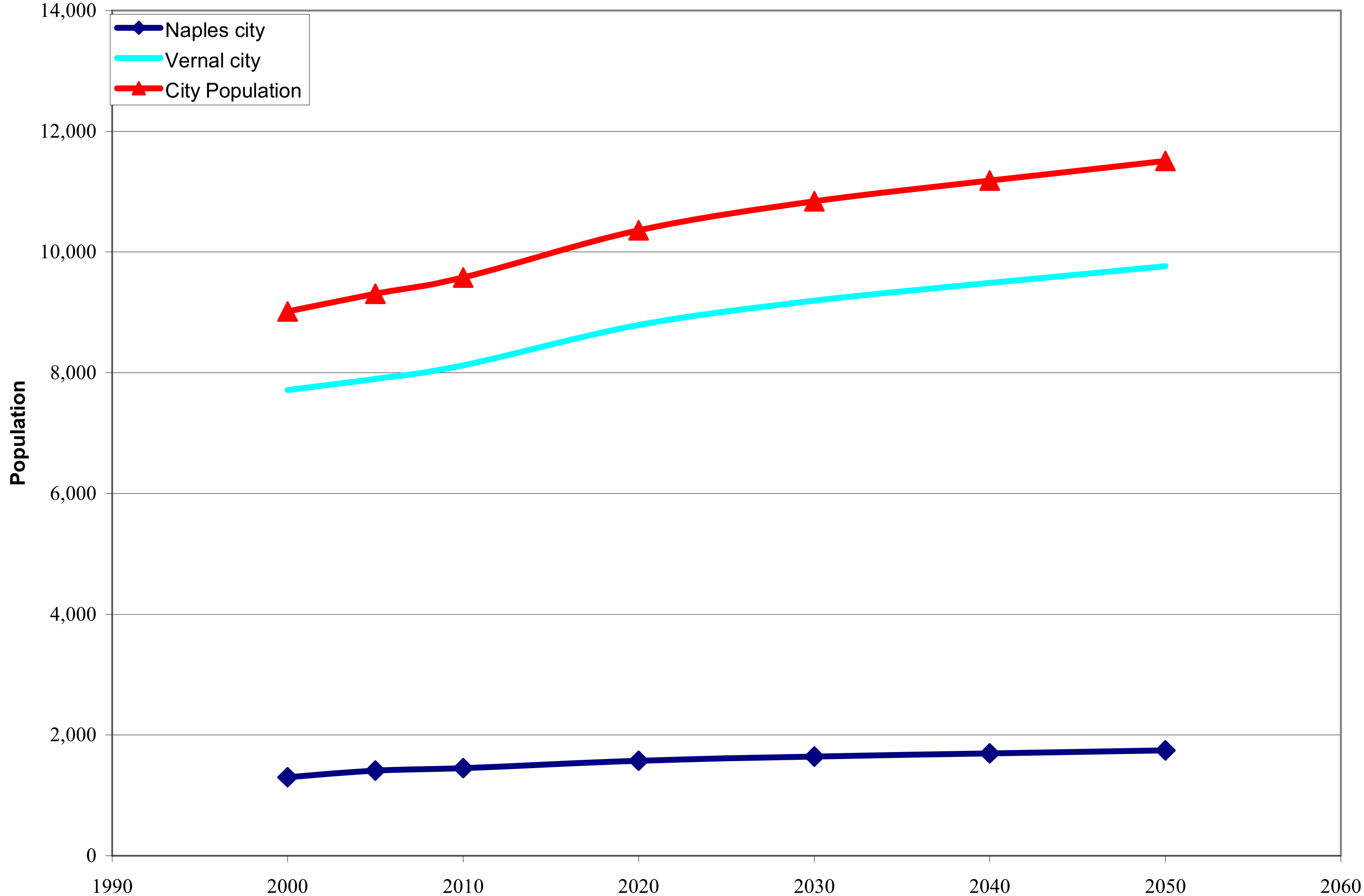
3.3 POPULATION GROWTH PROJECTIONS

Table 3-2 and Figure 3-2 below represent the population growth projections through the year 2050.

Table 3-2 Population Projections

	2010	2020	2030	2040	2050
Naples City	1,453	1,572	1,644	1,696	1,746
Vernal City	8,125	8,790	9,196	9,488	9,765
City Population	9,577	10,362	10,840	11,184	11,511
Growth Rate %	0.57%	0.79%	0.45%	0.31%	0.29%

Figure 3-2 Population Projections

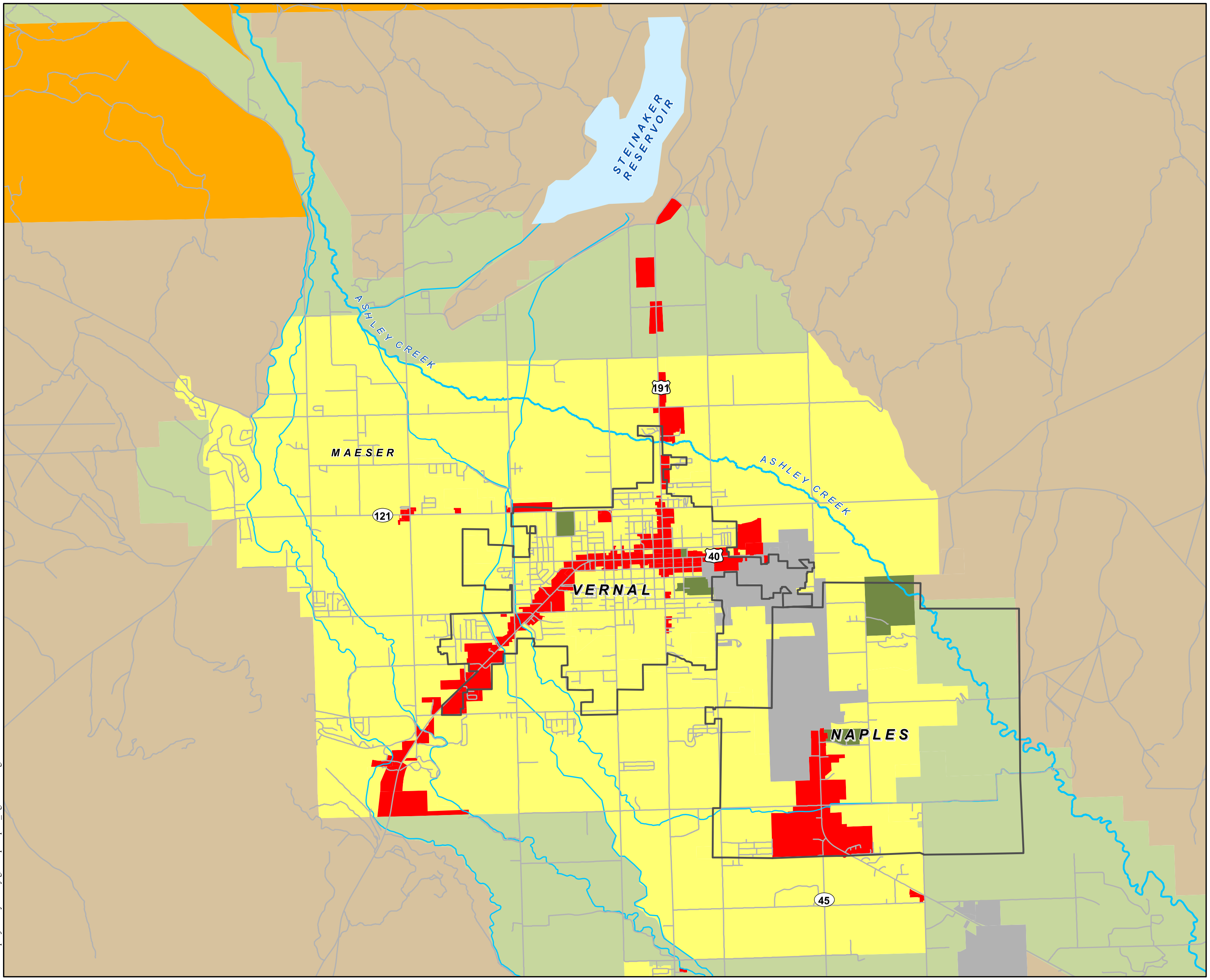


3.4 AREAS OF FUTURE GROWTH

In order to provide for future population growth it is anticipated that the developed areas of the Valley will continue to expand into areas currently used for agricultural and other undeveloped purposes. As this development continues, the area available for storm water to infiltrate naturally will decrease, artificially increasing the magnitude of runoff during future events.
In order to model the future runoff potential throughout the basin, this report assumes that the current zoning map, shown in Figure 3-3, represents how the Valley will eventually be developed at build-out.

FIGURE 3-3
FUTURE LAND USE
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Legend

- Road
- Major River or Canal
- Municipal Boundary

Zoning

- Agricultural
- Commercial
- Industrial
- Mining & Grazing
- Park
- Recreation, Forestry & Mining
- Residential

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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Chapter 4 STORM DRAINAGE MODELING METHODOLOGY

4.1 INTRODUCTION

The primary purpose of the storm water modeling is to develop criteria applicable to the design of the drainage facilities. This chapter discusses the modeling methods used and design criteria established to govern the modeling and establish the Level of Service (LOS) requirements for the existing and future build-out storm drain networks.

4.2 DESIGN CRITERIA

The following design criteria are used to complete the storm drain modeling:

1) The Level of Service for storm drain piping is to convey 110% of the 10-year storm event flows contributing to the pipe;

2) The Level of Service for irrigation ditches and artificial channels is to convey 100% of the 100-year event;

3) The Level of Service for natural channels is to convey 100% of the 100-year storm event;

4) The Level of Service for detention basins is to provide sufficient detention volume to contain the 100-year storm event with a peak outflow of less then pre-development levels;

5) The slope of the pipes is generally assumed to not be steeper than the slope of the ground surface above the pipe;

6) All closed conduit pipes are assumed to have a friction coefficient of 0.013;

7) Natural channels are initially assumed to have a friction coefficient of 0.035. During the calibration process, open channel friction coefficients may be adjusted to match field data;

8) Artificial channels are initially assumed to have a friction coefficient of 0.03.

4.3 HYDROLOGY MODEL

Given a number of parameters, the hydrology model predicts the volume of flow generated at any point in the watershed from the defined rainfall event. For this study, the soil conservation service (SCS) methods were selected to estimate the potential runoff. The SCS method is a series of empirical equations that were originally designed to compute the potential runoff from agricultural fields and other rural environments with similar characteristics to the Valley. This method has since been modified for use in both urban and rural settings and is the most effective method to estimate runoff from the drainage basins within the Valley.

The precipitation events of concern in this study are the extreme runoff events usually caused by cloudburst type storms that are characterized by short periods of high intensity rainfall. The SCS type II 24-hour storm distribution most closely reflects this type of event and is used to simulate the rainfall distribution within the model. Runoff from the drainage basins is computed using the SCS equation shown below and the runoff hydrograph. Peak discharge is estimated using simulated curvilinear hydrographs defined by the SCS TR-55 method. These methods account for the soil type, ground cover, ground slope, time of travel, and other parameters to accurately estimate the discharge hydrograph from each of the basins within the model.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where:
Q = Runoff depth (inches)
P = Precipitation (inches)
Ia = Initial abstraction
S = Storage or maximum retention

The discharge hydrographs from each of the basins are routed in the model to the lowest point in the basin, or the outlet node. The outlet nodes are then connected via hydraulic links which route the flow through the system to the bottom of the Valley drainage area.

4.4 Hydraulic Models

Each of the watershed discharge nodes are connected via hydraulic links. These links are pipes, ditches or natural channels. The depth of flow in each of the hydraulic links is calculated using Manning's equations for open channel flow shown below.

$$Q = \frac{1.49}{N} * A * \left(\frac{A}{P}\right)^{2/3} * S^{1/2}$$

Where:
Q = Flow in cubic feet per second
N = Friction coefficient
A = Area of flow
P = Wetted perimeter
S = Slope

The wetted perimeter and area of the natural channels are based on irregular channel shapes and cross-sections that are typical of those at the hydraulic link, or outlet node, location. The channel cross-sections are assumed to be uniform throughout the length of each hydraulic segment, and are typically modeled as trapezoidal channel sections.

Detention basins are also incorporated into the model to simulate the effect of the basins on the hydraulic routing. Inflow to the ponds is based on the routed basin discharge hydrographs. The outflow is based on the outlet structure type and depth in the pond. A series of time steps are used to calculate the flow differential through the pond to estimate the storage during the rainfall event.

4.5 MODEL IMPLEMENTATION

To reasonable model an area the size of the Valley while requires a large number of individual drainage basins to be identified. For the purpose of this study 200 drainages were defined throughout the Valley. The defined basins are shown in Figure 4-1. The basins vary in size from 100 acres to 1,470 acres with an average size of 490 acres. The flow path lengths of the basins vary from 1.3 to 32 miles in length with an average flow path of 5.9 miles. Modeling storm water runoff from 200 basins through a complex system of pipes, canals, streams, and ponds would be extremely difficult without the use of computer-based modeling software. The first step in creating a model is to calculate all of the input parameters that will be used to determine end results and evaluate various scenarios. A Geographic Information System (GIS) is best suited to accurately calculate all of the necessary input parameters for a model as large as the Valley. ESRI's ArcView^{tm} 9.2 software program was utilized to delineate the drainage basins from a highly accurate digital elevation model, and process the numerous variables discussed above.

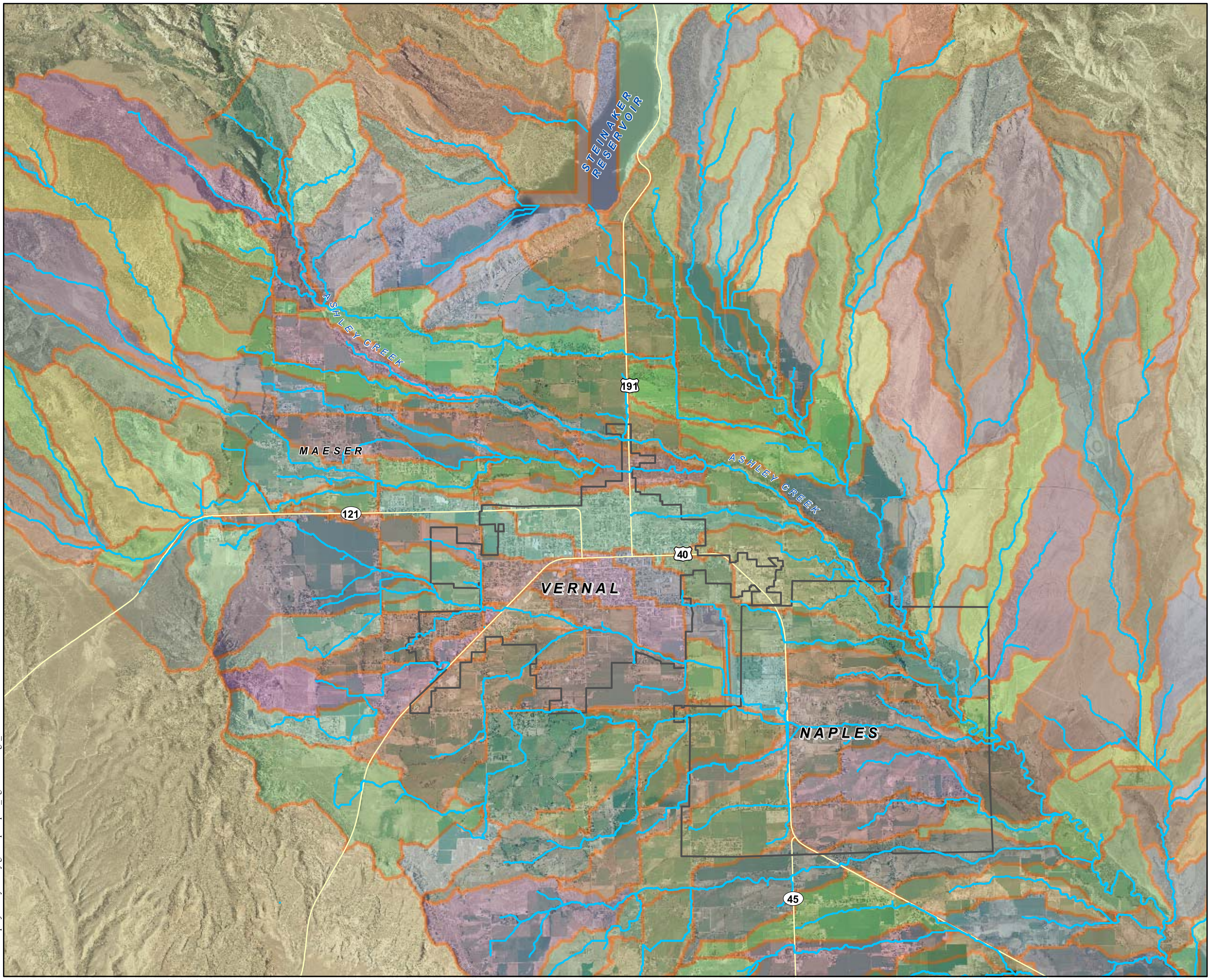

FIGURE 4-1
DRAINAGE BASINS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Stream Channel

Municipal Boundary

Drainage Basin

N
W E
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0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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Once these parameters were calculated, a storm and sanitary sewer modeling program called SewerGEMS^{tm} Version 8 was utilized to model the storm water runoff and routing throughout the Valley. SewerGEMS^{tm} was selected for several reasons, including its ability to (1) fully integrate with the GIS model, and (2) to provide calculation engines for runoff, open channel flow, pipe flow, and detention basin routing. Combining ArcView GIS software with the SewerGEMS modeling package results in an integrated, accurate, numerically robust model that can be efficiently updated to reflect future changing conditions as needed.

The first step in developing the model is to assemble a GIS database containing the relevant data, including: topography, soil type, land use, vegetation, hydrographic and other base map features. From the detailed topography, a series of drainage basins were developed. These basins were then verified through field observations and finalized through manual data entry. Next the soil type, land use, and vegetation layers were queried to determine a runoff coefficient for each of the defined basins, along with the average slope, flow path length and other critical information necessary for the hydrology model.

The information from the GIS is then compiled into the SewerGEMS model and the storm water runoff hydrograph for each basin is computed. Within the model, each basin was linked via stream channels or pipe segments to route the hydrographs through the system. To accurately model the natural stream channels "irregular cross sections" were selected as the channel type. Typical cross sections for the natural channels were entered manually from the detailed GIS data at key points in the system. The irrigation canals and other major ditches were modeled as "trapezoidal channels." Detention facilities were inserted and modeled as part of the system where detention basins were known to exist and along wide portions of the natural streams to simulate the natural stream attenuation processes.

Once the model for the existing system was completed and calibrated, the results were queried to determine the maximum depth and peak flow in each channel segment. Segments that appear as over capacity are flagged as potential problems. Various alternatives are then modeled to find potential solutions to any existing problems identified by the initial model.

Once the existing system is considered satisfactory, the GIS data is reprocessed to calculate new runoff coefficients (CN values) based on the future land use types. These future values are used to produce future basin hydrographs which are then routed through the system. Problem areas and high water lines are recorded. Necessary improvements are made within the model until the system components are operating at their respective LOS discussed in the previous chapter.

Chapter 5 EXISTING STORM DRAINAGE SYSTEM ANALYSIS

5.1 STORM WATER MODEL

The majority of the storm drain network for the existing model consisted of 155 natural stream channels, 200 drainage basins and 4 point inflows to represent irrigation and basin inflows. The storm water model is considered a "trunkline model" whereby the major storm water conveyance channels are modeled on a macro scale that does not require precise input of every minor collector, roadway and catch basin. This type of model is able to accurately determine major drainage issues and aid in planning purposes without incurring the cost associated with an overly detailed analysis. Major drainages that are flagged as potential problems can then be analyzed individually on a more detailed level. Irrigation canals were assumed to be full at the beginning of the rainfall event and therefore unable to convey storm water. Based on discussions with the major irrigation canal companies, a series of turnout gates are typically opened when heavy rains occur in an effort to minimize canal over topping. The locations of the major turnouts have been included in the model to simulate the full effects of the storm plus the flow from the irrigation canal diversions. The typical inflow from Ashley Creek during large storm events was also simulated to ensure the worst case flooding was evaluated.

5.1.1 SIMULATED CONDITIONS

This section describes the conditions which were simulated to approximate a 10-, 25-, 50-, 100- and 500-year flood event throughout the entire Valley using assumptions that are both realistic and conservative.

5.1.1.1 Drainage Basins

As noted in previous sections, the Valley has been divided into 200 individual drainage areas. In order to accurately estimate the timing and magnitude of storm water runoff, knowledge of the longest length of flow, slope, soil group, land use, and vegetation parameters for each basin are required. These parameters were determined through a series of advanced queries within the GIS database.

These data were then used to calculate the time to concentration (Tc) and curve number (CN) values. Tc is a measure of the length of time that is required for a rain drop that lands on the highest point within a drainage basin to reach the outlet. CN values effectively determine what percentage of the total rainfall will contribute to runoff, and what component will infiltrate into the soil. Higher values of CN indicate basins with higher runoff potential. The methods used to calculate these parameters are described below.

Time to Concentration Calculations

The time to concentration Tc is a measure of the time required for a water droplet that is released at the upper most point of a basin to travel to the outlet of the basin. The Tc for each basin determines the magnitude and outflow hydrograph for each basin. For this report, Tc is calculated as the sum of the following three components: overland flow, channel flow, and stream flow. The overland flow component of Tc (Tc1) is used for the first portion of the drainage where the water is flowing across open fields. Tc1 is calculated using the USBR modified Kirpich equation shown below.

$$Tc1 = 11.8 * L^2 / S^{0.385}$$

Where: L is the length of the longest flow path or the maximum allowed overland flow length
S is the average slope of the flow path
Tc1 is the time of flow in hours

The channel and stream flow components Tc2 and Tc3, respectively, are calculated using the Chezy channel flow equation.

$$Tc2 = L/15 * S^{0.5}$$

$$Tc3 = L/25 * S^{0.5}$$

Where: L is the length of the stream or channel flow component
Tc2 and Tc3 are time of flow in seconds

After modifying the units, the sum of the Tcs were calculated to determine the basin's Tc value. The maximum length of the overland flow and channel flow were determined as part of the calibration process. A maximum overland flow length of 1,500 ft and a maximum channel length of 74,000 feet were selected as parameters that resulted in the best calibrated model.

Curve Number Calculations

Curve numbers (CN) are empirically determined values that represent the fraction of rainfall that contributes to runoff. Higher CN values indicate greater runoff potential. The Soil Conservation Service has determined CN values for a wide variety of soil conditions. The CN values used in this report are shown in Table 5-1. Well vegetated areas were assumed over most of the Valley floor and poorly vegetated values were assumed on the slopes surrounding the Valley. Curve numbers were also increased where large portions of the basins were already developed.

Table 5-1 Existing Curve Numbers

Soil Type	Well vegetated	Poorly vegetated
A	68	39
B	61	79
C	74	86
D	80	89

The time to concentration calculations and the curve number ranges are presented in Table 5-2 below. Complete lists of the parameters for each basin are included in the appendix.

Table 5-2 Summary of Drainage Basin Input Parameters

	Area (acres)	Tc (hours)	CN
Min	102	2.8	65
Average	491	11.7	84
Max	1473	51.7	89
SD	264	12.7	5

5.1.1.2 Stream Channels

The stream channels in the model connect the basin outlet points to simulate storm water moving through the Valley. To accurately represent the flow width, depth, and velocity, an irregular cross-section for each segment was input. The model then used Manning's equation along with stream routing algorithms to calculate the flow rate at each segment over time. The model also determines the flow depth, width and other critical parameters used in determining stability and flooding concerns.

5.1.2 CALIBRATION

A key part of any complex storm water model is to verify that the simulated results match actual historic flows in the major stream channels. After the initial model simulations, the input parameters of time to concentration and Manning's n values are adjusted such that the simulated results better reflect the field data.

The basin contains a series of stream gauges on Ashley Creek and many of the tributary drainages. All of the stream gauge data were used in the calibration process, however, the irrigation channels often divert all or a large portion of the storm flows away from key drainages, thereby artificially decreasing the flow. In the model it is assumed that the irrigation channels are full prior to the rainfall event and therefore do not have capacity to carry storm water. The discrepancy between what has historically occurred throughout the basin and the model assumptions complicated the calibration process on the tributary stream level. The simulated flows generally exceed the recorded flows on the tributary streams; these results are expected given the conservative modeling assumptions. Table 5-3 indicates the USGS stations within the Valley along with peak flow and type of information available. The locations of the Stream Gauges are shown in Figure 5-1.

FIGURE 5-1
STREAM GAUGING STATION LOCATIONS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008

Legend

- USGS Stream Gauges
- DWQ Stream Gauges
- Major Road
- Major River or Canal
- Municipal Boundary

N E S W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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Table 5-3 Ashley Valley Stream Gauges

Location				Peak Flow		Record Length		
Site Number	Site Name	Lat	Long	Date	CFS	To	From	Years
9235600	POT CREEK ABOVE DIVERSIONS, NEAR VERNAL, UTAH	40°46'05"	109°19'06	May 10, 1973	286	1958	1993	35
9261500	BIG BRUSH CREEK (AB CAVE) NR VERNAL, UTAH	40°42'15	109°35'45"			1947	1955	8
9261700	BIG BRUSH CRK ABV RED FLEET RES, NR VERNAL, UT	40°35'20"	109°27'53"	May 22, 2005	423	1980	2006	26
9262000	BIG BRUSH CREEK NEAR VERNAL,UTAH	40°34'54"	109°26'03"	July 12, 1962	543	1940	1979	39
9262500	LT BRUSH CR BL E PK RES NR VERNAL UT	40°45'30"	109°32'00"			1950	1955	5
9263000	LITTLE BRUSH CREEK NR VERNAL, UT	40°42'58"	109°30'18"	May 30, 1950	608	1946	1952	6
9264000	ASHLEY C BELOW TROUT C NR VERNAL, UTAH	40°44'00"	109°40'40"	May 19, 1948	630	1944	1954	10
9264500	SOUTH FORK ASHLEY C NR VERNAL, UTAH	40°44'00"	109°42'10"	June 18, 1949	460	1944	1955	11
9265000	OAKS PARK CANAL NEAR VERNAL, UTAH	40°44'36"	109°37'18"			1946	1959	13
9265300	ASHLEY CREEK ABOVE RED PINE CREEK NR VERNAL, UT	40°40'47"	109°39'37"	June 10, 1965	7,400	1965	1975	10
9265500	ASHLEY CR ABV SP NR VERNAL UT	40°35'20"	109°37'20"			1941	1945	4
9266000	ASHLEY CR SPRING NR VERNAL UT	40°35'10"	109°37'20"			1943	1955	12
9266500	ASHLEY CREEK NEAR VERNAL, UT	40°34'39"	109°37'17"	June 15, 1995	4,100	1914	2006	92
9267100	ASHLEY CREEK ABOVE DRY FORK, NR VERNAL, UTAH	40°32'16"	109°36'33"	May 20, 1970	920	1969	1972	3
9271000	ASHLEY C, SIGN OF THE MAINE, NR VERNAL, UTAH	40°31'02"	109°35'45"	June 11, 1965	4,110	1939	1965	26
9271400	ASHLEY CREEK NEAR NAPLES, UT	40°26'01"	109°27'56"			2000	2003	3
9271450	ASHLEY CREEK BL SADLIER DRAW, NEAR NAPLES, UT	40°23'53"	109°25'44"			1999	2003	4

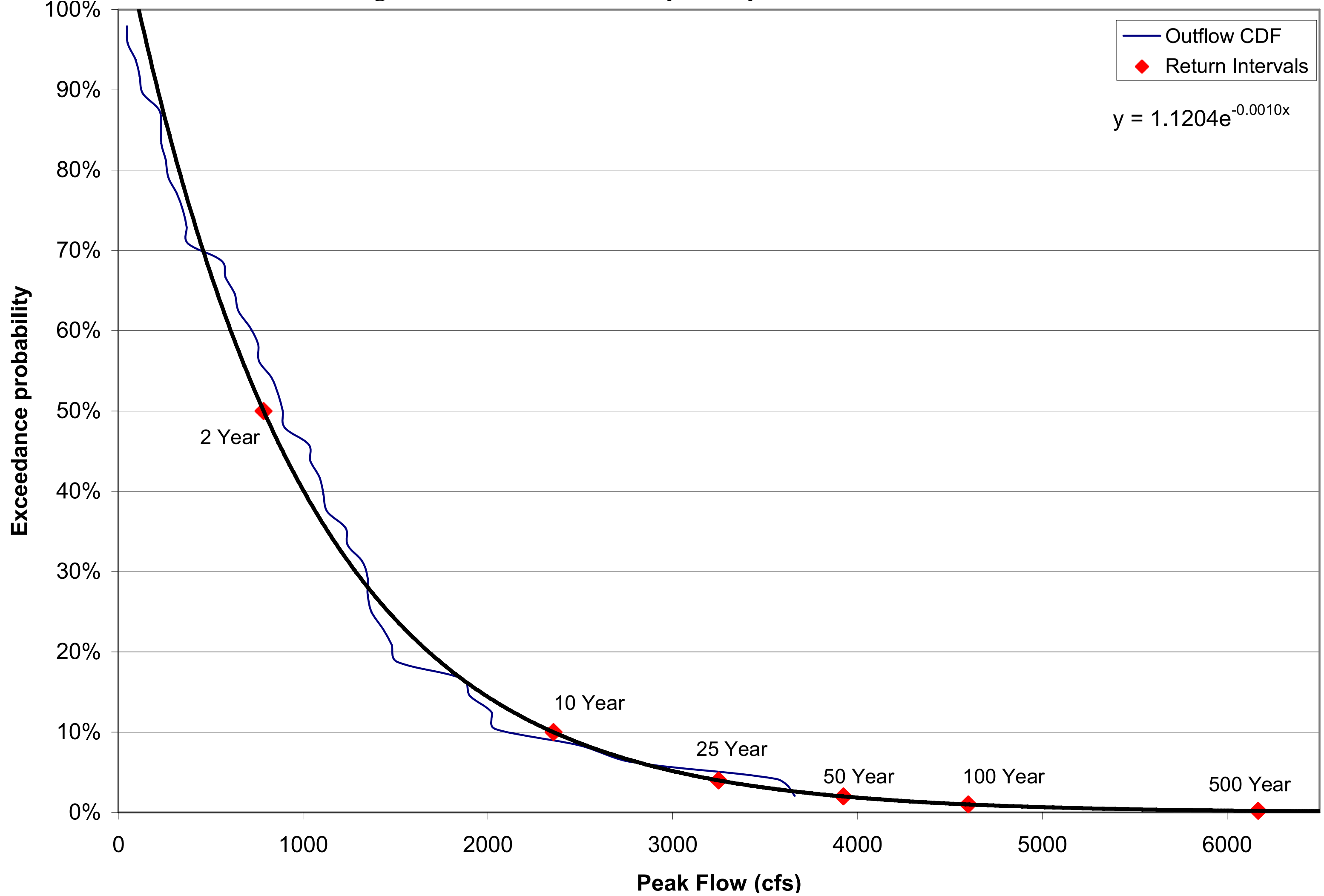
While the tributary stream gauging stations were able to provide only a qualitative calibration, a series of stream gauges along Ashley Creek were also analyzed to determine the total outflow from the basin. Evaluating the entire basin outflow provides a macro scale calibration of the model. The peak flow statistics from USGS stream gauge 9271500 (Ashley Creek near Jensen, Utah) is located below the study area and has recorded peak flows from approximately 1946 to 1983. This data was used to produce the cumulative distribution curve presented in Figure 5-2. The cumulative distribution curve is then used to statistically determine the 10-, 25-, 50-, 100- and 500-year historic flows. These flood flows are then compared to the simulated peak outflows and the model parameters adjusted through the calibration process. Statistically determined outflows as well as the calibrated model outflows are presented in Table 5-4.

Table 5-4 Outflow Calibration

		Stream Gauge Data	Valley Generated Flow (canals full)	Difference	% diff
Probability	Return Interval	CFS	CFS	CFS	
0.2%	500	6167	11,824	5,657	48%
1.0%	100	4599	7,697	3,098	40%
2.0%	50	3923	6,314	2,391	38%
4.0%	25	3248	4,924	1,676	34%
10.0%	10	2355	3,414	1,059	31%

The results of the peak flow analysis indicate that the model produces similar, but slightly elevated flows during the 10- and 25-year events. The elevated simulated peak flows are expected for two reasons. First, the model assumptions do not allow storm water to be routed through the irrigation canals. The irrigation canals increase the time to concentration and thereby artificially reduce the peak flows. Second, the tributary stream gauging data indicate that most storms affect only a portion of the Valley. The model simulates a basin-wide storm event. The basin-wide storm should produce elevated levels in Ashley Creek as all of the tributaries are contributing flow at the same time. Basin-wide storm events will result in less conformity to the statistical flows during large events, which is consistent with the calibration results presented in Table 5-4. By adjusting the Time to Concentration and Manning's n values, the model is adequately calibrated and appears to be producing conservatively realistic flows under the simulated conditions.

Figure 5-2 Estimated Ashley Valley Outflow Peak Values

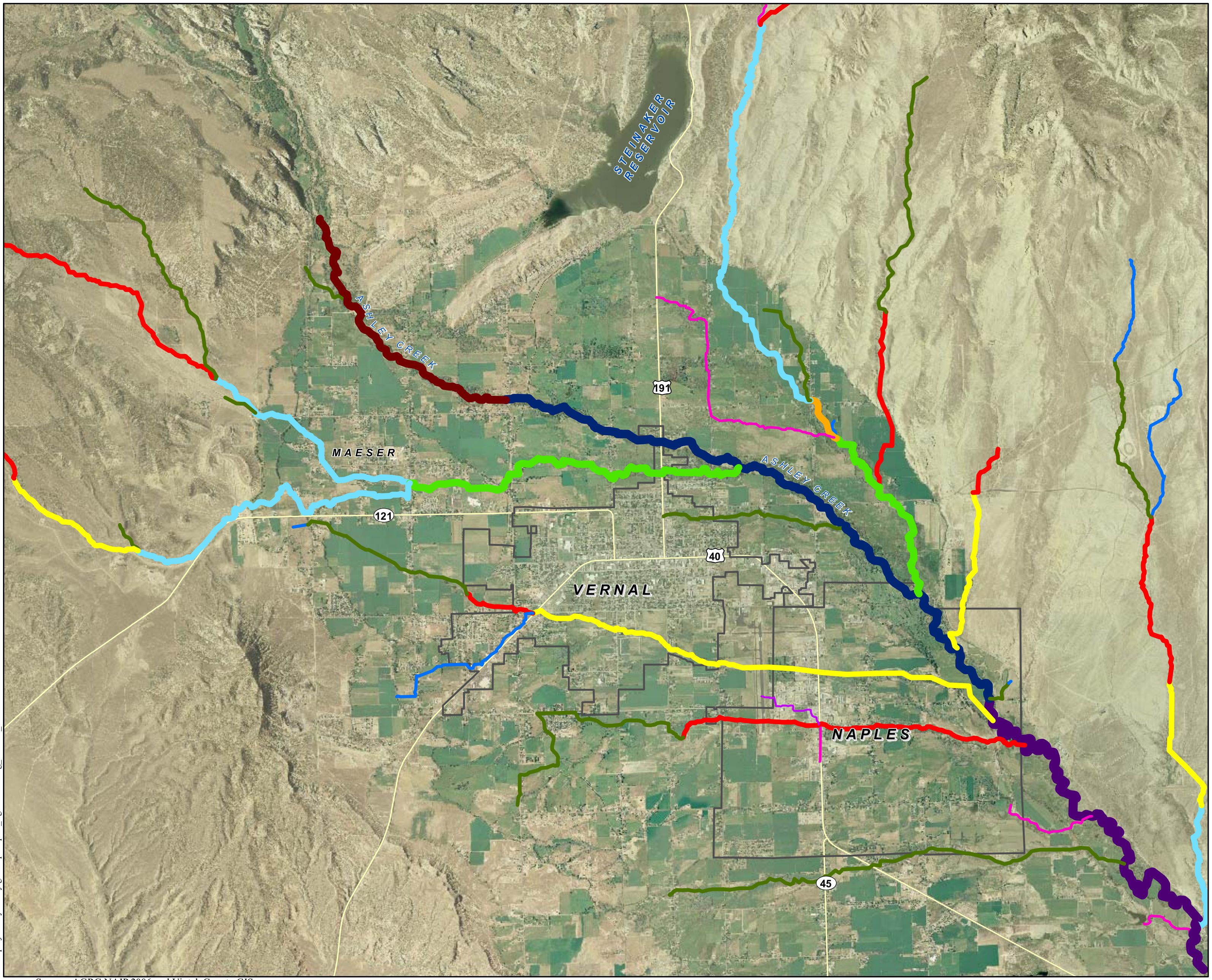


5.2 EXISTING MODEL FINDINGS

The following section presents the results of the modeling analysis described above. The peak flow rates from the 10-, 25-, 50-, 100- and 500-year flows are presented. Based on these calculated flows, the following parameters were identified: the capacity of the major culverts in the area, channel stability under flood conditions, and developed areas that may become inundated. The predicted flood flows throughout the Valley are shown in Figure 5-3, Figure 5-4, Figure 5-5, Figure 5-6, and Figure 5-7 for the 10-, 25-, 50-, 100- and 500-year storm events, respectively. In general, the modeling indicates that the majority of the Valley will be able to transport storm water flows that are likely to result from storms up to the 100-year event. The 500-year storm is modeled for comparison considerations. However, it is generally not economically viable to construct storm water protection above the 100-year event except for the most critical structures. The modeling also indicated a number of potential concerns where flooding is likely to occur. These potential concerns are discussed in detail in the following sections.

FIGURE 5-3
ESTIMATED EXISTING 10-YEAR CHANNEL FLOWS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008



Legend

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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50 EAST 100 SOUTH
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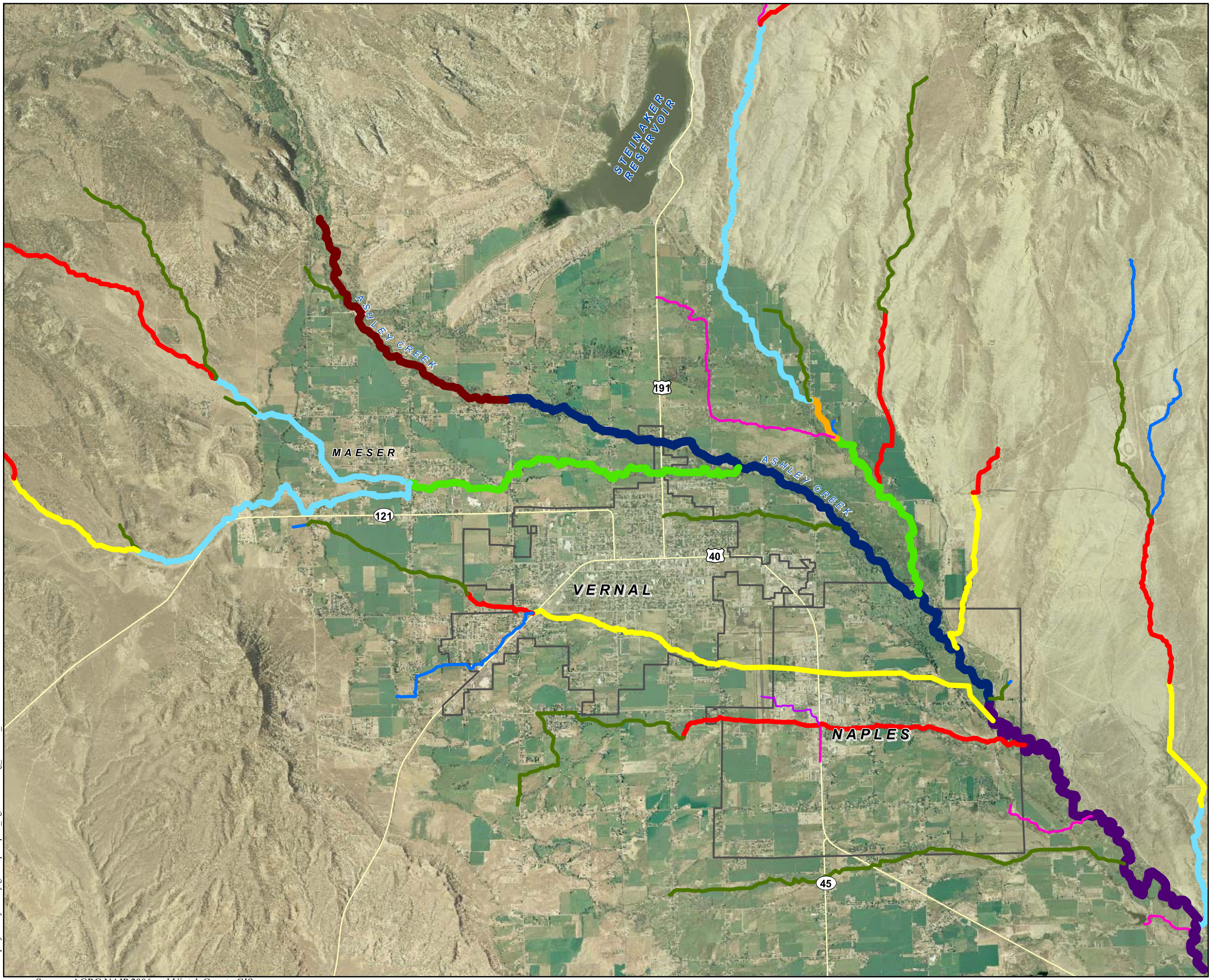
Source: AGRC NAIP 2006 and Uintah County GIS.

s:\proj\ashley-valley\gis\maps\report_figures\existing_channel_flows.mxd

FIGURE 5-4
ESTIMATED EXISTING 25-YEAR CHANNEL FLOWS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008



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Source: AGRC NAIP 2006 and Uintah County GIS.

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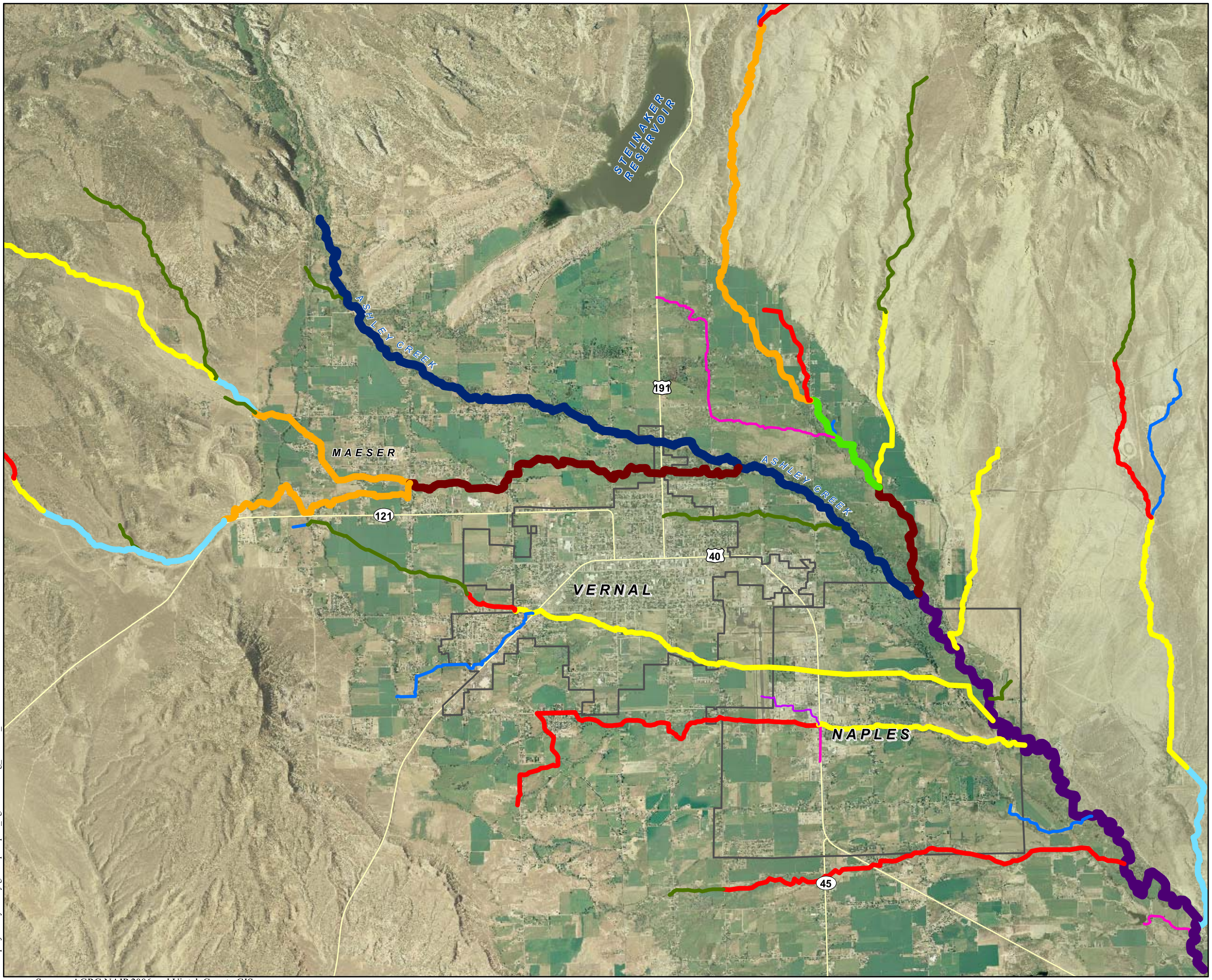

Source: AGRC NAIP 2006 and Uintah County GIS.

FIGURE 5-5
ESTIMATED EXISTING 50-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

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1 INCH = 5,000 FEET

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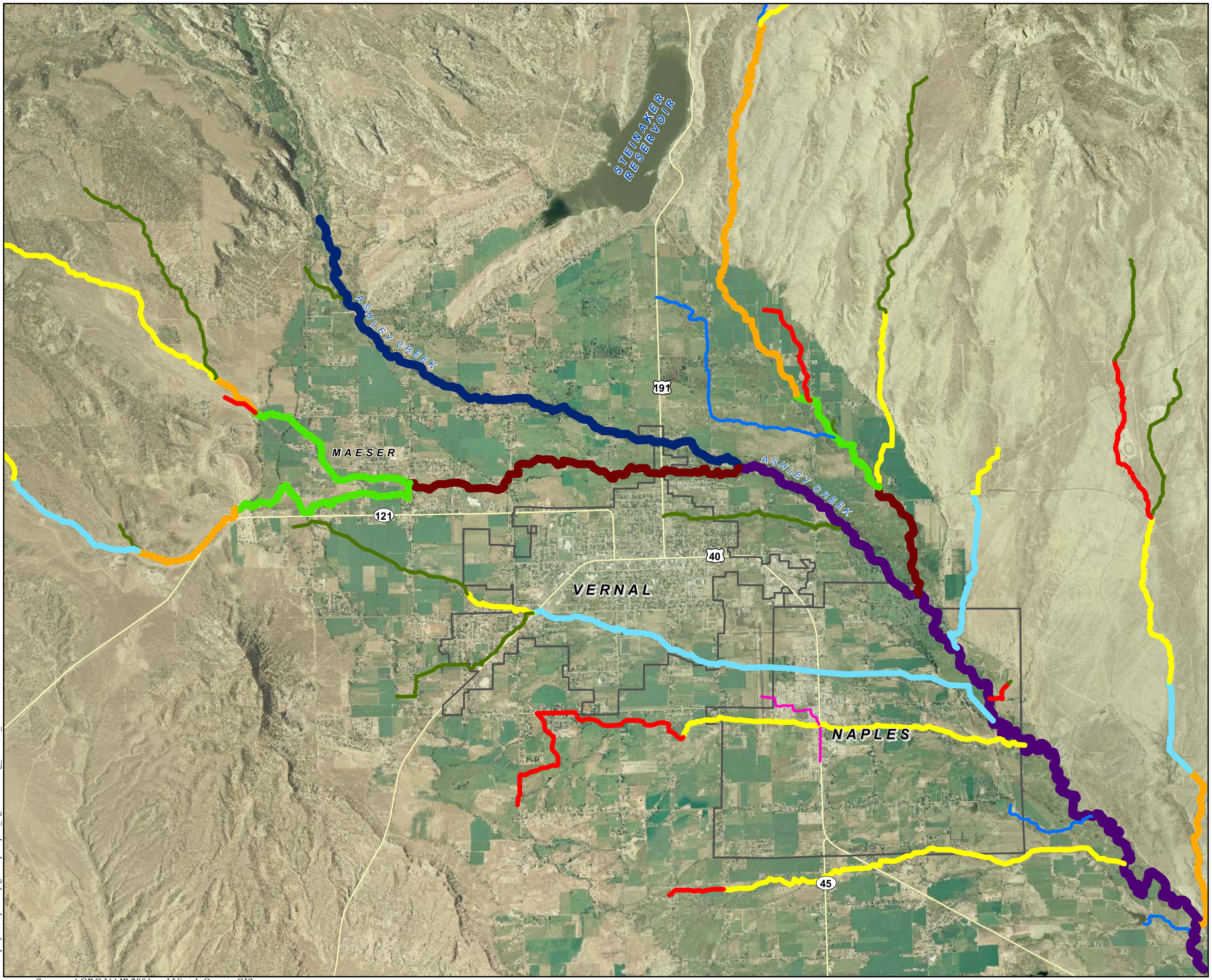

FIGURE 5-6
ESTIMATED EXISTING 100-YEAR CHANNEL FLOWS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

< 25

26 - 50

51 - 100

101 - 200

201 - 300

301 - 500

501 - 700

701 - 1,000

1,001 - 1,500

1,501 - 3,000

3,001 - 6,000

> 6,000

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

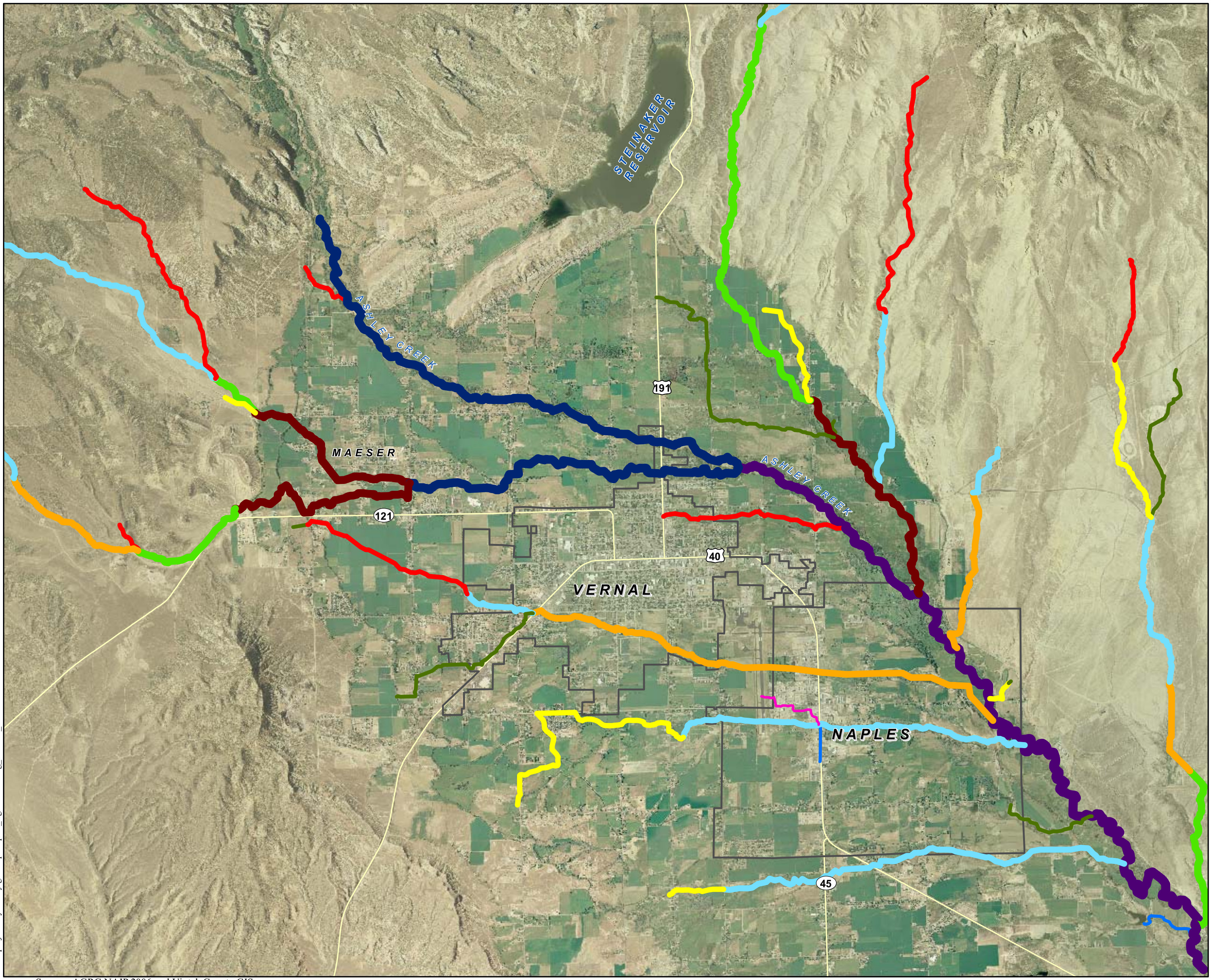

FIGURE 5-7
ESTIMATED EXISTING 500-YEAR CHANNEL FLOWS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

< 25

26 - 50

51 - 100

101 - 200

201 - 300

301 - 500

501 - 700

701 - 1,000

1,001 - 1,500

1,501 - 3,000

3,001 - 6,000

> 6,000

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1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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5.2.1 Major Culvert Analysis

In an effort evaluate the general condition of the culvert crossings throughout the Valley, seventeen (17) culverts were selected for analysis in areas where the modeling predicted relatively high flows and the culverts appeared to be relatively small. Of the 17 culverts that were analyzed, 14 culverts were determined to have insufficient capacity to convey the 10-year event, 15 culverts will become overwhelmed in a 25-year event, and a total of 16 culverts are insufficient to prevent flooding during a 50-year event. Based on discussions with Naples, Vernal City and Uintah County personnel, ***it is recommended that all culverts be designed to capacitate the 25-year event at a minimum and that culverts under critical roadways be designed for a minimum of the 50-year event.*** The results of the culvert analyses are presented in Table 5-5. Flows and approximate recommended sizing for the major culverts throughout the Valley are identified later in this report.

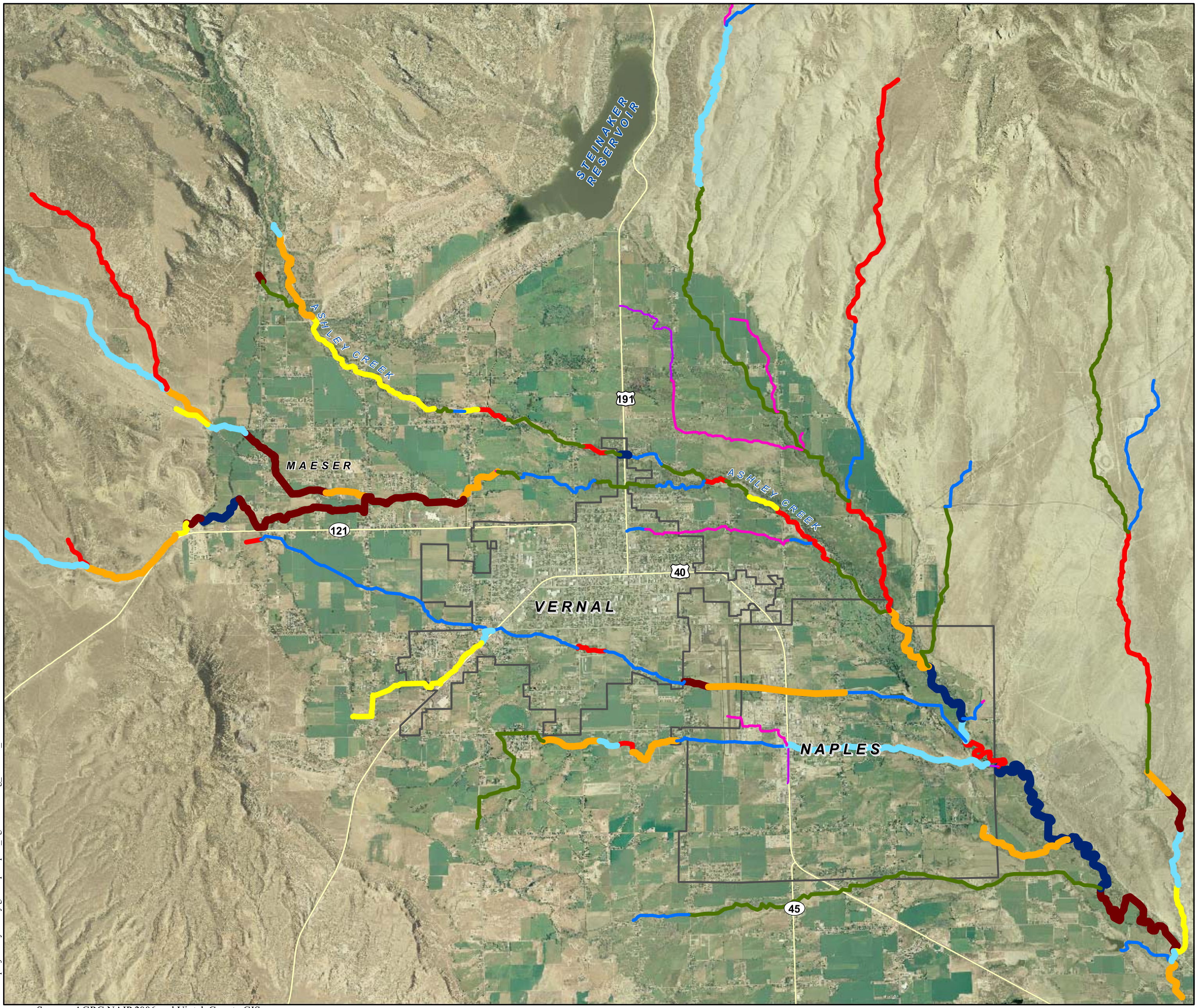
Table 5-5 Culvert Capacities

		Simulated Flows (cfs)		
Culvert	Current Capacity	10 year	25 year	50 year
A	584	500	704	893
B	549	14	22	29
C	16	67	100	128
D	41	67	100	128
E	27	162	248	326
F	45	162	248	326
G	133	162	248	326
H	31	167	256	335
I	26	31	49	66
J	38	80	120	157
K	26	80	120	157
L	30	89	138	185
M	540	341	503	658
N	160	341	503	658
O	44	68	80	92
P	38	68	80	92
Q	27	68	80	92

*Yellow cells denote simulated flows in excess of the culvert capacity
*Orange cells denote simulated flows in excess of 2x the culvert capacity

5.2.2 Channel Stability

As part of the modeling effort, the maximum stream velocity in each channel reach was determined for each storm event. Channels can become unstable when the water velocity reaches sufficient speed to cause large-scale bank erosion and destabilization of the channels. For the purposes of this report, peak flood velocities below 7 feet per second (fps) are not considered to be at risk of destabilization. Channels where the peak velocity is calculated to be in excess of 7 fps are more likely to become destabilized. Maximum stream velocities for the 10-, 25-, 50-, 100- and 500-year storm events are highlighted in Figure 5-8, Figure 5-9, Figure 5-10, Figure 5-11, and Figure 5-12, respectively.



Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 5-8
ESTIMATED EXISTING 10-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Velocity (ft/s)

< 1.0

1.1 - 2.0

2.1 - 3.0

3.1 - 4.0

4.1 - 5.0

5.1 - 6.0

6.1 - 7.0

7.1 - 12.0

12.1 - 20.0

> 20

N E S W

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1 INCH = 5,000 FEET

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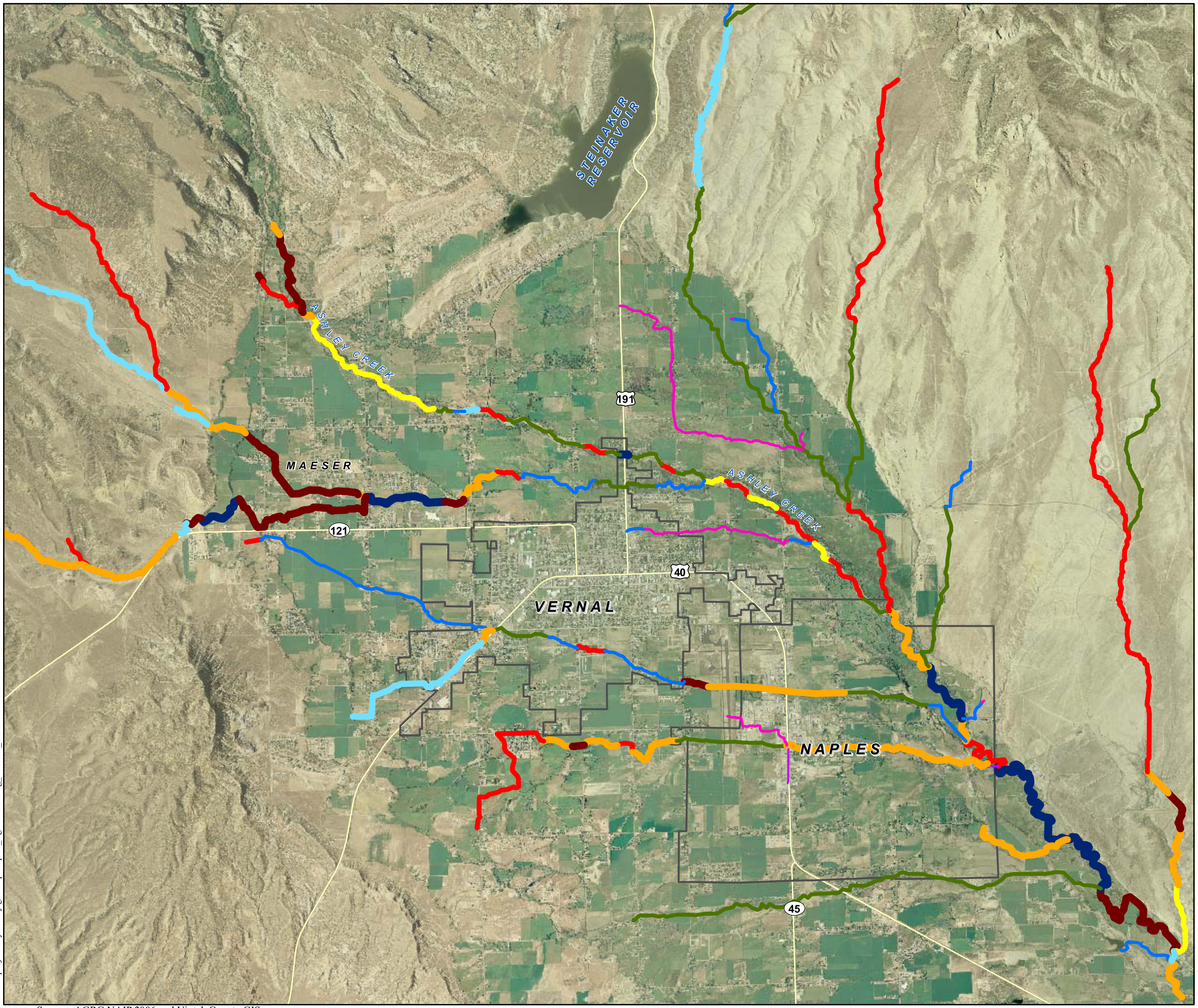

Source: AGRC NAIP 2006 and Uintah County GIS.

FIGURE 5-9
ESTIMATED EXISTING 25-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

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1 INCH = 5,000 FEET

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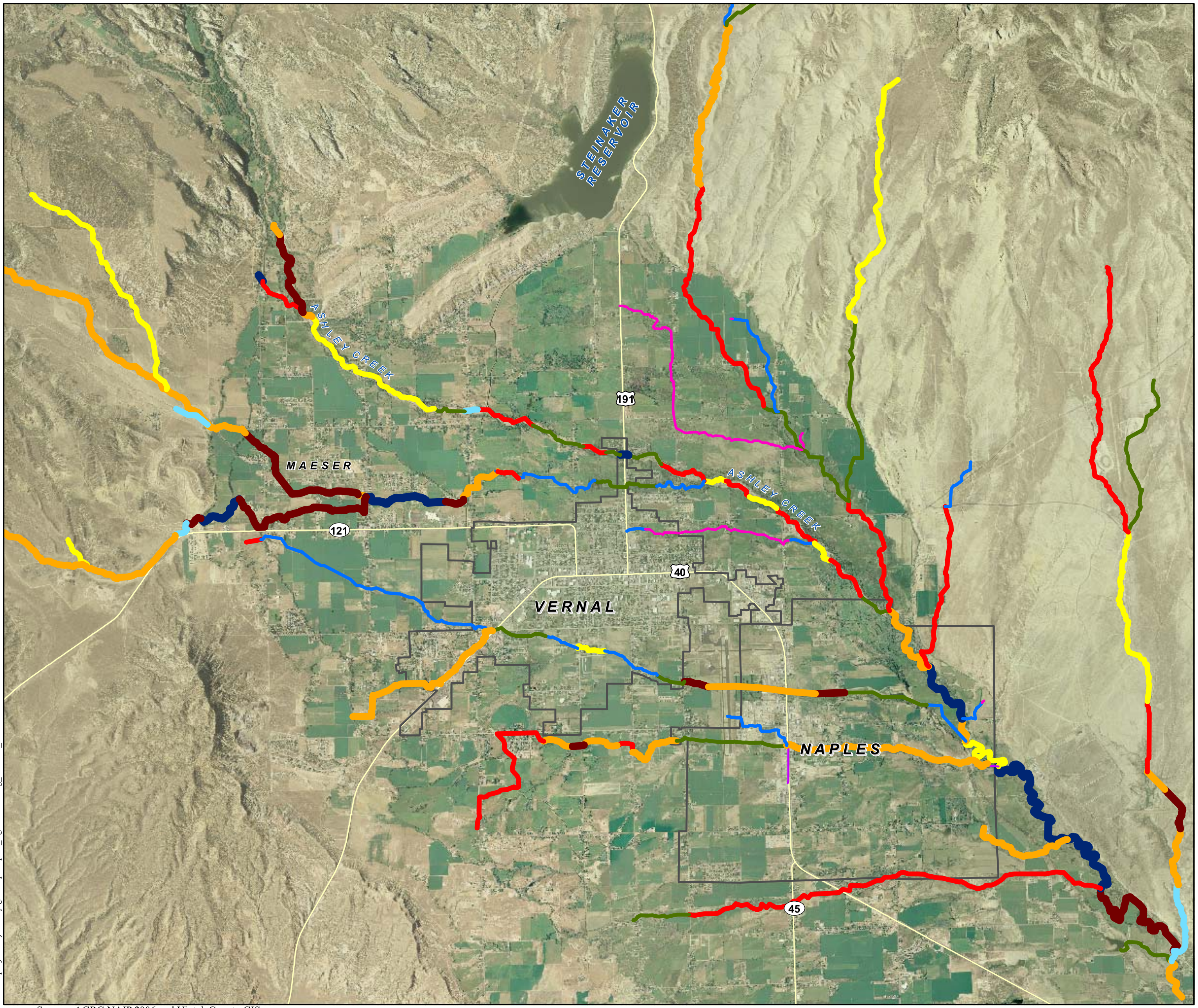

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 5-10
ESTIMATED EXISTING 50-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N E S W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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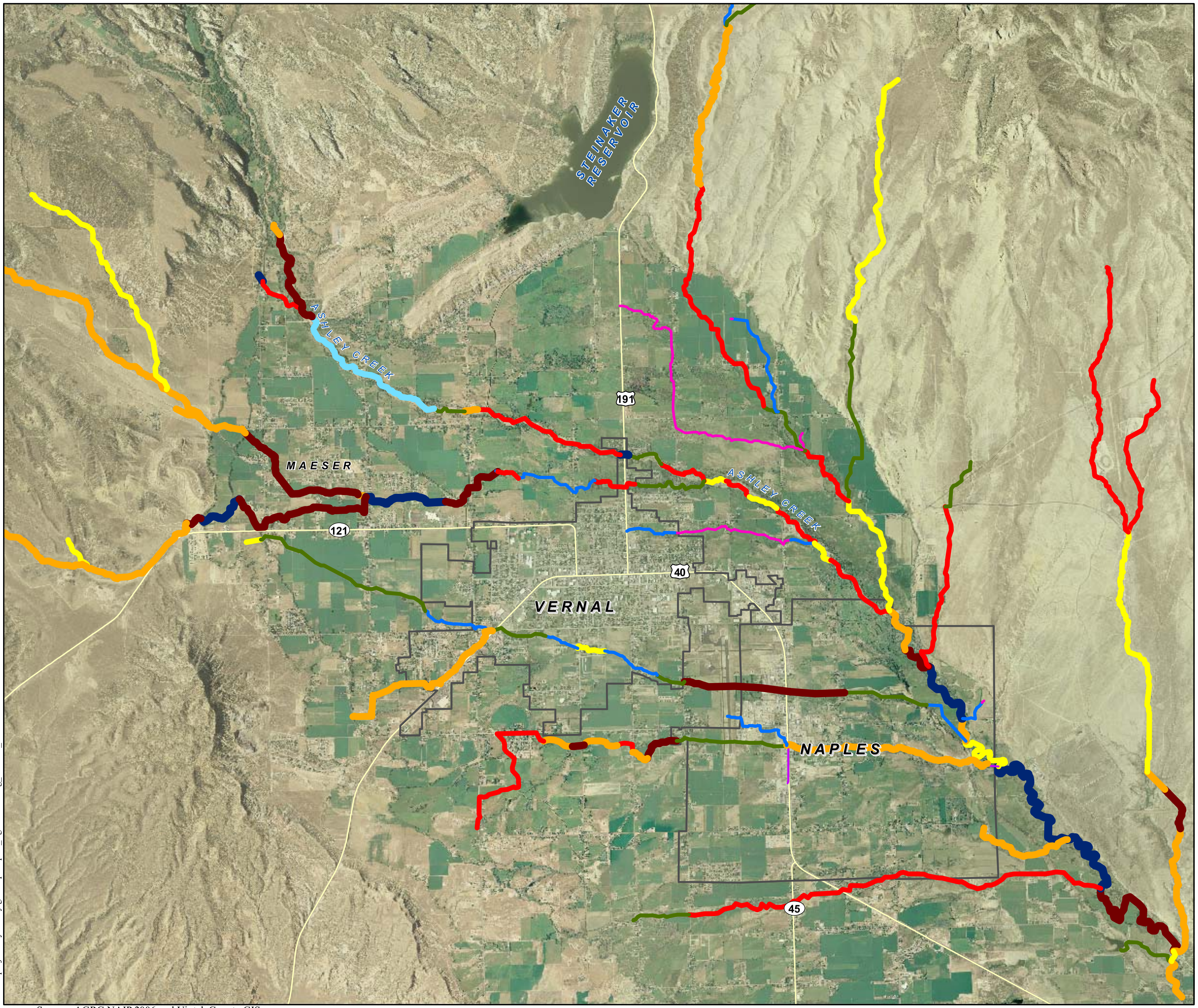
FIGURE 5-11
ESTIMATED EXISTING 100-YEAR CHANNEL VELOCITIES ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

Legend

Major Road

Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

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1 INCH = 5,000 FEET

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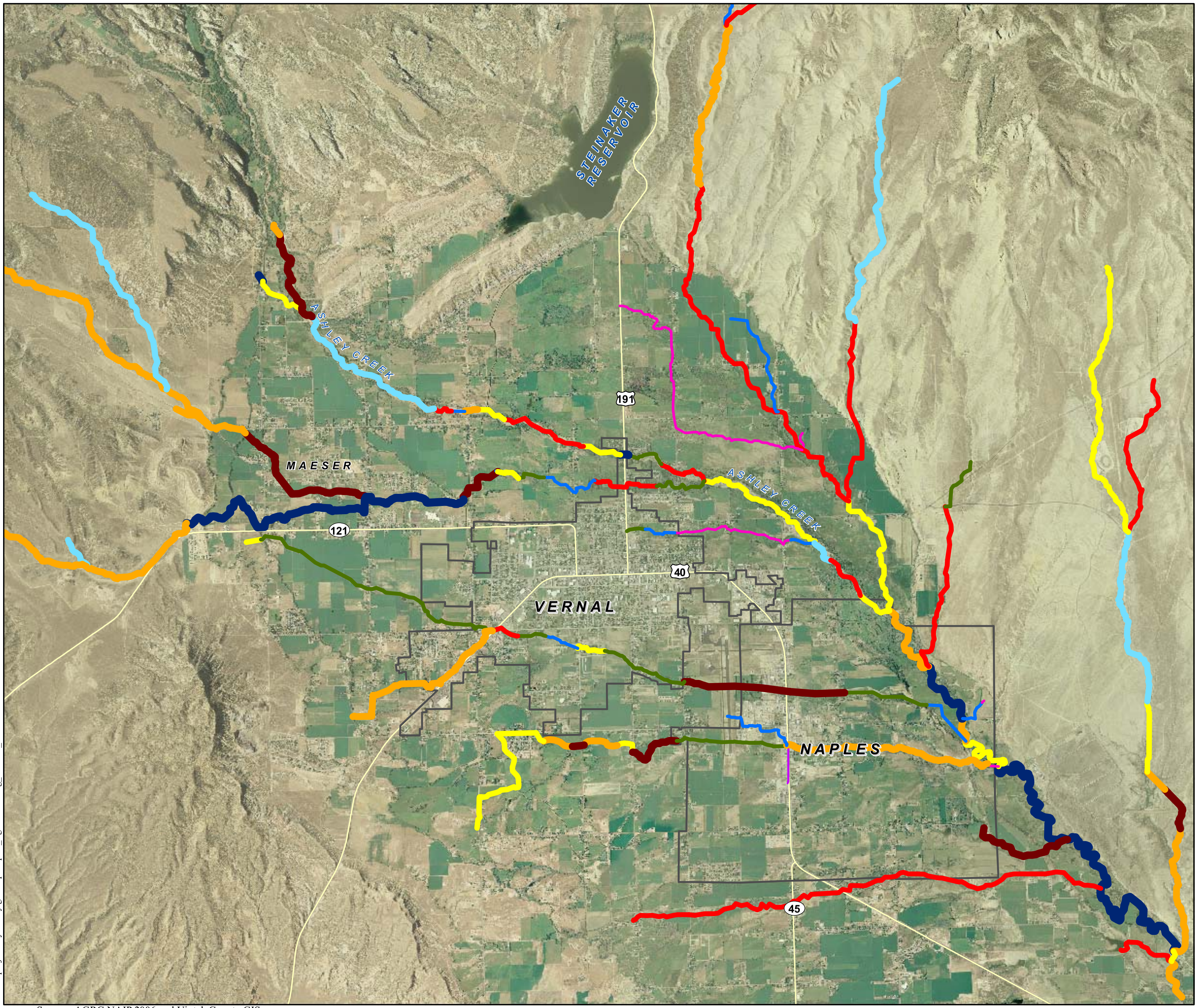

FIGURE 5-12
ESTIMATED EXISTING 500-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Velocity (ft/s)

< 1.0

1.1 - 2.0

2.1 - 3.0

3.1 - 4.0

4.1 - 5.0

5.1 - 6.0

6.1 - 7.0

7.1 - 12.0

12.1 - 20.0

> 20

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1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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5.2.3 INUNDATED ZONES

The Federal Emergency Management Administration (FEMA) generally requires that all major structures be constructed above the 100-year flood elevation. The majority of structures throughout the basin are above this minimum flood elevation and should not be inundated by flood waters under normal circumstances. Figure 5-13 highlights the zones throughout the basin that will likely become inundated during the 100-year event, based on the modeling results presented in this report. Existing structures within these zones should be closely evaluated and the construction of future structures limited or disallowed. Some of the areas of highest concern include the areas immediately north and south of Vernal City and through Naples. These areas are of high concern at this time because growth from the cities is rapidly encroaching upon these flood plains. At the time of this report FEMA is in the process up updating the current flood plain maps for Uintah County. ***When the final revisions are complete it is recommended that Figure 5-13 in this report be replaced with the basin wide FEMA map and the flooding recommendations updated accordingly.***

FIGURE 5-13
POTENTIAL 100-YEAR FLOOD ZONES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

100-Year Floodplain

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

Source: AGRC NAIP 2006 and Uintah County GIS.

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Chapter 6 FUTURE STORM DRAIN CAPACITY ANALYSIS

6.1 FUTURE STORM WATER MODEL

As the Valley continues to develop, the network of systems used to control and direct storm water runoff safely through the Valley will become increasingly important. The developed lands will have a higher runoff potential. New development may also encroach on the historic flood plains reducing the Valleys capacity to efficiently transmit storm water through the Valley. The combination of higher runoff and smaller channels to carry the flow has the potential to create numerous and expensive flooding problems throughout the Valley. The purpose of this chapter is to identify the potential problem areas that will most likely result from additional development before they occur. Through proactive thinking and proper planning, the majority of future potential flooding can be prevented.

6.1.1 FUTURE DRAINAGE BASIN CONFIGURATION

The existing basin configuration is utilized for the future modeling. Using the same basin configuration requires that the future development will not affect the macro scale drainage basins throughout the Valley. Given the size of the delineated basins and minimal influence the existing basins have had on the natural flow, this is a reasonable assumption to make at this time.

While the basin configuration remains the same between the existing and future models, the CN values for each basin were recalculated to reflect the anticipated developed land use shown in Figure 3-3. The curve number assigned to each basin was calculated as an area weighted average of the soil types and zoning within each basin. The curve numbers assigned to each soil type and land use pair are shown in Table 6-1. For basins where development is not anticipated (i.e. the hill sides surrounding the Valley), the historic CN values were retained in the future analysis.

Table 6-1 Future CN values

	Soil Group			
Land Use	A	B	C	D
Commercial /industrial/ governmental	89	92	94	95
Developed Open Spaces / parks	49	69	79	84
Residential <1/8 acre lots	77	85	90	92
Residential 1/3 acre lots	57	72	81	86
Residential 1/2 acre lots	54	70	80	85
Residential >1 acre lots	51	68	79	84

The time to concentration calculations for the future modeling were also re-evaluated. Time to concentration values are typically much shorter in developed areas than in undeveloped areas. However, the Valley currently requires storm water mitigation through retention or detention basins. This future simulation assumes that the existing basins combined with similar requirements for all future development will generally prevent the macro scale Tc values from decreasing. When detention basins are sized and constructed properly, they function to keep the future peak flow at or below the historical flows. Retention basins capture a large portion of the storm event and then overflow beyond their capacity. To account for these basins throughout the future developed areas the Tc values were adjusted (increased) such that the peak storm event for the 100-year storm were not increased by more then 20%. This assumption provides conservative, yet realistic, flow predictions for the larger events where some basins may fail, others will prematurely overtop and others will function correctly. The Tc values that were used to model future conditions are included in the appendix.

6.2 Analysis of the Existing System Under Future Conditions

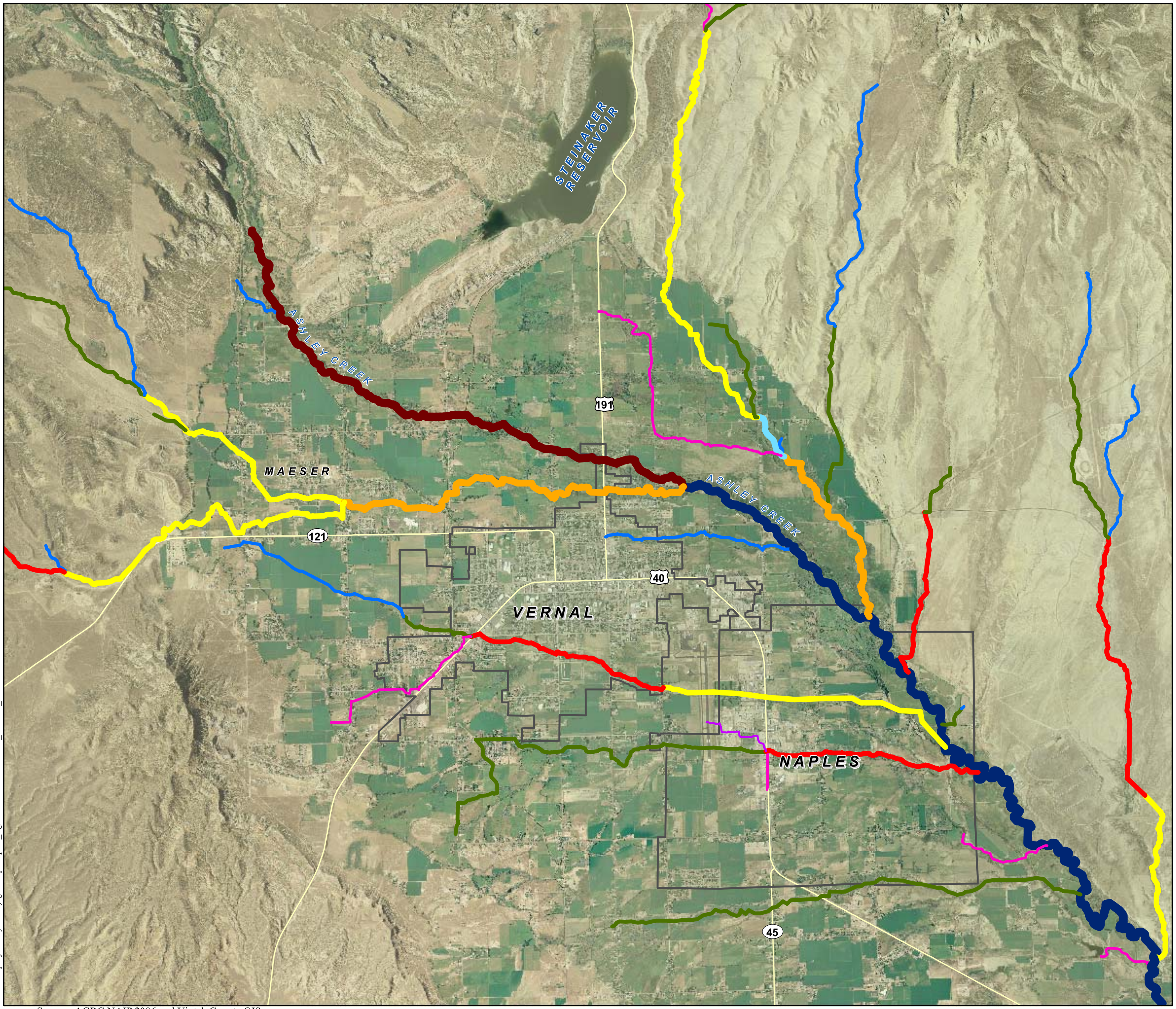
The following sections present the results of the future modeling effort, as well as highlight the areas of future concern as the Valley develops. The following chapter presents recommended modifications to zoning, ordinances, and resolutions, as well as capital improvement projects that will protect the Valley from flooding as development continues.

6.2.1 Future Predicted Flows

This section presents the anticipated future flows for the 10-, 25-, 50-, 100-, and 500-year storm events. The predicted future flows throughout the Valley are presented in Figure 6-1, Figure 6-2, Figure 6-3, Figure 6-4, and Figure 6-5 and discussed below.

FIGURE 6-1
ESTIMATED FUTURE UNMODIFIED 10-YEAR CHANNEL FLOWS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008



Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N E S W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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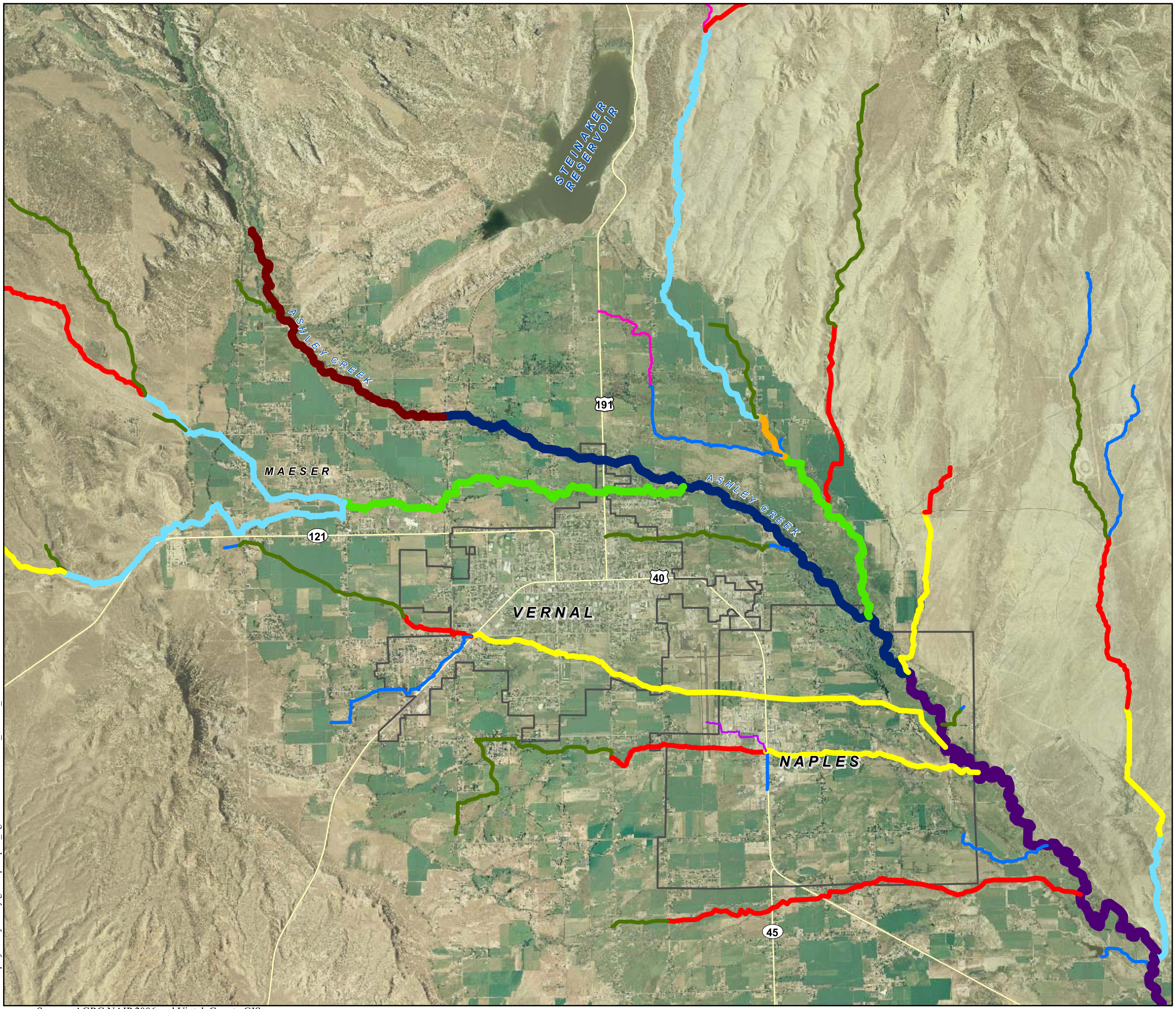
FIGURE 6-2
ESTIMATED FUTURE UNMODIFIED 25-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N W E S

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1 INCH = 5,000 FEET

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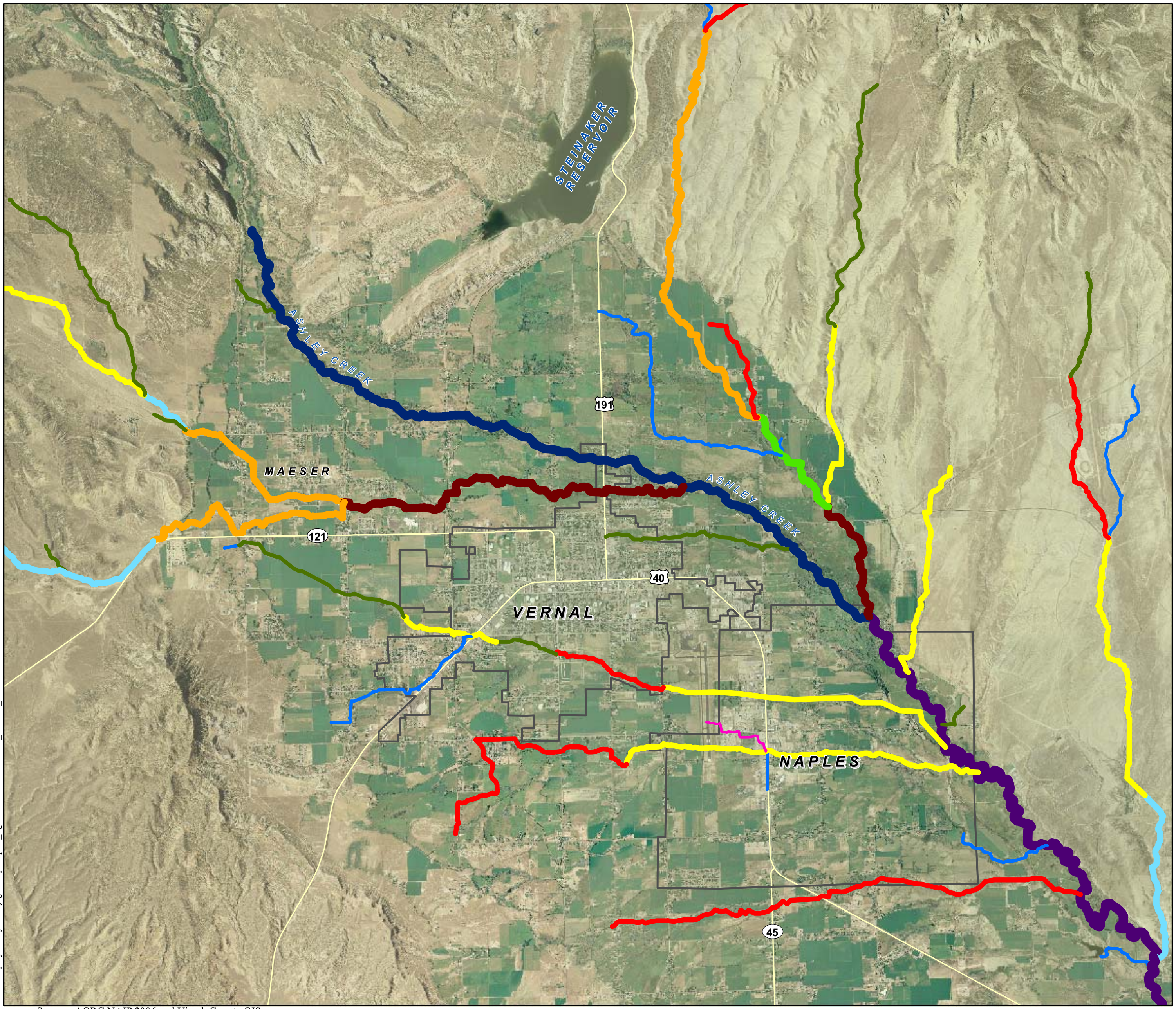

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 6-3
ESTIMATED FUTURE UNMODIFIED 50-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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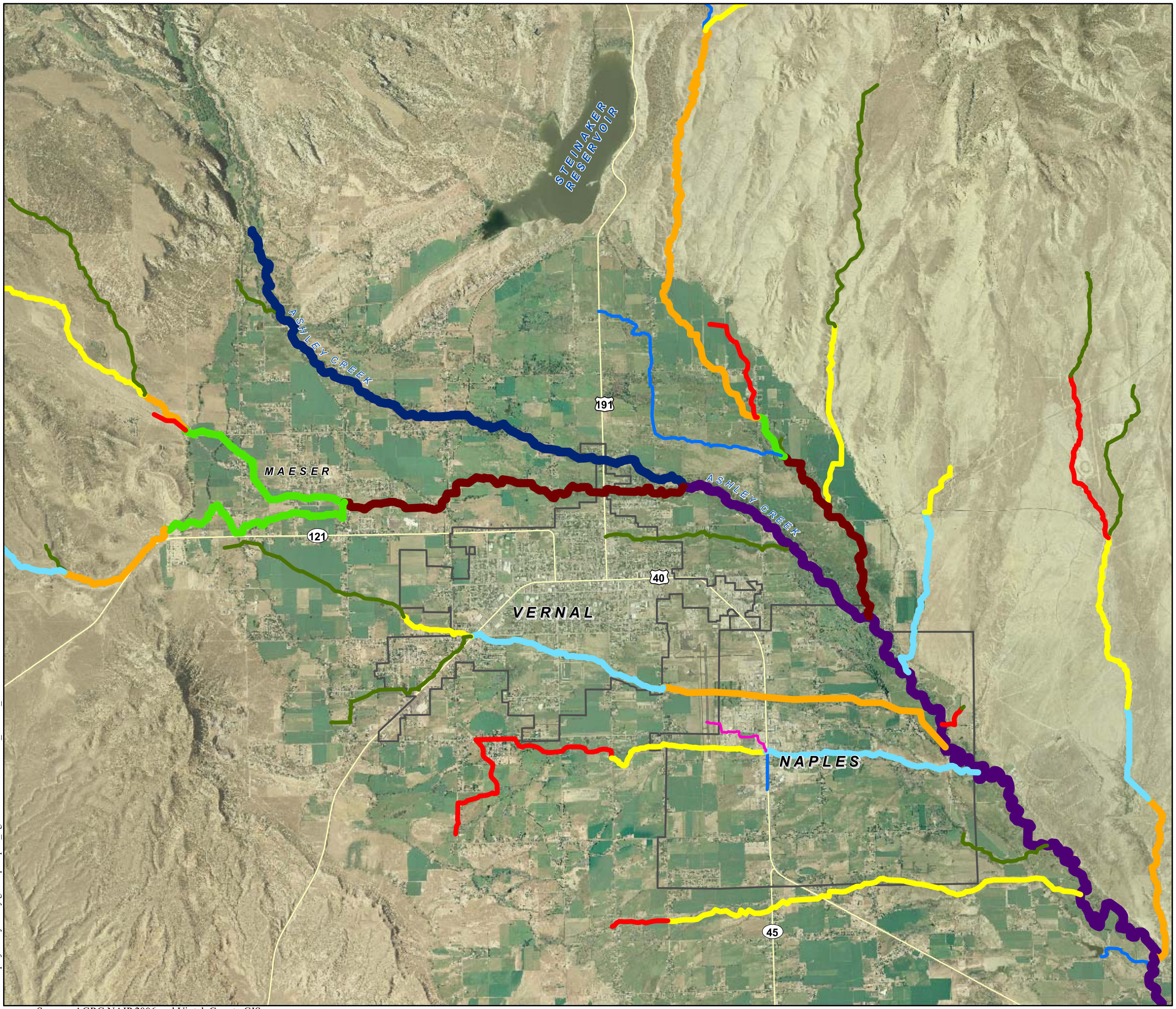

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 6-4
ESTIMATED FUTURE UNMODIFIED 100-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

< 25

26 - 50

51 - 100

101 - 200

201 - 300

301 - 500

501 - 700

701 - 1,000

1,001 - 1,500

1,501 - 3,000

3,001 - 6,000

> 6,000

N
W E
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0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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HEBER CITY, UTAH 84032
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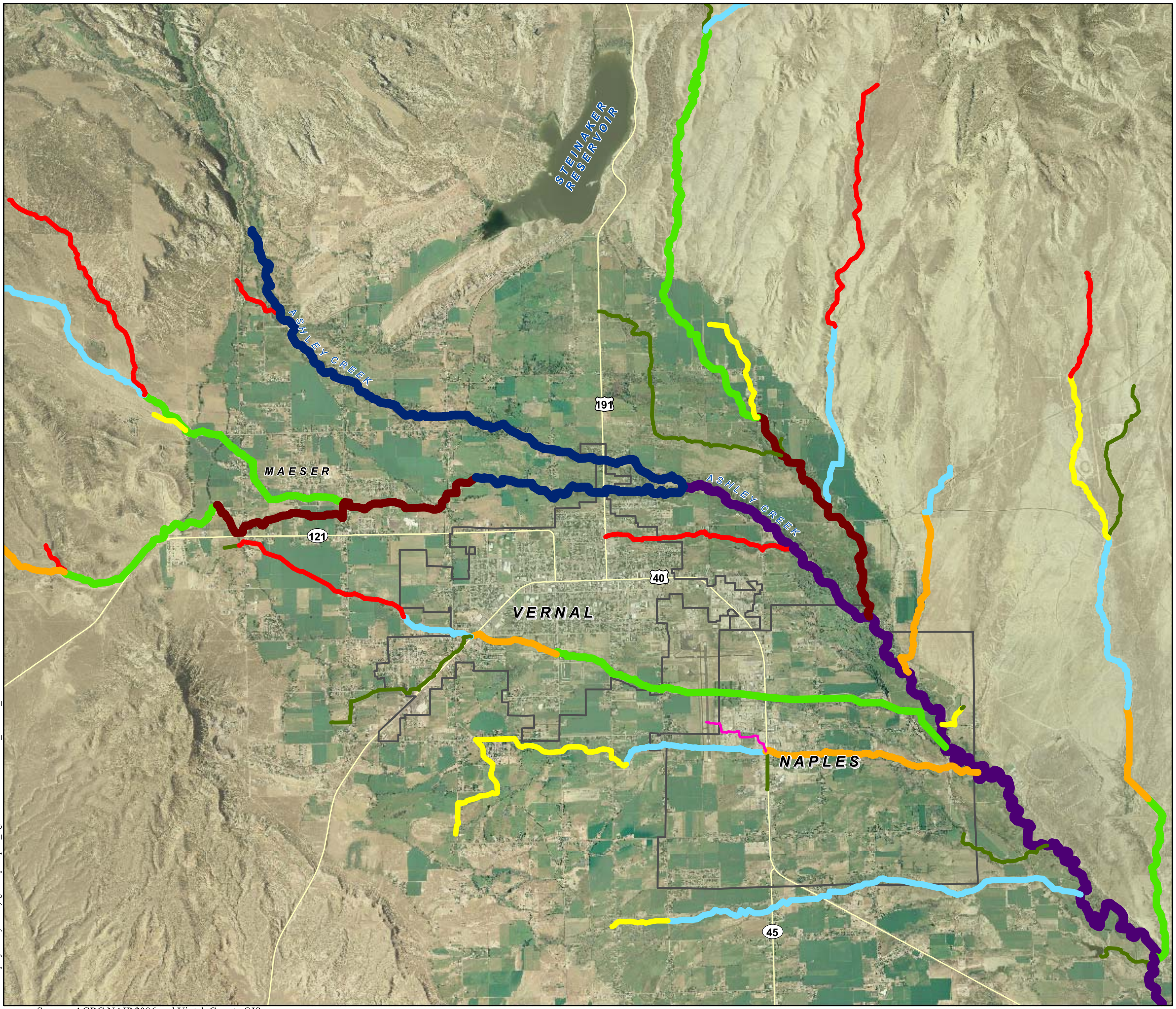

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 6-5
ESTIMATED FUTURE UNMODIFIED 500-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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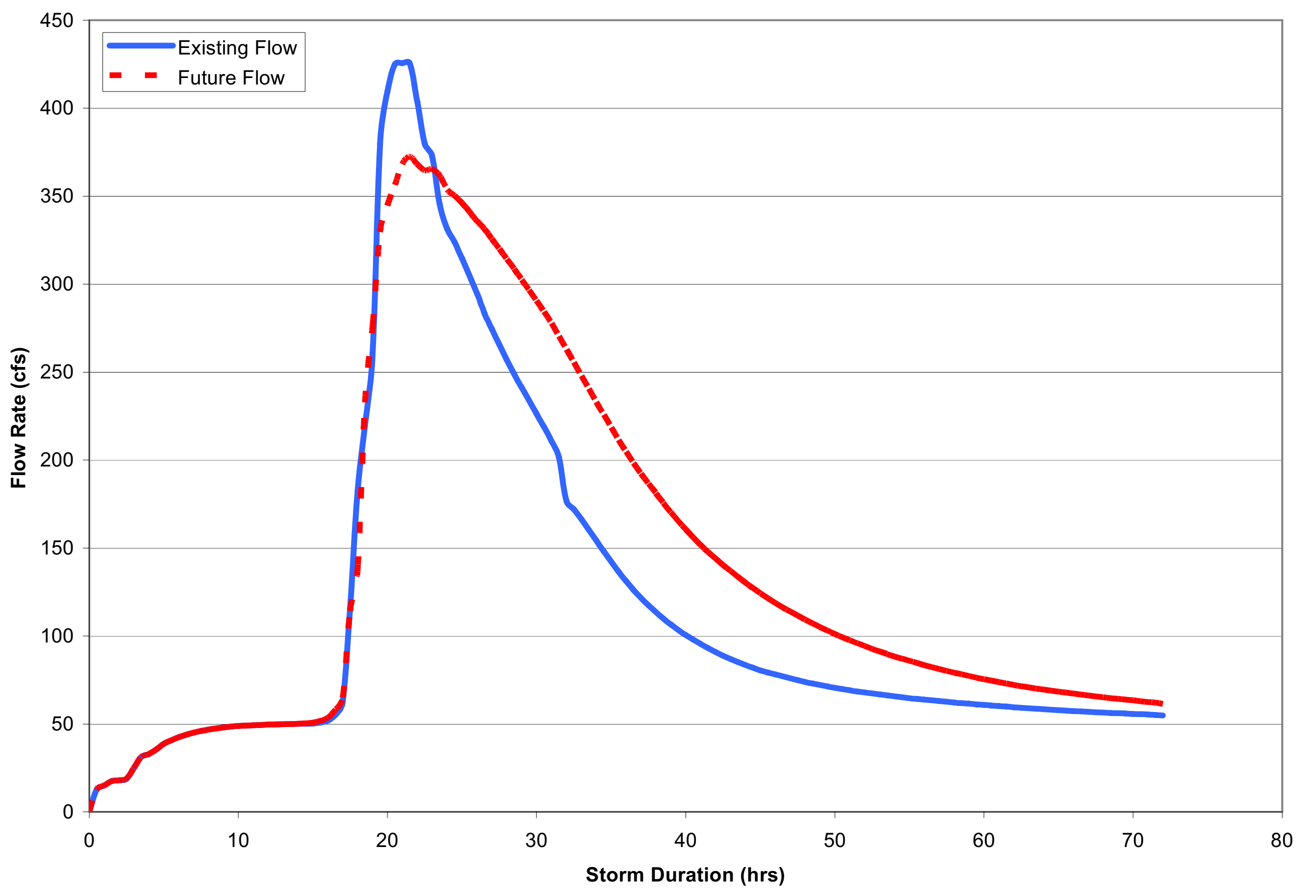
Generally, the peak flows entering the Valley from the mountains are equal in magnitude and duration, as expected with limited to no development in those areas. Similarly, flows immediately downstream from existing developments are comparable as the conditions are not greatly altered. The major changes between the simulations occur immediately downstream of areas that are anticipated to develop. Future peak flows may be slightly higher (see Tc assumptions above). More importantly, the storm hydrographs from the future developed areas are longer and the total volume of water to be conveyed is greater. The increased volume of water, even with a lower peak flow, may result in additional flooding, and potentially more stream channel erosion. The modified hydrographs must also be carefully considered when designing regional detention areas as a larger volume will be required to achieve the same reduction in flow. Figure 6-6 demonstrates the existing and predicted flows at a location East of Naples. In the figure the peak flow is actually decreased slightly as a result of local detention retention basins, however, the duration of the flow is increased by 20% to 30%.

Figure 6-6 Example Hydrograph

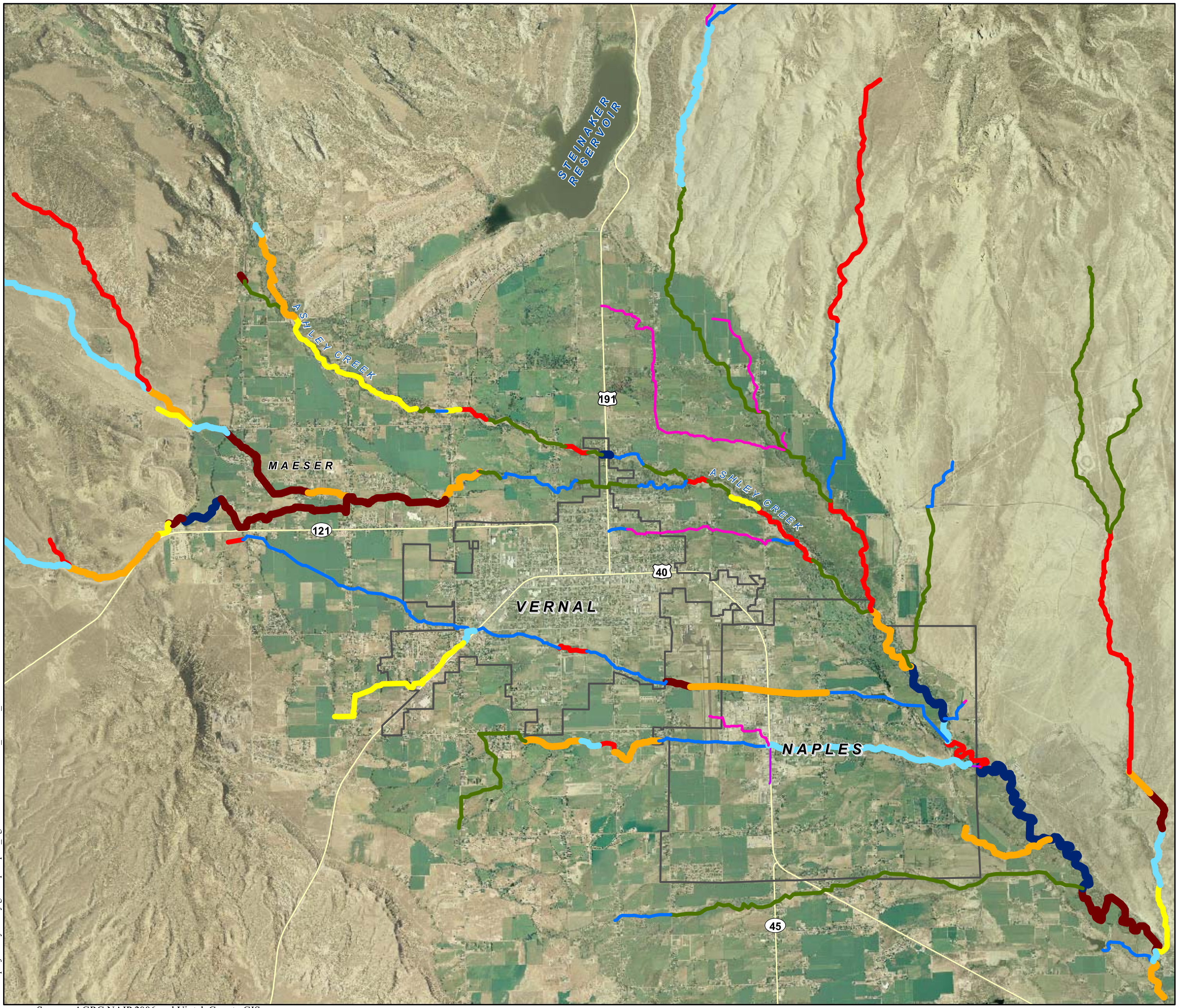


6.2.2 FUTURE PREDICTED VELOCITIES

The maximum stream velocity in each channel reach was determined for each storm event under the future conditions using a similar process described in Chapter 5. Channels can become unstable when the water velocity reaches sufficient speed to cause large-scale bank erosion and destabilization of the channels. For the purposes of this report, peak flood velocities below 7 feet per second (fps) are not considered to be at risk of destabilization. Channels where the peak velocity is calculated to be in excess of 7 fps are more likely to become destabilized. Maximum stream velocities for the 10-, 25-, 50-, 100- and 500-year storm events under the future conditions are highlighted in Figure 6-7, Figure 6-8, Figure 6-9, Figure 6-10, and Figure 6-11, respectively.

FIGURE 6-7
ESTIMATED FUTURE UNMODIFIED 10-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

Source: AGRC NAIP 2006 and Uintah County GIS.

s:\proj\ashley-valley\gis\maps\report_figures\futureunmodified_channel_velocities.mxd

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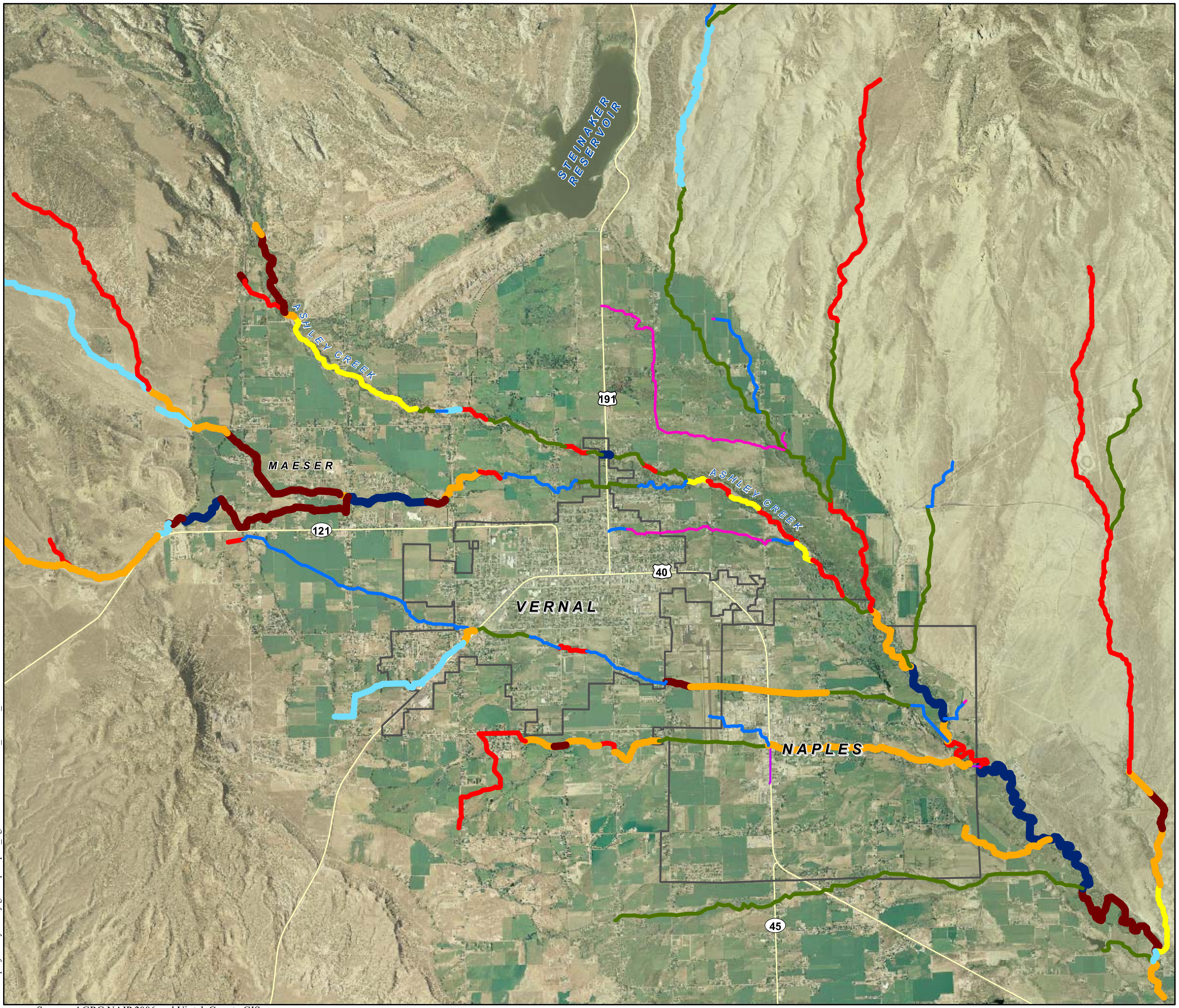
FIGURE 6-8
ESTIMATED FUTURE UNMODIFIED 25-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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FIGURE 6-9
ESTIMATED FUTURE UNMODIFIED 50-YEAR CHANNEL VELOCITIES ASHLEY VALLEY UINTAH COUNTY, UTAH

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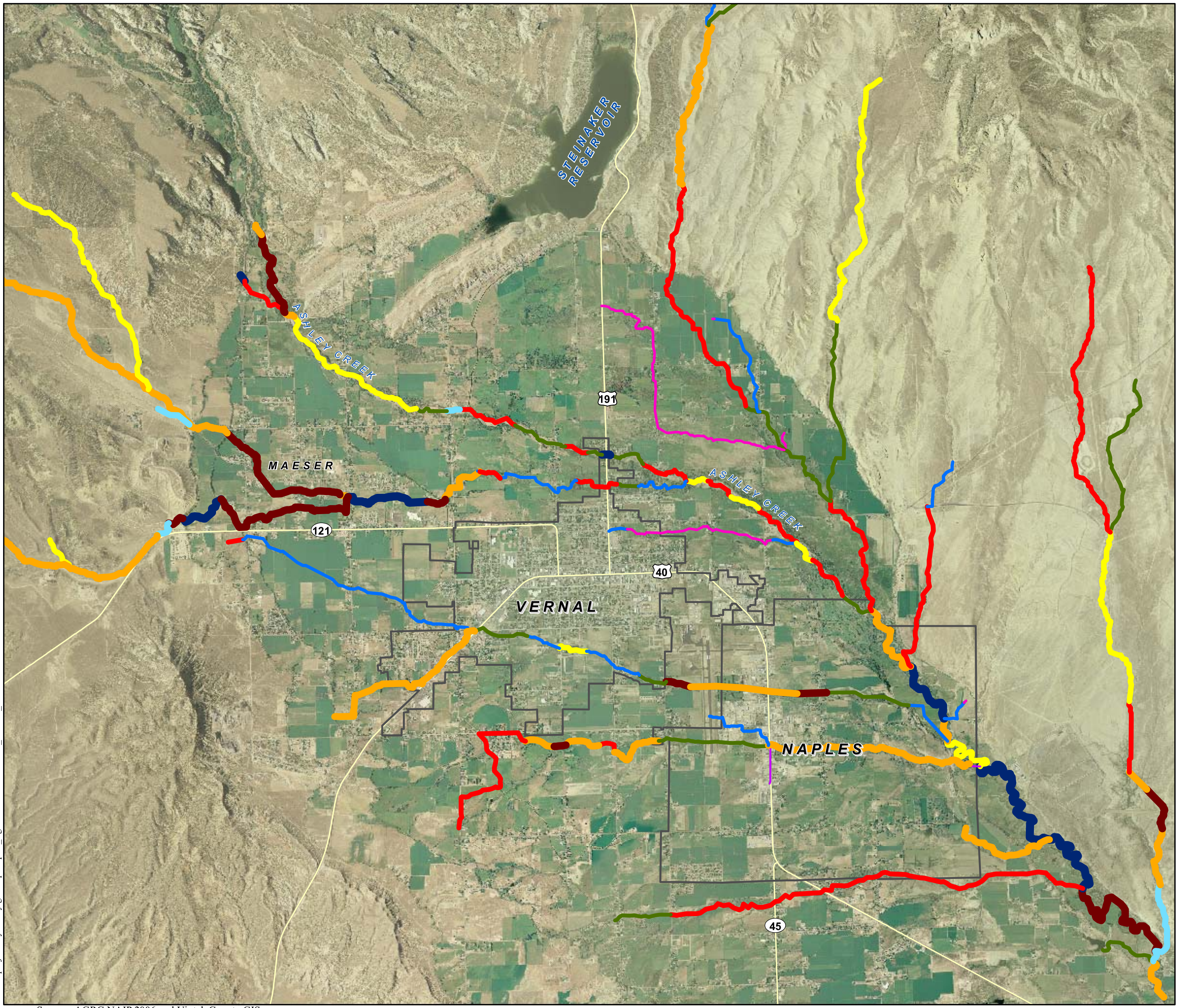

Legend

Major Road

Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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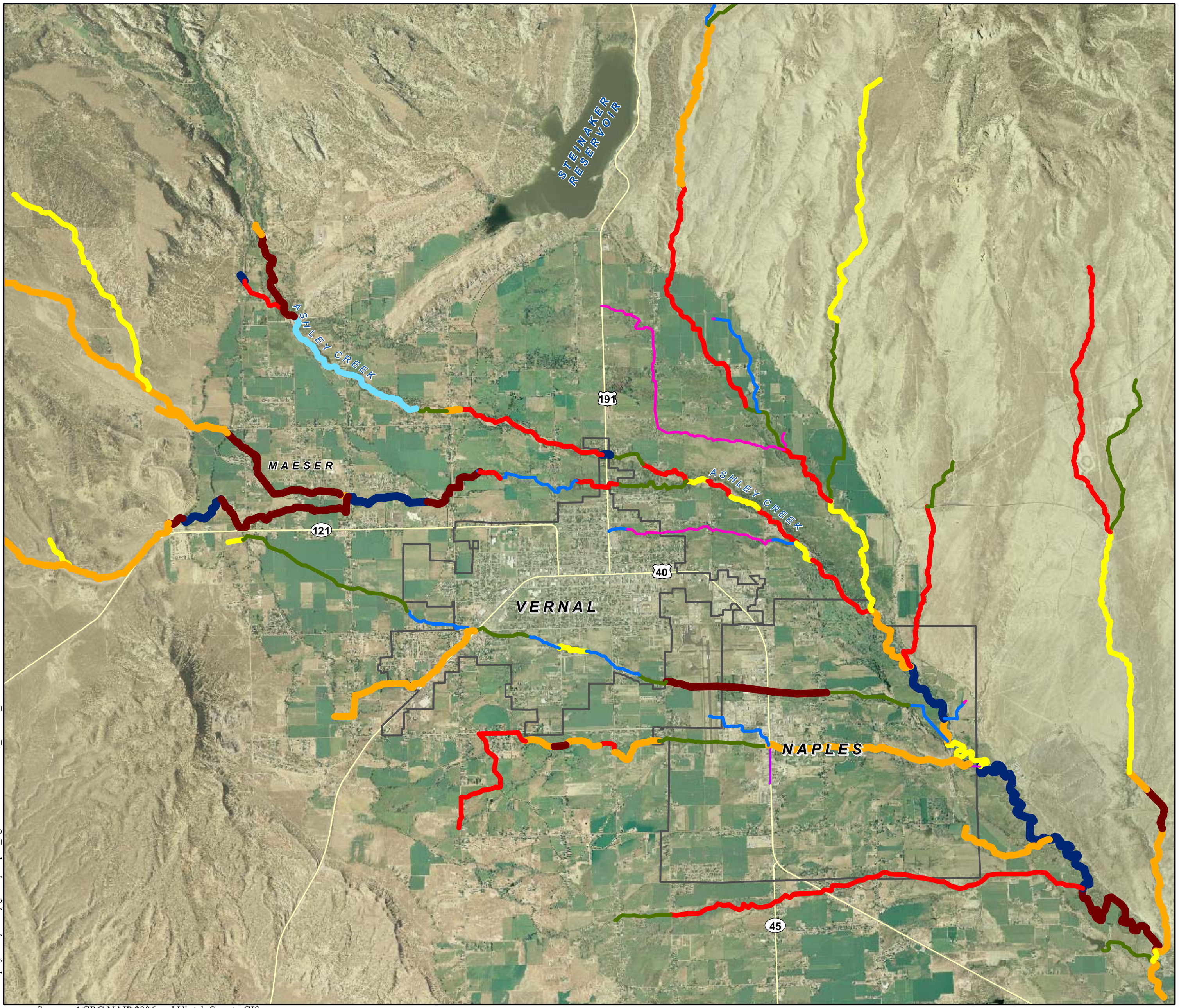

Source: AGRC NAIP 2006 and Uintah County GIS.

FIGURE 6-10
ESTIMATED FUTURE UNMODIFIED 100-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N S W E

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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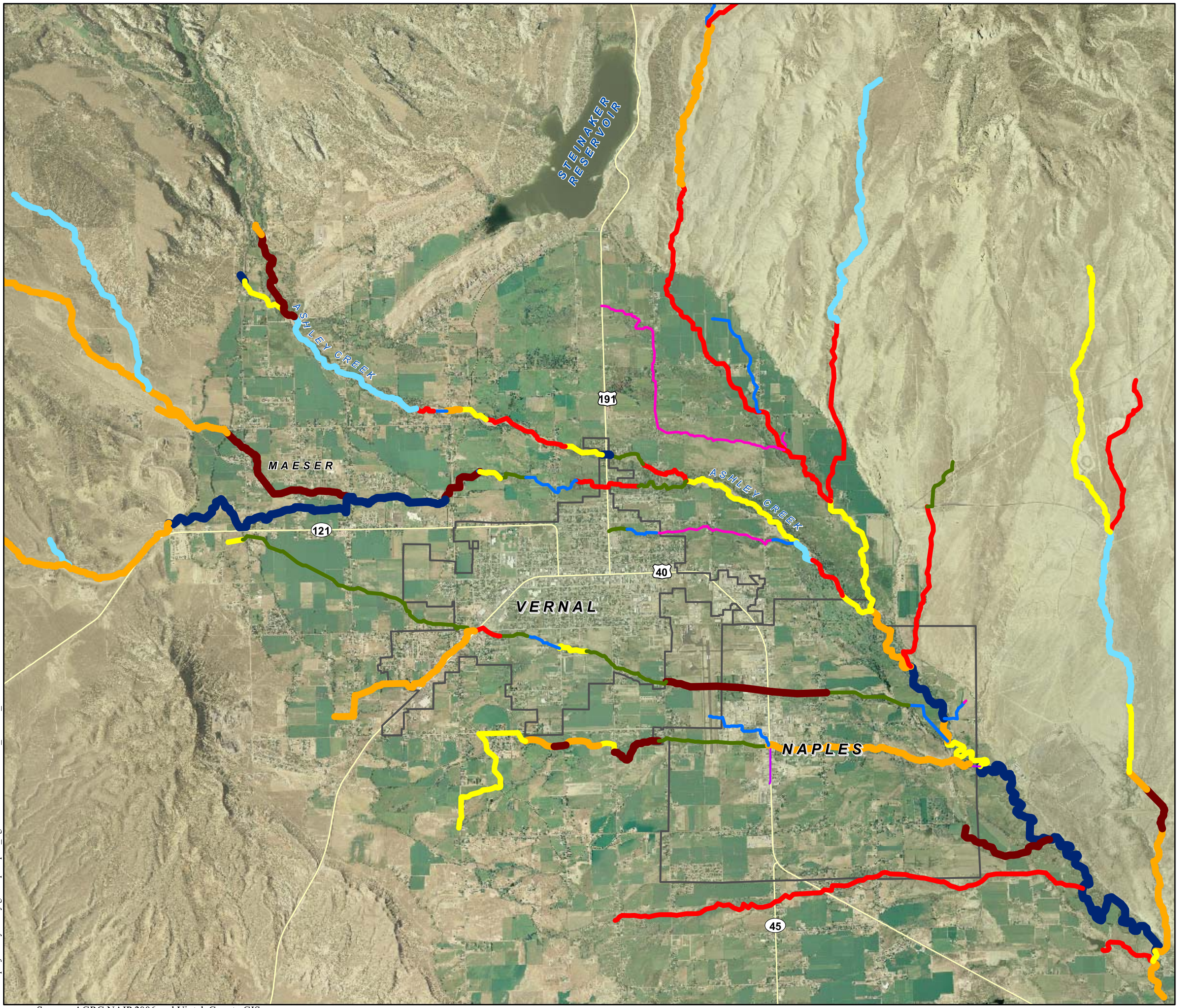
FIGURE 6-11
ESTIMATED FUTURE UNMODIFIED 500-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

Legend

- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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6.2.3 Capacity of Existing Drainage Facilities

A number of concerns were identified in the previous chapter under the existing simulated conditions. Future simulation indicates that all of the existing problems will likely be exacerbated through development. Additionally, the future simulation indicates that additional problems will occur if modifications to the drainages are not properly managed. Areas throughout the Valley where roads and utilities cross drainage channels are of high concern. In the previous chapter, a number of culverts were identified as unable to pass the 10-year event. Under the future conditions model, it appears that most of the major crossings are ill-equipped to pass the 25-year or larger event. While some roadway flooding may be permissible during large flooding events, it is imperative that major utility corridors and evacuation routes remain operable during even the most extreme events. The necessary upgrades to correct both the existing and future flooding concerns are discussed in detail in the following chapter.

Chapter 7 RECOMMENDED UPGRADES

A number of existing and potential storm water concerns have been identified in the previous chapters. This chapter presents a series of recommendations to mitigate the existing and potential flooding concerns. The methodology behind the recommended capital improvements as well as the estimated costs for the improvements is also presented below.

7.1 IMPROVEMENT METHODOLOGY SELECTION

Identifying storm water problems is a complex process, but it is also one that relies on technical expertise and proven scientific methods. Identifying alternatives to mitigate the identified problems can be far more challenging. In addition to requiring sound engineering and technical knowledge to identify effective solutions, a number of factors must be evaluated, including:

- Cost / Fundability
- Effectiveness
- Sustainability
- Liability
- Community Impact
- Political Climate
- Water users /Water rights
- Environmental effects
- Community acceptance
- Property rights
- Future land uses

With few exceptions, the list above does not include technical or easily quantifiable items. Therefore, involvement of the political entities is required to effectively implement the recommended improvements. To that end, Epic Engineering staff attended numerous city and county meetings as well as meeting with governmental planning staff in an effort to understand the communities needs and desires as well as inform them of the flooding concerns and work collectively to develop the recommendations methodology herein. It is our hope that by including the governmental entities throughout the process, the recommendations will be implemented by the respective entities and the Valley protected from future flooding events.

The sections below detail three recommendation selection methodologies that were discussed throughout the process. Each of the methodologies has strengths and weaknesses and none of the methodologies provide a perfect solution to all of the problems. After numerous discussions, a hybrid of the three methodologies was selected to provide the most optimal list of recommended improvements.

7.1.1 DO NOTHING METHODOLOGY

The ‘do nothing’, or the ‘don’t do anything new’ methodology is founded on the basis that flooding is a natural process and structures within the flood plain are not necessary the responsibility of the government to protect. With this logic, new development is responsible for managing the storm water on-site, and the local entities will not be responsible for flooding in the future.

At first glance this alternative appears to be the least costly since it does not require major improvements. However, damage costs associated with the ‘do nothing’ methodology after a large storm event could far exceed the costs of the other alternatives.

7.1.2 Historic Drainage Restoration and Preservation

Methodology behind preserving (or restoring where necessary) the natural drainages throughout the basin is based on the concept that water has naturally established the most effective flow paths over time. Allowing the storm water to follow its natural course provides two primary benefits. First, the stream channels are already defined and will require little improvement. Second, since the flows are naturally occurring, governing entities can designate the channels as un-developable more easily than if flooding occurred through artificial diversions.

The primary shortfall of this methodology is that a number of drainages have already been filled, developed, or altered to the point that it is not feasible or economically possible to restore the channel to its natural condition.

7.1.3 Storm Water Basin Diversion and Storage

Divert, store and protect methodology is fundamentally opposite from the ‘do nothing’ strategy. The strategy behind diverting, storing and protecting is to construct artificial storm channels and detention or debris basins in an effort to minimize the floodplains throughout the Valley. Storm events of all sizes will be managed through a series of pipes, canals, and diversions.

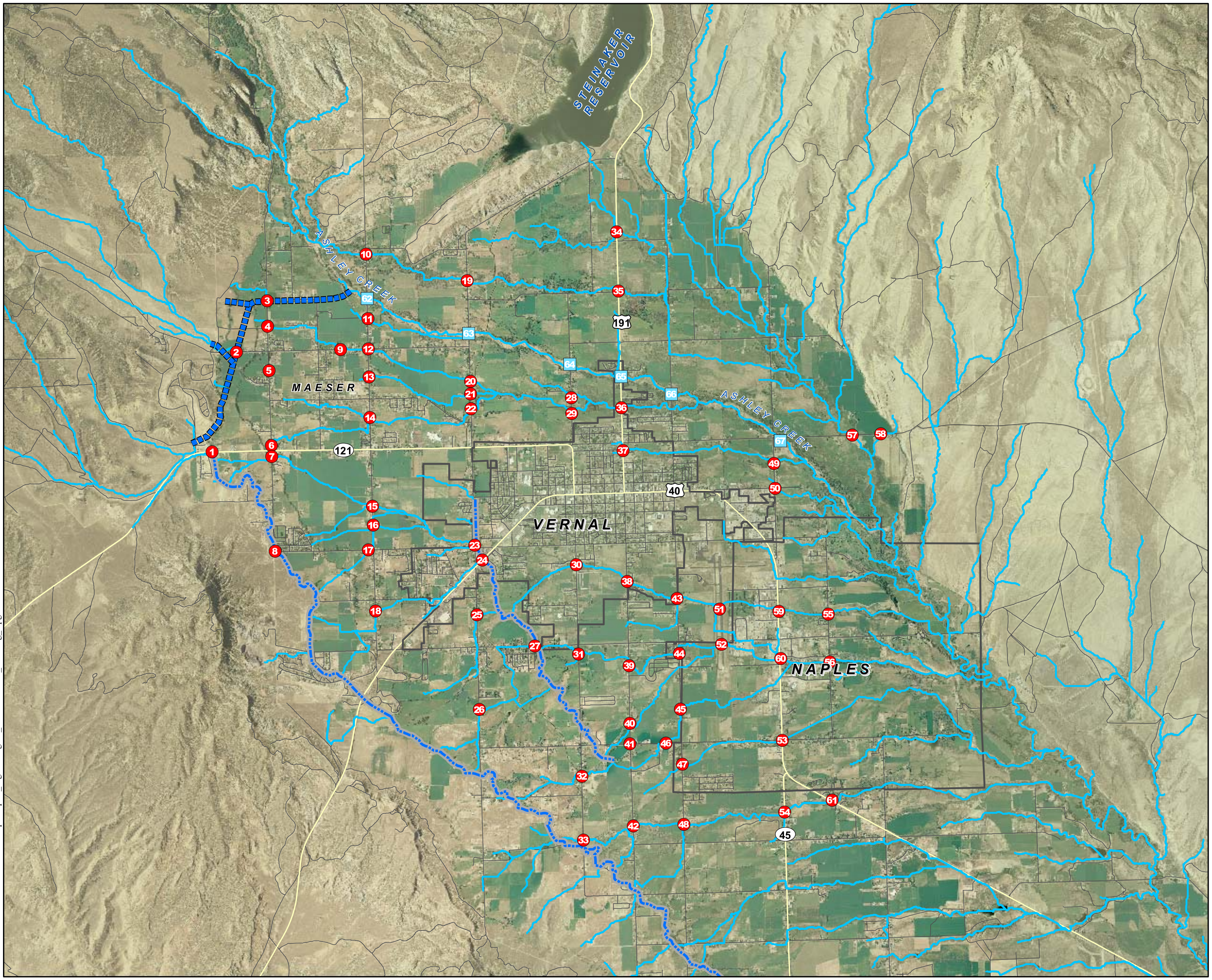
Applying this methodology to the entire basin would be extremely costly. Additionally, operation and maintenance of such a complex system would be labor intensive and the liability associated with a mechanical failure higher then with the other possible methodologies. The advantage to this methodology is that the floodplains would be minimized and could potentially allow for higher density developments closer to or within the low lying areas.

7.1.4 Recommended Methodology

After many discussions with the local municipalities it was determined that a hybrid of preservation and diversion methodologies was best suited for the Valley. The concept behind the recommended methodology is to preserve the natural drainage channels where possible, restore the natural channels when there are only minor encroachments and, finally, divert storm water from the highly developed areas where restoration of the natural channels is not feasible. This alternative is intended to minimize the cost and liability associated with implementation while providing adequate protection of the Valley. Additionally, preserving the natural drainages will provide open space for the community that can also serve as recreational corridors.

7.2 Recommended Upgrade Projects

The following sections describe the recommended alternatives based on the methodology described above. The upgrades have been classified into debris basins, detention basins to treat the storm water and remove storm water peaks, storm water canals to divert water away from developed areas, recommended road and utility crossing upgrades to ensure emergency ingress/egress is maintained, as well as the channels that should be protected and resorted to provide adequate capacity in the less developed areas. Proposed locations of these recommended improvement projects are shown in Figure 7-1 and Figure 7-2.



Source: AGRC NAIP 2006 and Uintah County GIS.

FIGURE 7-1
RECOMMENDED CROSSING UPGRADES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Natural Stream Channel

Major Road

Minor Road

New Channel

Channel Improvement

Municipal Boundary

Recommended Crossing Upgrade

1 Culvert

1 River Crossing Upgrade

N
W E
S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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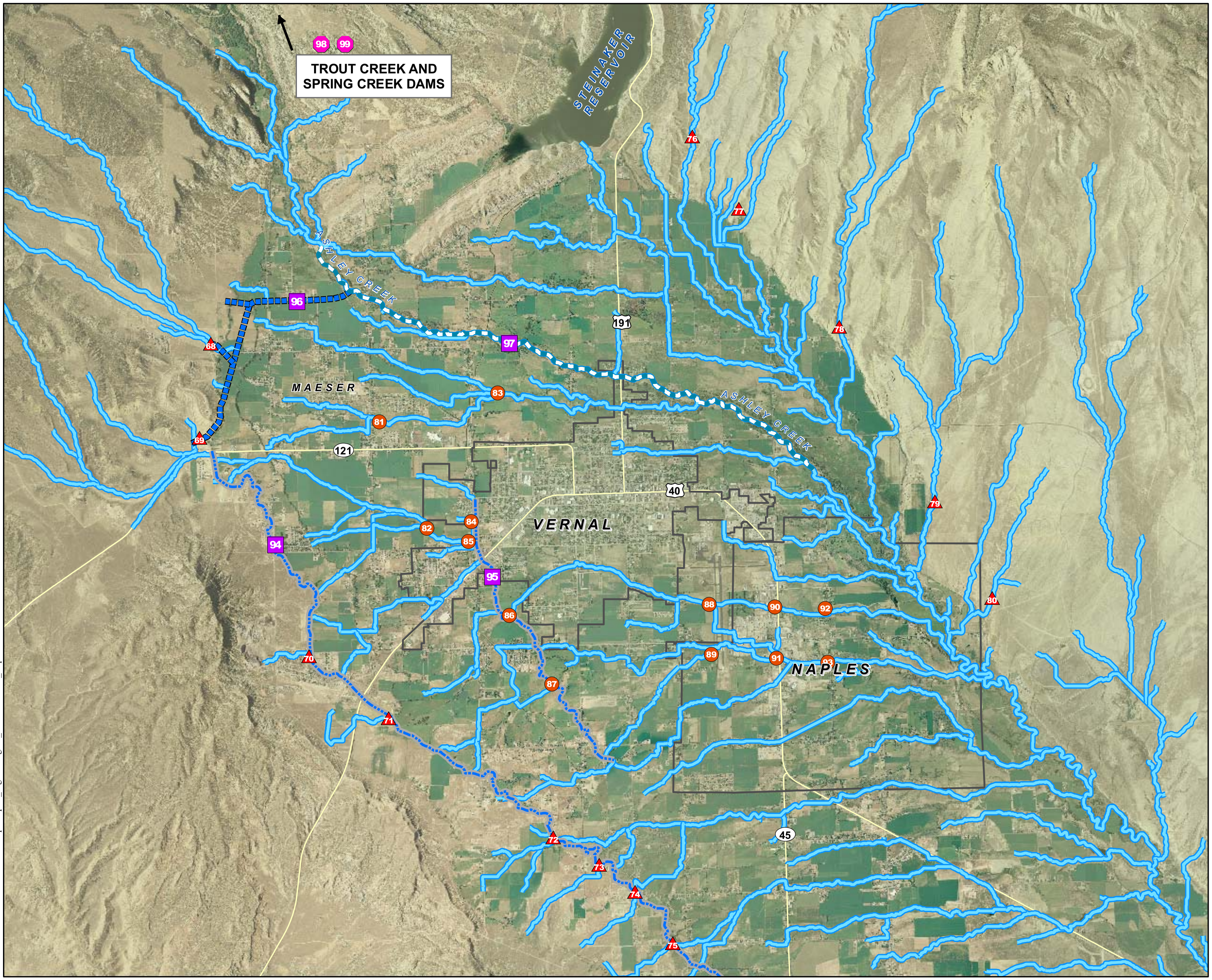

FIGURE 7-2
RECOMMENDED IMPROVEMENTS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Natural Stream Channel

Major Road

Municipal Boundary

Recommended Improvements

1 Debris Basin

1 Detention Pond

1 Dam

1 Channel Improvement

1 New Channel

Preserve and Protect

1 Restore Channel to Natural Conditions

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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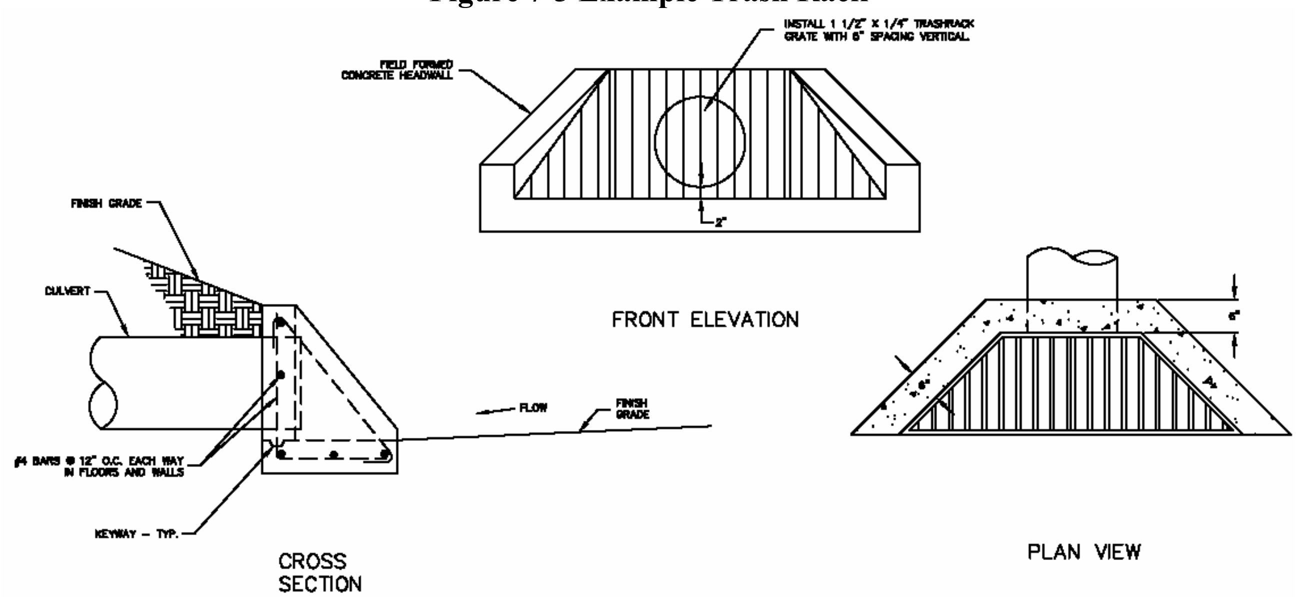
7.2.1 RECOMMENDED CROSSING UPGRADES

The following section describes the major recommended improvements that are required to divert flow around or reduce peak flow through developed areas of the Valley. A summary of the recommended improvements are listed in Table 7-1.

Analysis of numerous road and utility crossings throughout the basin indicates that many of the crossings are not currently equipped to be safely passable during the 10-year or larger event. During high flow events it is critical that key evacuation routes and utilities be maintained for the safety of the community. To that end, this report recommends that key crossings throughout the basin be improved to ensure they will remain operable. Additional road and utility corridor crossings should be upgraded to withstand a minimum of a 25-year or larger event to protect the Valley from frequent washouts and high replacement costs. It is recommended that the upgrades highlighted in Table 7-1 be constructed to ensure that utilities remain in operable condition and ingress / egress is maintained during extreme precipitation events. Note that that the recommended culvert crossing upgrades consistently recommend two or more parallel culverts.

During large storm events it is common for smaller crossings, such as culverts, to become blocked with debris even with well designed debris racks. Installing parallel culverts provides a level of redundancy to ensure that storm water will be conveyed even when partially blocked. ***It is recommended that multiple culverts with upstream trash racks, similar to Figure 7-3, be installed at all major existing and future crossings.***

Figure 7-3 Example Trash Rack



For the purposes of this report culverts were sized based on the flow and the nearest round culvert(s) that would provide the required capacity. While this concept provides an excellent idea of the required culvert size it is not intended to be all inclusive. When the crossing upgrades are designed ***it is recommended that site specific considerations be evaluated and a variety of culvert types considered including box culverts, squash pipe and bridges.***

Table 7-1 Recommended Crossing Upgrades

		Storm Event				Recommended Sizing and Barrels for Respective Storm Event							
		10	25	50	100	10		25		50		100	
Item	Location	Flow (cfs)				Size (in)	Barrels	Size (in)	Barrels	Size (in)	Barrels	Size (in)	Barrels
1	4105 W State 121 @ future canal	216	319	408	509	54	2	60	2	66	2	60	3
2	3850 W 1500 N @ future canal	386	562	693	829	66	2	66	3	66	3	66	4
3	2000 N 3500 W @ future canal	814	1190	1528	1821	60	5	72	5	78	5	78	5
4	1750 N 3500 W	17	24	31	39	24	2	24	2	24	2	30	2
5	1250 N 3500 W	1	2	3	4	18	2	18	2	18	2	18	2
6	550 N 3500 W	19	28	36	45	24	2	24	2	30	2	30	2
7	400 N 3500 W	78	113	145	179	36	2	42	2	48	2	48	2
8	500 S 3500 W	216	319	408	509	54	2	60	2	66	2	72	2
9	2750 W 1500 N	17	24	31	39	24	2	24	2	24	2	30	2
10	2450 N 2500 W	3	4	6	7	18	2	18	2	18	2	24	2
11	1800 N 2500 W	17	24	31	39	24	2	24	2	24	2	30	2
12	1500 N 2500 W	17	24	31	39	24	2	24	2	24	2	30	2
13	1200 N 2500 W	12	17	22	28	24	2	24	2	24	2	24	2
14	750 N 2500 W	20	30	39	49	24	2	24	2	30	2	30	2
15	100 S 2500 W	78	113	145	179	36	2	42	2	48	2	48	2
16	250 S 2500 W	78	113	145	179	36	2	42	2	48	2	48	2
17	500 S 2500 W	78	113	145	179	36	2	42	2	48	2	48	2
18	1100 S 2500 W	78	113	145	179	36	2	42	2	48	2	48	2
19	2200 N 1500 W	3	4	6	7	18	2	18	2	18	2	24	2
20	1200 N 1500 W	17	24	31	39	24	2	24	2	24	2	30	2
21	1000 N 1500 W	12	17	22	28	18	2	24	2	24	2	24	2
22	900 N 1500 W	19	30	39	49	24	2	24	2	30	2	30	2
23	450 S 1500 W	152	222	284	359	48	2	54	2	60	2	60	2
24	600 S 1400 W	147	213	259	299	48	2	54	2	54	2	60	2
25	1150 S 1500 W	84	126	162	203	36	2	42	2	48	2	48	2
26	2100 S 1500 W	84	126	162	203	36	2	42	2	48	2	48	2
27	900 W 1500 S	68	111	149	158	36	2	42	2	48	2	48	2
28	1000 N 500 W	3	4	6	8	18	2	18	2	18	2	18	2
29	750 N 500 W	45	67	86	108	30	2	36	2	36	2	42	2

		Storm Event				Recommended Sizing and Barrels for Respective Storm Event							
		10	25	50	100	10		25		50		100	
Item	Location	Flow (cfs)				Size (in)	Barrels	Size (in)	Barrels	Size (in)	Barrels	Size (in)	Barrels
30	700 S 500 W	141	176	200	222	42	2	48	2	48	2	54	2
31	1580 S 500 W	81	124	161	202	36	2	42	2	48	2	48	2
32	2800 S 500 W	110	163	208	260	42	2	48	2	54	2	54	2
33	3500 S 500 W	110	163	208	260	42	2	48	2	54	2	54	2
34	2600 N Vernal Ave	25	39	52	67	24	2	30	2	30	2	36	2
35	2000 N Vernal Ave	3	4	6	7	18	2	18	2	18	2	24	2
36	750 N Vernal Ave	48	71	92	115	30	2	36	2	36	2	42	2
37	400 N Vernal Ave	84	116	143	172	36	2	42	2	48	2	48	2
38	900 S Vernal Ave	140	174	198	221	42	2	48	2	48	2	54	2
39	1750 S Vernal Ave	90	138	180	224	36	2	42	2	48	2	54	2
40	2250 S Vernal Ave	110	163	208	260	42	2	48	2	54	2	54	2
41	2500 S Vernal Ave	110	163	208	260	42	2	48	2	54	2	54	2
42	3300 S Vernal Ave	110	163	208	260	42	2	48	2	54	2	54	2
43	1100 S 500 E	158	209	242	276	48	2	54	2	54	2	54	2
44	1580 S 500 E	110	166	215	272	42	2	48	2	54	2	54	2
45	2100 S 500 E	3	37	61	93	18	2	30	2	36	2	36	2
46	360 E 2500 S	3	37	61	93	18	2	30	2	36	2	36	2
47	2800 S 500 E	3	37	61	93	18	2	30	2	36	2	36	2
48	3300 S 500 E	110	163	208	260	42	2	48	2	54	2	54	2
49	250 N 1500 E	76	104	131	160	36	2	42	2	42	2	48	2
50	1500 E Main	25	35	44	53	24	2	30	2	30	2	30	2
51	1200 S Airport	127	180	211	248	42	2	48	2	54	2	54	2
52	1550 S Airport	86	147	192	236	36	2	48	2	48	2	54	2
53	US-40, 2500 S	3	37	61	93	18	2	30	2	36	2	36	2
54	3200 S 1500 E	110	163	208	260	42	2	48	2	54	2	54	2
55	1200 S 2000 E	120	160	199	238	42	2	48	2	48	2	54	2
56	1750 S 2000 E	89	185	254	329	36	2	48	2	54	2	60	2
57	2300 E State 121	578	822	1065	1262	66	3	66	4	72	4	72	5
58	2500 E State 121	176	250	307	375	48	2	54	2	60	2	66	2
59	HWY 40 and 1200 South	120	160	199	238	42	2	48	2	48	2	54	2
60	HWY 40 and 1700 South	89	185	254	329	36	2	48	2	54	2	60	2
61	HWY 40 and 3625 South	120	160	199	238	42	2	48	2	48	2	54	2

			Ashley Creek only		Ashley Creek & rainfall		Recommendation
Item	Location		100	500	100	500	
			Flow (cfs)				
62	2500 West and Ashley Creek		4,134	4,655	4,717.46	5,542.40	Bridge
63	1500 West and Ashley Creek		4,134	4,655	4,717.46	5,542.40	Bridge
64	500 West and Ashley Creek		4,134	4,655	4,767.13	5,625.10	Bridge
65	HWY 191 and Ashley Creek		4,134	4,655	4,767.13	5,625.10	Bridge
66	500 East and Ashley Creek		4,134	4,655	4,770.97	5,631.73	Bridge
67	500 North and Ashley Creek		4,134	4,655	6,838.95	8,658.80	Bridge

Table 7-1 provides recommended sizes for culverts to safely pass the 10- through 100-year event. On average it is recommend that these crossings be sized to pass a minimum of the 25-year event. However, at key locations such as primary roadways, the 50- and 100-year flows should be considered. Larger flows should also be considered in cases where backing up storm water would result in flooding. Where the impoundment of storm water could result in damage to structures the crossings should be designed to pass a minimum of the 100-year event. The crossing upgrades should be ranked as a medium priority.

7.2.2 RECOMMENDED CONTROL AND DIVERSION IMPROVEMENTS

The following table describes the major recommended control and diversion improvements that are required to divert flow around or reduce peak flow through developed areas of the Valley. A summary of the recommended improvements are listed in Table 7-2.

Table 7-2 Recommended Control and Diversion Improvements

Item	Approximate Location	Recommended Action	Units	Unit	Priority*
68	4000 West, 1500 North (Coalmine Basin)	Construct Large Debris Basin	160	AF	A
69	4000 West and 2000 North	Construct Large Debris Basin	112	AF	A
70	1500 South above Highline Canal	Construct Small Debris Basin	5	AF	B
71	3000 South above Highline Canal	Construct Small Debris Basin	5	AF	B
72	3300 South above Highline Canal	Construct Small Debris Basin	5	AF	B
73	3700 South above Highline Canal	Construct Small Debris Basin	5	AF	B
74	4000 South above Upper Ashley Canal	Construct Small Debris Basin	5	AF	B
75	5000 South above Upper Ashley Canal	Construct Small Debris Basin	5	AF	B
76	3300 North, 750 East	Construct Large Debris Basin	130	AF	B
77	1200 East, 2900 North	Construct Large Debris Basin	90	AF	B
78	2850 East, 1500 North	Construct Large Debris Basin	95	AF	B
79	500 South, 3200 East	Construct Large Debris Basin	120	AF	B
80	1400 South, 3900 East	Construct Large Debris Basin	75	AF	B
81	2400 West and 700 North	Construct Detention Pond	24	EA	B
82	1750 West and 350 South	Construct Detention Pond	25	EA	B
83	1200 West and 1000 North	Construct Detention Pond	20	EA	B
84	1580 West and 475 South	Construct Detention Pond	15	EA	B
85	1560 West and 300 South	Construct Detention Pond	20	EA	B
86	Ashley Central Canal at 1200 West and 1200 South	Construct Detention Pond	10	EA	B
87	Ashley Central Canal at 300 West and 2700 South	Construct Detention Pond	40	EA	B
88	800 East and 1100 South	Construct Detention Pond	50	EA	B
89	800 East and 1600 South	Construct Detention Pond	45	EA	B
90	HWY 40 and 1200 South	Construct Detention Pond	45	EA	B
91	HWY 40 and 1700 South	Construct Detention Pond	40	EA	B
92	2000 East and 1200 South	Construct Detention Pond	50	EA	B
93	2000 East and 1750 South	Construct Detention Pond	100	EA	B
94	Highline / Upper Ashley Canal from US-191 to ~4000 S	Construct Storm Water Canal	50,000	LF	C
95	Ashley Central Canal from 300 S to 2500 S	Construct Storm Water Canal	15,000	LF	C
96	US-191 & 4000 W to 3000 W & Ashley Creek	Construct Storm Water Canal	20,000	LF	C

Item	Approximate Location	Recommended Action	Units	Unit	Priority*
	Miscellaneous	Restore Natural Channel	10,000	LF	A

* Priority A: Short-term, B: Intermediate-term, C: Long-term

7.2.2.1 Debris Basins

Debris basins are recommended on the outskirts of the Valley where major drainages from the hillsides enter the Valley flats. As the name implies, the purpose of debris basins is to capture debris that flows down the mountain channels during high-flow events. Due to the nature of the local topology, the higher regions of the Valley are steep, resulting in high-energy storm water runoff which often mobilizes large debris such as rocks and tree limbs. When this debris enters the flat, lower energy Valley, the debris settles out and can potentially block key flow paths during flooding events. To ensure that the waterways remain free flowing during high-flow events, it is important that as much debris as possible be removed from the flow in a controlled manner. ***It is recommended that debris basins be constructed at the base of the major drainage basins.***

7.2.2.2 Detention Basins

Detention basins are recommended in numerous locations throughout the Valley. The purpose of the detention basin is to alter the storm water hydrograph. Existing flows generally result in high intensity, short duration peak flows that can cause large amounts of erosion and require a fairly large floodplain. Detention basins store the highest portion of the peak flow and instead release a smaller, controlled flow over a longer period of time. By constructing detention basins along the major drainages, the flows can be controlled to be less damaging, and allow for smaller, less costly downstream improvements to provide adequate protection. ***It is recommended that detention basins be constructed throughout the major channels within the Valley to minimize the peak flow and protect downstream channels and structures.***

7.2.2.3 Storm Water Canals

Construction of two major storm water canals is recommended in order to divert water around Vernal City and the community of Maeser. The first canal is located in the northwest corner of the Valley. The canal will divert water from the drainage near US-121 and from Coal Mine Basin north to Ashley Creek following an alignment generally between the Highline Canal and the Upper Ashley Canal. Working in tandem with debris basins, this canal will divert the majority of storm water that currently threatens Maeser and the northern portions of Vernal City.

The second recommended canal will follow the existing alignments of the Highline Canal and the Upper Ashley Canal beginning at US-121 and running south around the Valley and either diverting storm water into adjacent canals or carrying flow all the way to the Green River. This canal will collect the highest runoff of the Valley and serve to collect much of the debris that currently runs off the hillsides. The canal will also provide a means to divert some storm water away from channels that may be experiencing capacity limitations or have not yet been fully upgraded. ***It is recommended that two canals be constructed to divert storm water around the key development areas of Maeser and Vernal City.***

7.2.3 ASHLEY CREEK IMPROVEMENTS

As stated previously, the major drainage through the Valley is Ashley Creek. Over the years, portions of this drainage have been modified in an attempt to increase channel capacity, limit flooding, divert flows for irrigation and to provide transportation and utility crossings. [1]The largest single change to Ashley Creek occurred in the 1960's when the Army Corps channelized and straightened a reach of Ashley Creek from the Thornburgh Diversion to approximately the golf course. The intent of this project was to increase the capacity and reduce flooding of the main channel. Providing additional capacity in the main channel allowed the historic meanders of Ashley Creek (the north and south channels) to be developed for agricultural and urbanization. The project increased the bed slope by approximately 50%, removed the meanders and provided sufficient capacity for approximately the 50 year event. The increased main channel capacity resulted in increased erosion, and ultimately, stream instability.

A detailed study of the stability of Ashley Creek was conducted in 1998 and 2000 by Mussetter Engineering Incorporated (MEI). The study indicated that the increased sediment transport and subsequent downstream deposition will likely continue to modify the river channel and may result in increased flooding potential near and below the golf course. Additionally, excessive erosion between the Thornburgh Diversion and the golf course will eventually result in channel migration and threaten existing structures. The bridges across Ashley Creek are also noted as undersized, which results in local flooding and sediment deposition.

Also, in May 2000 MEI and Franson Noble & Assoicates, Inc published a Stabilization/Restoration Report based on the MEI analysis. The alternatives for stream rehabilitation ranged from no changes to complete restoration of the entire channel reach. Erosion control measures, debris basins, and dam construction were also evaluated as part of the study. The study also considered diverting high water flows into the irrigation canals to relieve the peak flow from Ashley Creek. The basin-wide flood analysis contained herein suggests that the canals will fill with storm water from sources other than Ashley Creek, and as development of the Valley continues, locations to turn out the water will become more limited. ***It is recommended that the irrigation canals not be used as part of the Ashley Creek flood control project so that they can be used to control other flooding concerns throughout the Valley.***

Each of the proposed alternatives in the May 2000 report was compared to the flood protection methodology recommended in this report, "to protect and restore drainages where possible, and divert where necessary." The Ashley Creek improvement project alternative that is most closely aligned with the recommended methodology is alternative 9. This alternative consists of the following parameters and specific major projects described in Table 7-3:

1) Creek management to develop a monitoring and maintenance program;

2) Bridge enlargement (discussed in the previous section);

3) Soft Bank Stabilization to control erosion;

[1] Historic information summarized from Hydraulic and Geomorphic Analyses May 2000 MEI

4) Riparian restoration to reduce stream velocities and provide numerous other desirable benefits;

5) Provide upstream storage to minimize peak flows and provide water to future riparian zones.

Table 7-3 Recommended Ashley Creek Improvements

Item	Approximate Location	Recommended Action	Units	Unit	Priority*
97	Ashley Creek from Thornburgh Diversion to Golf Course	Restore Natural Channel	330,000	LF	C
98	Trout Creek Dam	Construct Spring Creek (or equivalent) Dam	1	EA	C
99	Spring Creek Drainage above Ashley Creek	Construct Spring Creek (or equivalentl) Dam	1	EA	C
100	20% of area above Thornburg Diversion	Watershed management	30,000	AC	B

* Priority A: Short-term, B: Intermediate-term, C: Long-term

It is recommended that the modified version of alternative 9 be implemented to restore Ashley Creek and mitigate future flooding concerns and minimize sediment transport.

Providing additional storage reservoir(s) above the Valley may become a controversial and environmentally challenging project to obtain funding and the necessary permits. While it is the preferred alternative in this report it may not be a feasible flood protection alternative. In the event that upstream storage cannot be constructed, the next best alternative for Ashley Creek would be to provide a series of small in-stream debris basins and deepen the channel to provide additional capacity through the developed areas of the Valley.

7.2.4 Irrigation Canals

As discussed previously in this document there are a number of locations throughout the Valley where storm water is directed into the irrigation network. As the cities grow this co-mingled water can cause diminished water quality. ***It is recommended that future construction projects be required to maintain separate conduits for irrigation and storm water, and that existing storm water discharge into irrigation channels be modified to maintain the required separation as future improvements are constructed throughout the region.***

7.3 OPINION OF PROBABLE IMPROVEMENT COST

The following section provides a cost estimate to construct the projects described in the sections above. These costs are based on estimates for excavation, engineered fill, storm water piping and other construction activities obtained from 2007 and 2008, in addition to engineering judgment. Additional detail describing the basis for these costs is provided in the Appendix. The costs provided are intended to provide an approximate funding price tag. These costs do not include property acquisition (with the exception of detention basins), replacement of other utilities, or costs not directly associated with the design and construction of the recommended improvement. The opinion of probable costs is presented in 2008 U.S. dollars, ENR cost index 8,184.94; no attempt to project the future cost of these improvements is presented herein. Table 7-4 below presents the estimated unit costs to construct the general types of improvements described above.

Table 7-4 Opinion of Probable Unit Construction Costs

Improvement	Base Cost Unit Cost	Base Cost Unit	Incremental Cost Unit Cost	Incremental Cost Unit
*Canal Construction / Reconstruction	$ 51.00	LF	$ 0.20	CFS-LF
*Concrete Levee Construction	$ 541.00	LF	$ -	-
*Earth Levee Construction	$ 393.00	LF	$ -	-
*Detention Basin Construction	$ 42,621.00	EA	$ 2,640.00	AF
*Debris Basin Construction	$ 282,710.00	EA	$ 2,640.00	AF
**Bridge Replacement	$ 320,000.00	EA	$ -	-
**Stream Rehabilitation	$ 120.00	LF	$ -	-
**Spring Creek Dam	$38,000,000.00	EA	$ -	-
**Watershed Management	$ 500.00	AC	$ -	-
Increase Culvert – 18 in.	$ 106.75	LF	$ -	-
Increase Culvert – 21 in.	$ 91.50	LF	$ -	-
Increase Culvert – 24 in.	$ 97.60	LF	$ -	-
Increase Culvert – 27 in.	$ 109.80	LF	$ -	-
Increase Culvert – 30 in.	$ 122.00	LF	$ -	-
Increase Culvert – 36 in.	$ 146.40	LF	$ -	-
Increase Culvert – 42 in.	$ 183.01	LF	$ -	-
Increase Culvert – 48 in.	$ 231.81	LF	$ -	-
Increase Culvert – 54 in.	$ 274.51	LF	$ -	-
Increase Culvert – 60 in.	$ 301.96	LF	$ -	-
Increase Culvert – 66 in.	$ 305.01	LF	$ -	-
Increase Culvert – 72 in.	$ 366.01	LF	$ -	-
Increase Culvert – 78 in.	$ 475.81	LF	$ -	-
Increase Culvert – 84 in.	$ 640.52	LF	$ -	-
Increase Culvert – 90 in.	$ 869.27	LF	$ -	-
Increase Culvert – 96 in.	$ 1,098.03	LF	$ -	-
Increase Culvert – 102 in.	$ 1,296.29	LF	$ -	-

* Costs do not include land acquisition
** Costs from Franson-Noble & Associates, Inc May 2000 report Table 4-1 Cost Estimates for Components plus 3% annual inflation

In addition to the estimated direct construction costs, the design, construction management, legal, and administrative costs must also be considered. This report assigns overhead costs as a percentage of the raw construction cost estimates as shown in Table 7-5.

Table 7-5 Administrative Fees

Item	Percent of Construction Cost
Engineering	8%
Construction Management and Survey	7%
Administration	2%
Legal	1%
Contingency	15%
Total	33%

Based on the unit costs described above, an estimated cost for each of the recommended construction projects are shown in Table 7-7, Table 7-6, and Table 7-8 below.

Table 7-6 Opinion of Probable Crossing Improvement Cost

		Storm Event			
Item	Location	10	25	50	100
1	4105 W State 121 @ future canal	$ 62,573.37	$ 82,872.33	$ 106,853.81	$ 124,308.49
2	3850 W 1500 N @ future canal	$ 106,853.81	$ 160,280.72	$ 160,280.72	$ 213,707.63
3	2000 N 3500 W @ future canal	$ 207,180.82	$ 336,898.86	$ 417,059.95	$ 417,059.95
4	1750 N 3500 W	$ 7,198.40	$ 7,198.40	$ 7,198.40	$ 13,051.57
5	1250 N 3500 W	$ 3,342.46	$ 3,342.46	$ 3,342.46	$ 3,342.46
6	550 N 3500 W	$ 7,198.40	$ 7,198.40	$ 13,051.57	$ 13,051.57
7	400 N 3500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
8	500 S 3500 W	$ 62,573.37	$ 82,872.33	$ 106,853.81	$ 134,759.54
9	2750 W 1500 N	$ 7,198.40	$ 7,198.40	$ 7,198.40	$ 13,051.57
10	2450 N 2500 W	$ 3,342.46	$ 3,342.46	$ 3,342.46	$ 7,198.40
11	1800 N 2500 W	$ 7,198.40	$ 7,198.40	$ 7,198.40	$ 13,051.57
12	1500 N 2500 W	$ 7,198.40	$ 7,198.40	$ 7,198.40	$ 13,051.57
13	1200 N 2500 W	$ 7,198.40	$ 7,198.40	$ 7,198.40	$ 7,198.40
14	750 N 2500 W	$ 7,198.40	$ 7,198.40	$ 13,051.57	$ 13,051.57
15	100 S 2500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
16	250 S 2500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
17	500 S 2500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
18	1100 S 2500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
19	2200 N 1500 W	$ 3,342.46	$ 3,342.46	$ 3,342.46	$ 7,198.40
20	1200 N 1500 W	$ 7,198.40	$ 7,198.40	$ 7,198.40	$ 13,051.57
21	1000 N 1500 W	$ 3,342.46	$ 7,198.40	$ 7,198.40	$ 7,198.40
22	900 N 1500 W	$ 7,198.40	$ 7,198.40	$ 13,051.57	$ 13,051.57
23	450 S 1500 W	$ 45,707.01	$ 62,573.37	$ 82,872.33	$ 82,872.33
24	600 S 1400 W	$ 45,707.01	$ 62,573.37	$ 62,573.37	$ 82,872.33
25	1150 S 1500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
26	2100 S 1500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
27	900 W 1500 S	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
28	1000 N 500 W	$ 3,342.46	$ 3,342.46	$ 3,342.46	$ 3,342.46
29	750 N 500 W	$ 13,051.57	$ 21,223.30	$ 21,223.30	$ 32,013.83
30	700 S 500 W	$ 32,013.83	$ 45,707.01	$ 45,707.01	$ 62,573.37
31	1580 S 500 W	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
32	2800 S 500 W	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
33	3500 S 500 W	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
34	2600 N Vernal Ave	$ 7,198.40	$ 13,051.57	$ 13,051.57	$ 21,223.30
35	2000 N Vernal Ave	$ 3,342.46	$ 3,342.46	$ 3,342.46	$ 7,198.40
36	750 N Vernal Ave	$ 13,051.57	$ 21,223.30	$ 21,223.30	$ 32,013.83
37	400 N Vernal Ave	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 45,707.01
38	900 S Vernal Ave	$ 32,013.83	$ 45,707.01	$ 45,707.01	$ 62,573.37
39	1750 S Vernal Ave	$ 21,223.30	$ 32,013.83	$ 45,707.01	$ 62,573.37
40	2250 S Vernal Ave	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
41	2500 S Vernal Ave	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37

		Storm Event			
Item	Location	10	25	50	100
42	3300 S Vernal Ave	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
43	1100 S 500 E	$ 45,707.01	$ 62,573.37	$ 62,573.37	$ 62,573.37
44	1580 S 500 E	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
45	2100 S 500 E	$ 3,342.46	$ 13,051.57	$ 21,223.30	$ 21,223.30
46	360 E 2500 S	$ 3,342.46	$ 13,051.57	$ 21,223.30	$ 21,223.30
47	2800 S 500 E	$ 3,342.46	$ 13,051.57	$ 21,223.30	$ 21,223.30
48	3300 S 500 E	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
49	250 N 1500 E	$ 21,223.30	$ 32,013.83	$ 32,013.83	$ 45,707.01
50	1500 E Main	$ 7,198.40	$ 13,051.57	$ 13,051.57	$ 13,051.57
51	1200 S Airport	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
52	1550 S Airport	$ 21,223.30	$ 45,707.01	$ 45,707.01	$ 62,573.37
53	US-40, 2500 S	$ 3,342.46	$ 13,051.57	$ 21,223.30	$ 21,223.30
54	3200 S 1500 E	$ 32,013.83	$ 45,707.01	$ 62,573.37	$ 62,573.37
55	1200 S 2000 E	$ 32,013.83	$ 45,707.01	$ 45,707.01	$ 62,573.37
56	1750 S 2000 E	$ 21,223.30	$ 45,707.01	$ 62,573.37	$ 82,872.33
57	2300 E State 121	$ 160,280.72	$ 213,707.63	$ 269,519.08	$ 336,898.86
58	2500 E State 121	$ 45,707.01	$ 62,573.37	$ 82,872.33	$ 106,853.81
59	HWY 40 and 1200 South	$ 32,013.83	$ 45,707.01	$ 45,707.01	$ 62,573.37
60	HWY 40 and 1700 South	$ 21,223.30	$ 45,707.01	$ 62,573.37	$ 82,872.33
61	HWY 40 and 3625 South	$ 32,013.83	$ 45,707.01	$ 45,707.01	$ 62,573.37
		Ashley Creek only		Ashley Creek & rainfall	
		Storm Event		Storm Event	
		100	500	100	500
62	2500 West and Ashley Creek	$ 320,000.00	$ 320,000.00	$ 320,000.00	$ 320,000.00
63	1500 West and Ashley Creek	$ 320,000.00	$ 320,000.00	$ 320,000.00	$ 320,000.00
64	500 West and Ashley Creek	$ 320,000.00	$ 320,000.00	$ 320,000.00	$ 320,000.00
65	HWY 191 and Ashley Creek	$ 320,000.00	$ 320,000.00	$ 320,000.00	$ 320,000.00
66	500 East and Ashley Creek	$ 320,000.00	$ 320,000.00	$ 320,000.00	$ 320,000.00
67	500 North and Ashley Creek	$ 320,000.00	$ 320,000.00	$ 320,000.00	$ 320,000.00
	Totals	**$ 3,607,643.44**	**$ 4,418,062.91**	**$ 5,028,597.71**	**$ 5,482,387.32**

Table 7-7 Opinion of Probable Control and Diversion Improvement Cost

Item	Approximate Location	Recommended Action	Estimated Cost
68	4000 West, 1500 North (Coalmine Basin)	Construct Large Debris Basin	$ 937,796.30
69	4000 West and 2000 North	Construct Large Debris Basin	$ 769,258.70
70	1500 South above Highline Canal	Construct Small Debris Basin	$ 31,727.48
71	3000 South above Highline Canal	Construct Small Debris Basin	$ 31,727.48
72	3300 South above Highline Canal	Construct Small Debris Basin	$ 31,727.48
73	3700 South above Highline Canal	Construct Small Debris Basin	$ 31,727.48
74	4000 South above Upper Ashley Canal	Construct Small Debris Basin	$ 31,727.48
75	5000 South above Upper Ashley Canal	Construct Small Debris Basin	$ 31,727.48
76	3300 North, 750 East	Construct Large Debris Basin	$ 832,460.30
77	1200 East, 2900 North	Construct Large Debris Basin	$ 692,012.30
78	2850 East, 1500 North	Construct Large Debris Basin	$ 709,568.30
79	500 South, 3200 East	Construct Large Debris Basin	$ 797,348.30
80	1400 South, 3900 East	Construct Large Debris Basin	$ 639,344.30
81	2400 West and 700 North	Construct Detention Pond	$ 140,954.73
82	1750 West and 350 South	Construct Detention Pond	$ 144,465.93
83	1200 West and 1000 North	Construct Detention Pond	$ 126,909.93
84	1580 West and 475 South	Construct Detention Pond	$ 109,353.93
85	1560 West and 300 South	Construct Detention Pond	$ 126,909.93
86	Ashley Central Canal at 1200 West and 1200 South	Construct Detention Pond	$ 91,797.93
87	Ashley Central Canal at 300 West and 2700 South	Construct Detention Pond	$ 197,133.93
88	800 East and 1100 South	Construct Detention Pond	$ 232,245.93
89	800 East and 1600 South	Construct Detention Pond	$ 214,689.93
90	HWY 40 and 1200 South	Construct Detention Pond	$ 214,689.93
91	HWY 40 and 1700 South	Construct Detention Pond	$ 197,133.93
92	2000 East and 1200 South	Construct Detention Pond	$ 232,245.93
93	2000 East and 1750 South	Construct Detention Pond	$ 407,805.93
94	Highline / Upper Ashley Canal from US191 to~ 4000 S	Construct Storm Water Canal	$ 3,391,766.00
95	Ashley Central Canal from 300 S to 2500 S	Construct Storm Water Canal	$ 1,017,556.40
96	US-191 & 4000 W to 3000 W & Ashley Creek	Construct Storm Water Canal	$ 1,356,999.00
	Misc.	*Restore Natural Channel	$ 1,596,000.00
		Total	**$ 15,366,812.69**

Table 7-8 Opinion of Probable Ashley Creek Improvement Cost

Item	Approximate Location	Recommended Action	Estimated Cost
97	Ashley Creek from Thornburgh Diversion to Golf Course	*Restore Natural Channel	$ 52,668,000.00
98	Trout Creek Dam	* Construct Spring Creek (or equivalent) Dam	$ 50,540,000.00
99	Spring Creek Drainage above Ashley Creek	* Construct Spring Creek (or equivalent) Dam	$ 66,500,000.00
100	20% of area above Thornburg Diversion	Watershed Management	$ 19,950,000.00
		Total	**$ 189,658,000.00**

7.4 RECOMMENDED STORM WATER POLICIES

A number of policy changes will be required to protect the Valley from flooding. The most important policy change is to require that all of the remaining natural drainages be preserved. Other policy issues that should be evaluated are the requirements for storm water management under conditions of new development. These policy issues are discussed in more detail below.

7.4.1 DESIGNATED FLOODWAY PROTECTION / RESTORATION

In addition to the recommended improvements discussed above, the key component to ensuring that both existing and future developments are protected from flooding is to ensure that the remaining natural channels be preserved. Currently, there are no clearly defined policies in place to prevent the development of a historic floodway. For the plan proposed herein it is imperative that each of the three major governing entities within the Valley adopt policies that do not allow development within or modification of natural floodways, and prohibit the rebuilding of existing structures within floodways. The major channels are highlighted in Table 7-2 above.

The second and potentially more difficult portion of the recommended methodology is to restore drainages where possible. There are a number of drainages throughout the basin that are largely intact and can be preserved for future flows. However, in one or two locations these channels have been modified and developed. It is recommended that the channels shown in Table 7-2 as preserve and protect be restored or reconstructed as required to maintain the historic channel capacity. One example of a floodway that has been developed is along the drainage channel south of Vernal near 500E. The channel in this location has been filled and the historic drainage capacity significantly diminished.

It is recommended that each governing adjacencies modify their zoning code and ordinances, etc., to reflect the following actions:

- ***Prohibit development within existing flood channels highlighted in Table 7-2;***
- ***Prohibit the modification, including piping, of major drainage channels;***
- ***Prohibit the reconstruction of developments currently within the existing flood channels.***

7.4.2 FUTURE DEVELOPMENT REQUIREMENTS

In addition to preventing development in flood channels it is imperative that the flood channels are not obstructed or filled through future roadways or similar development.

7.4.2.1 Onsite Detention / Retention

One of the key assumptions throughout the modeling is that the local municipalities will continue to require detention or retention for each new development. Continuing to require local retention / detention will preserve the existing flow patterns which will keep the high water flows in the banks. of the existing channels. The regional detention basins described above are intended to reduce the peak flows and velocities through key areas. They are not intended to replace or diminish the requirements for local detention basins.

It is recommended that each municipality adopt or continue to include requirements on new development that:

- ***Require local detention/ retention of storm water for all new development;***
- ***Require that each detention/ retention basin contain an overflow designed to safely discharge the 100-year flow into a natural stream channel;***
- ***That the basins be designed such that the final discharge is less then historical peak flows for the 10-, 50-, and 100-year storm events.***

7.4.2.2 Parks, Open Space and Trail System

To offset the costs, monetary and otherwise by requiring the flood channels be preserved, ***it is recommended that the preserved flood corridors be preserved through open space credits and to potentially provide trail corridors and parks.***

7.5 SIMULATED PEAK FLOWS WITH IMPROVEMENTS

Once the recommended improvements were identified and conceptually designed, they were entered into the model to determine: 1) the size required for each improvement, 2) the downstream flows with the recommended system working and 3) to ensure that large storm events will pass through the communities without major flooding when the recommended improvements are in place. Figure 7-4, Figure 7-5, Figure 7-6, Figure 7-7 and Figure 7-8 below indicate the anticipated modified peak flows from the respective storm events utilizing the recommended improvements.

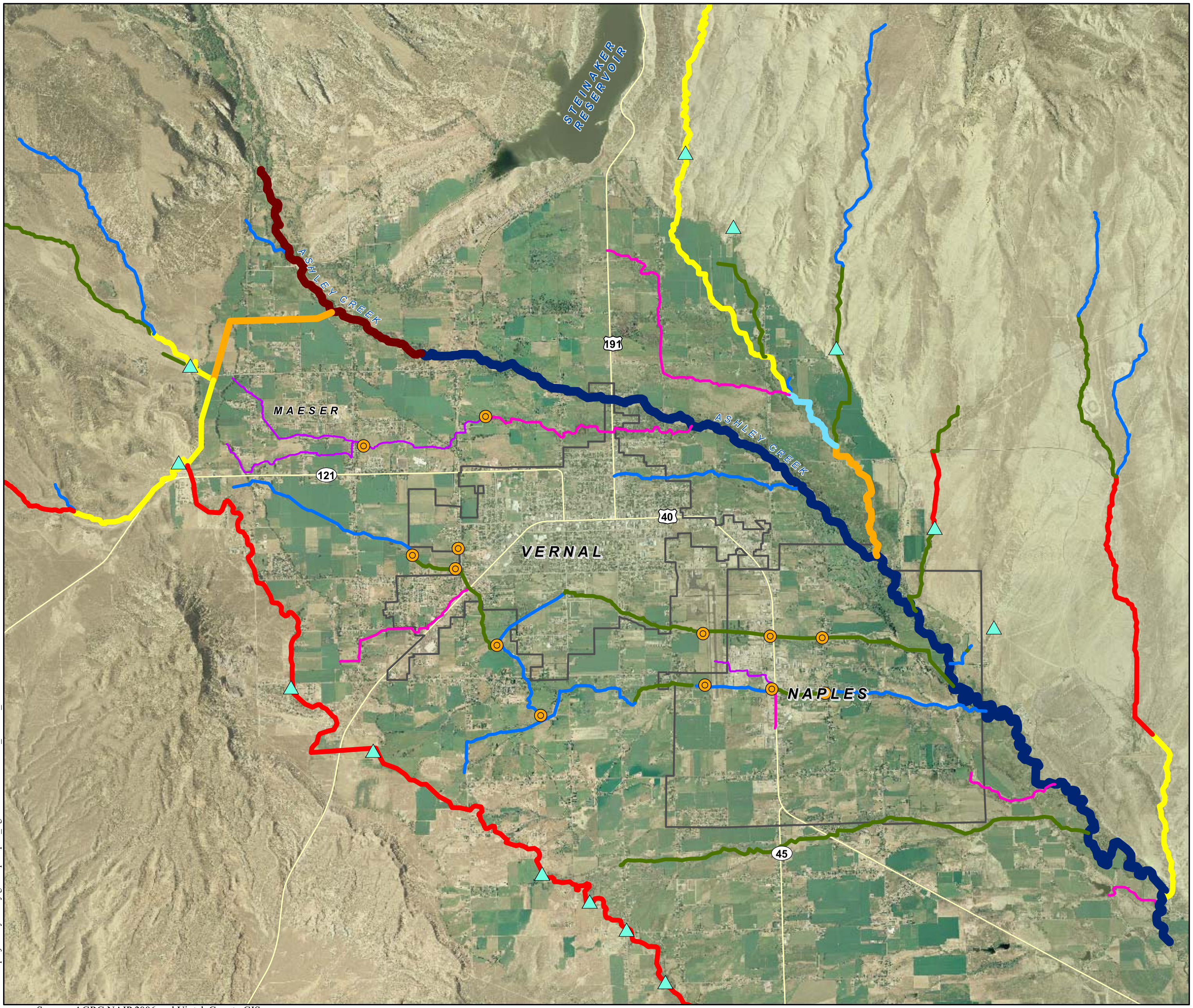


FIGURE 7-4
ESTIMATED FUTURE MODIFIED 10-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

Debris Basin

Detention Pond

Major Road

Municipal Boundary

Storm Event Maximum Flow (cfs)

< 25

26 - 50

51 - 100

101 - 200

201 - 300

301 - 500

501 - 700

701 - 1,000

1,001 - 1,500

1,501 - 3,000

3,001 - 6,000

> 6,000

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

Epic Engineering

3341 SOUTH 4000 WEST
WEST VALLEY CITY, UTAH 84120
(801) 955-5605

50 EAST 100 SOUTH
HEBER CITY, UTAH 84032
(435) 654-6600

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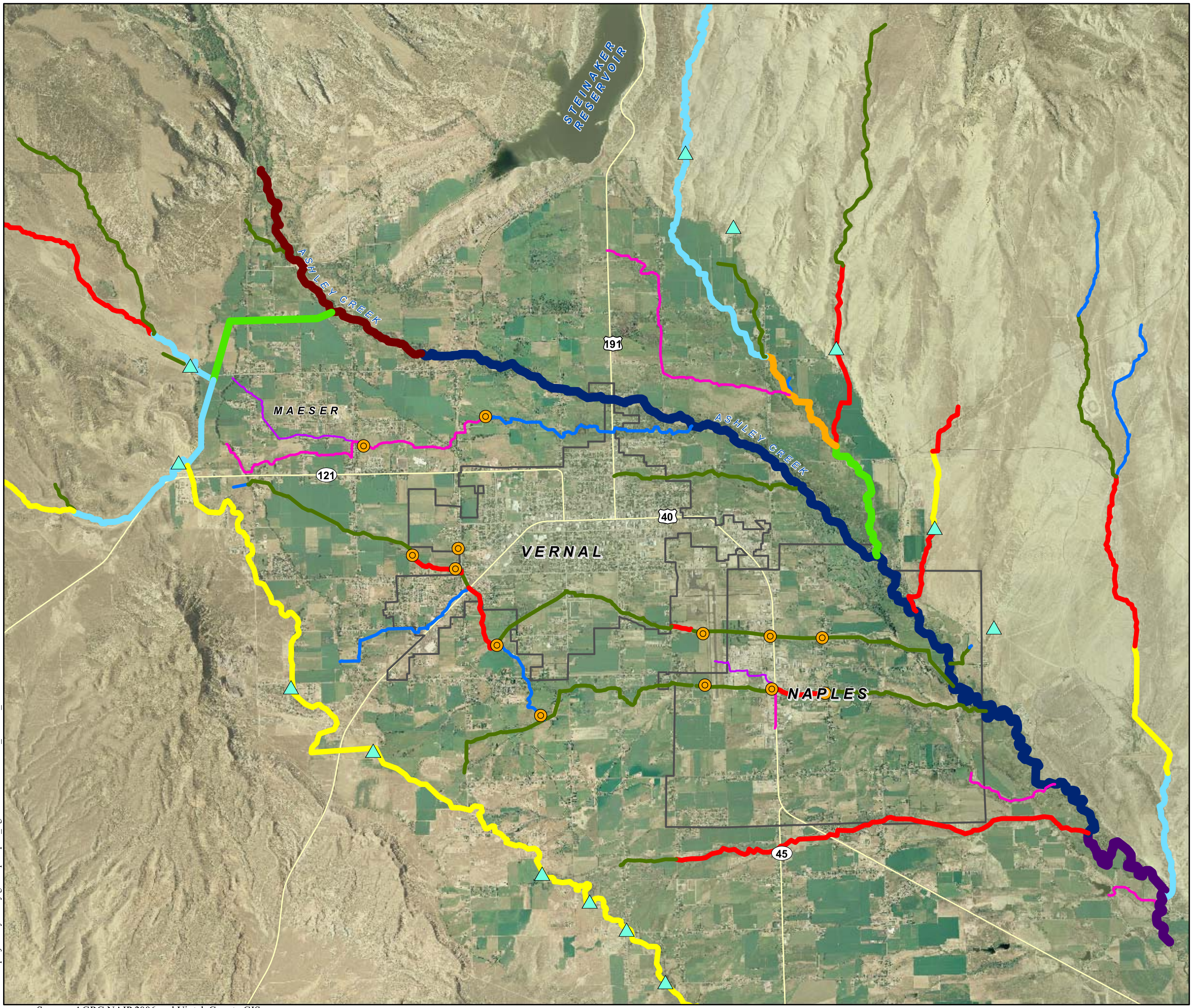
Source: AGRC NAIP 2006 and Uintah County GIS.



Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 7-5
ESTIMATED FUTURE MODIFIED 25-YEAR CHANNEL FLOWS ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

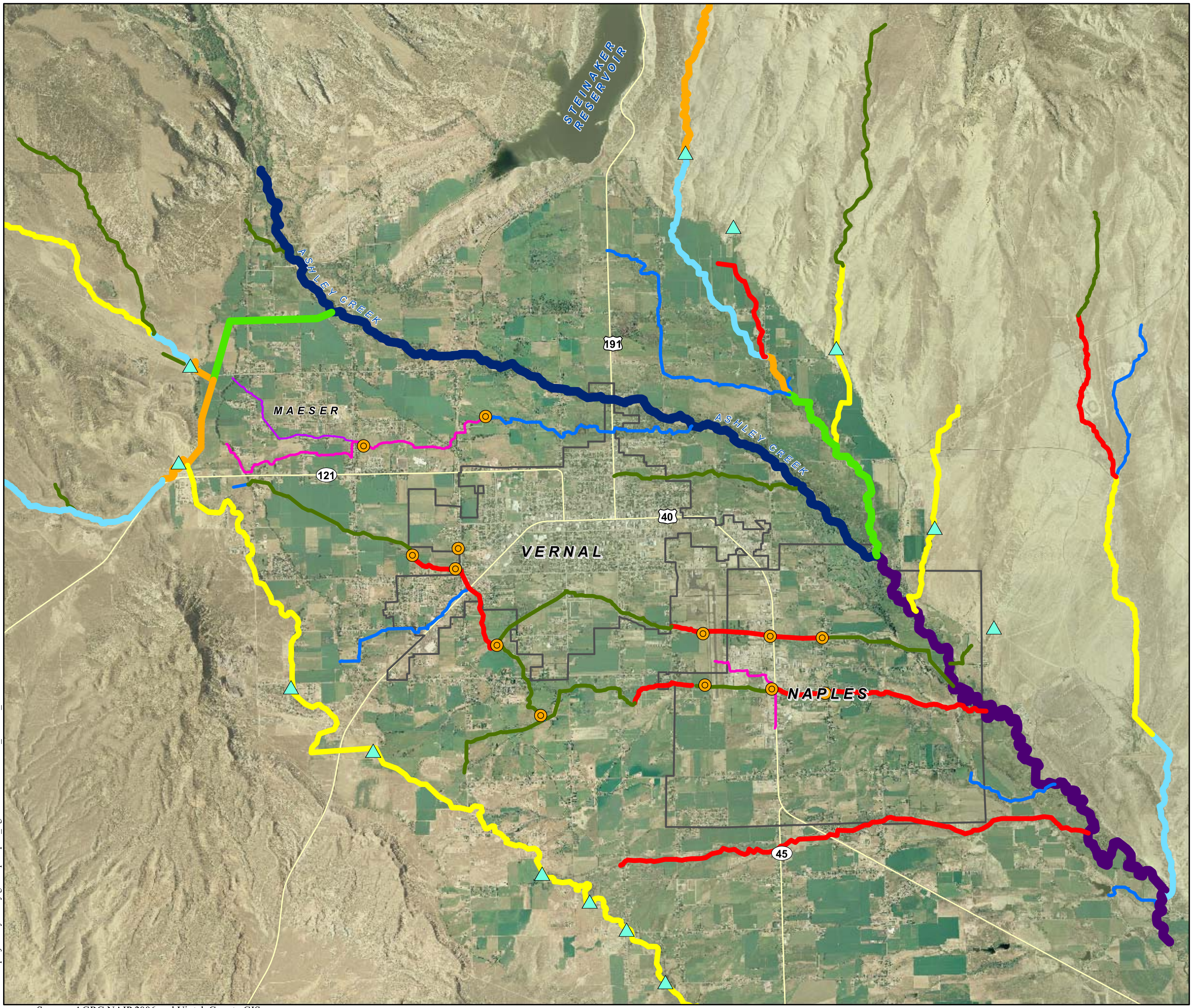
Epic Engineering

3341 SOUTH 4000 WEST
WEST VALLEY CITY, UTAH 84120
(801) 955-5605

50 EAST 100 SOUTH
HEBER CITY, UTAH 84032
(435) 654-6600

FIGURE 7-6
ESTIMATED FUTURE MODIFIED 50-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

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Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N E S W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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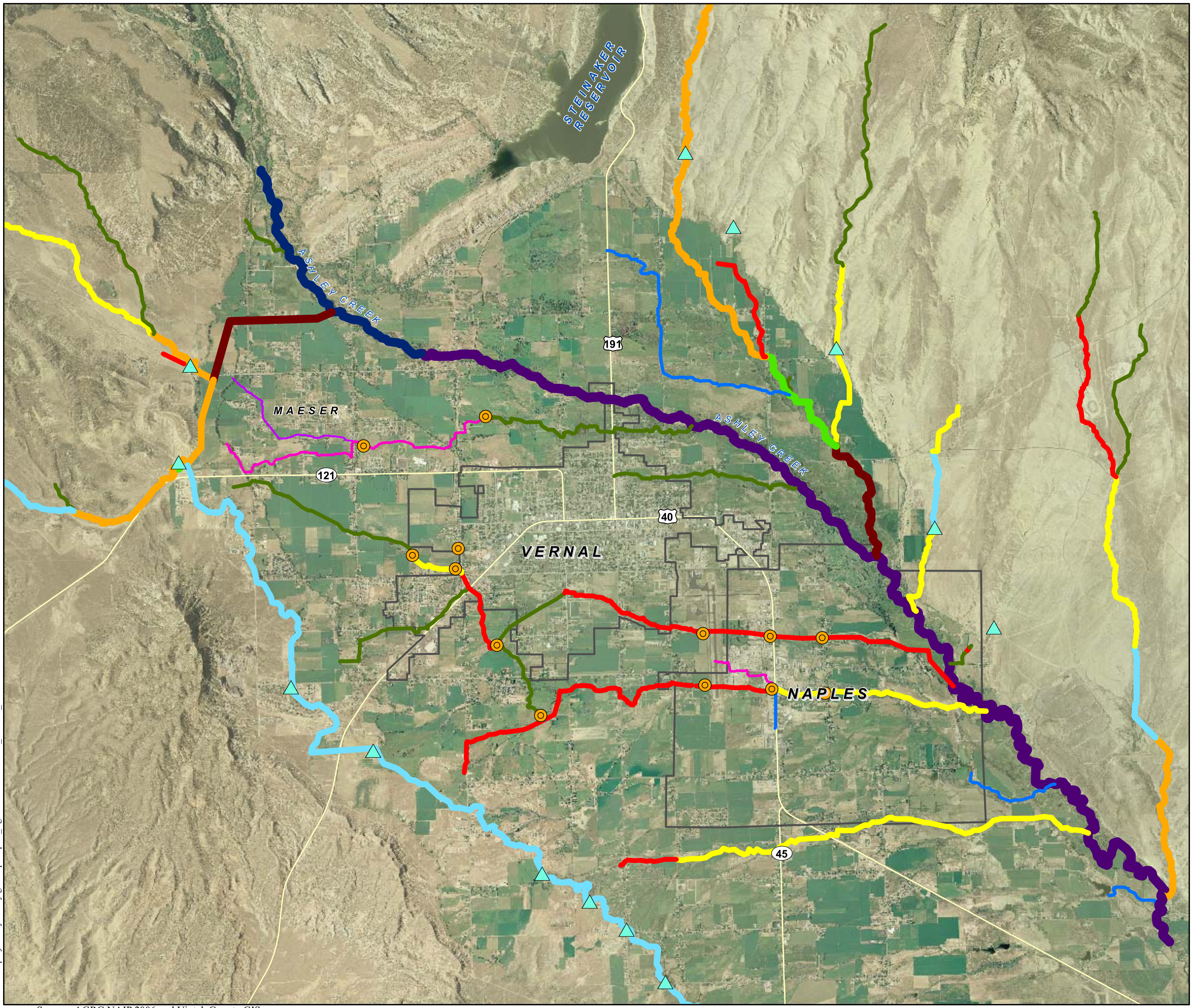

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 7-7
ESTIMATED FUTURE MODIFIED 100-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

N E S W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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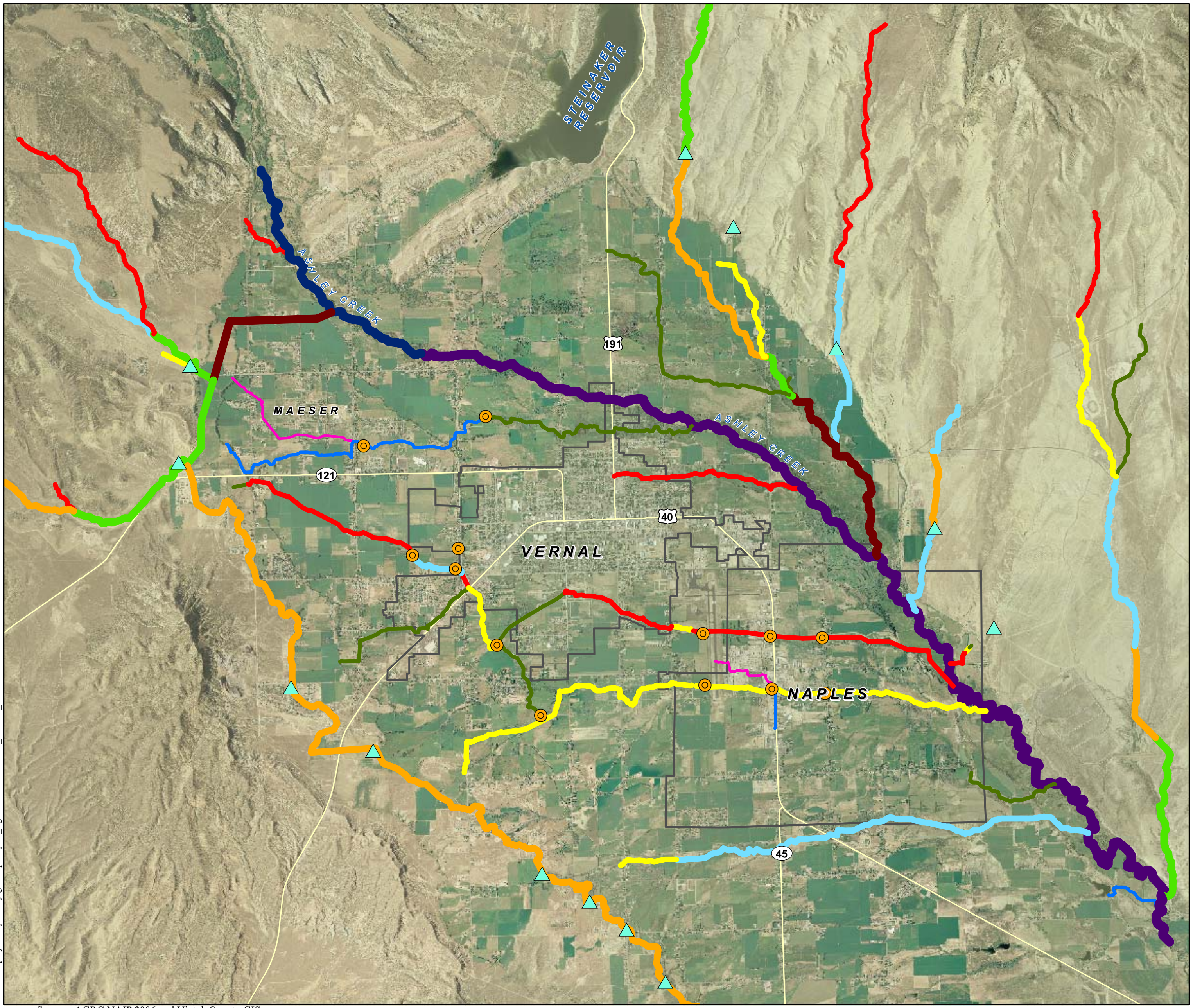

Source: AGRC NAIP 2006 and Uintah County GIS.

FIGURE 7-8
ESTIMATED FUTURE MODIFIED 500-YEAR CHANNEL FLOWS
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Flow (cfs)

- < 25
- 26 - 50
- 51 - 100
- 101 - 200
- 201 - 300
- 301 - 500
- 501 - 700
- 701 - 1,000
- 1,001 - 1,500
- 1,501 - 3,000
- 3,001 - 6,000
- > 6,000

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1 INCH = 5,000 FEET

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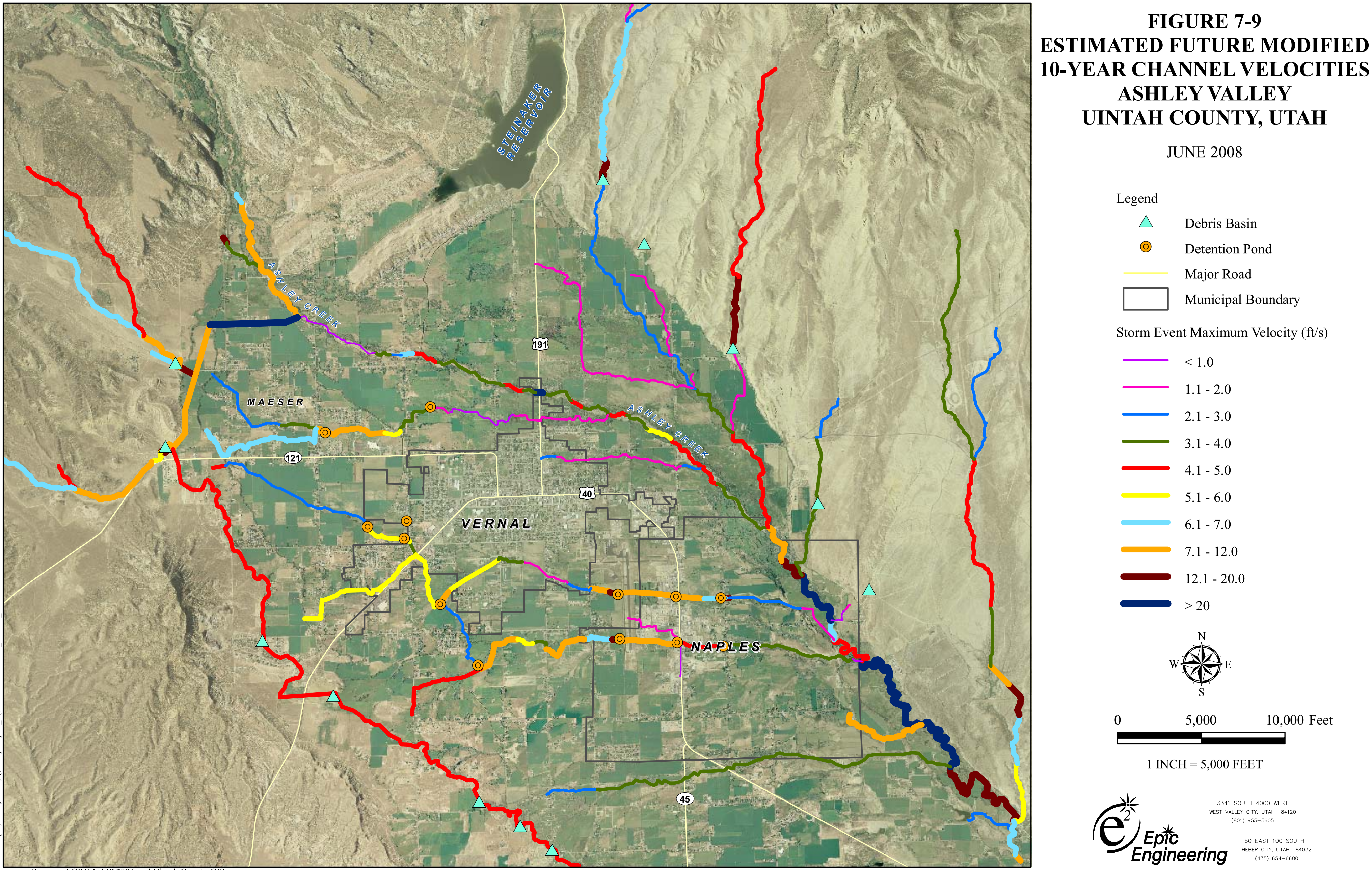
7.6 SIMULATED PEAK VELOCITIES WITH IMPROVEMENTS

As with the existing and future model output the peak velocities for each channel were once again computed with the major improvements integrated. Note that through the majority of the Valley the peak storm events, especially for the 10- year and 25- year storms the peak velocities are greatly reduced over existing conditions. The reduced velocities should improve channel stability. The improved channel stability will help maintain the current channel alignment in the future to aid their preservation. The peak channel velocities with the improvements are shown in Figure 7-9, Figure 7-10, Figure 7-11, Figure 7-12, and Figure 7-13, respectively.

FIGURE 7-9
ESTIMATED FUTURE MODIFIED 10-YEAR CHANNEL VELOCITIES ASHLEY VALLEY UINTAH COUNTY, UTAH

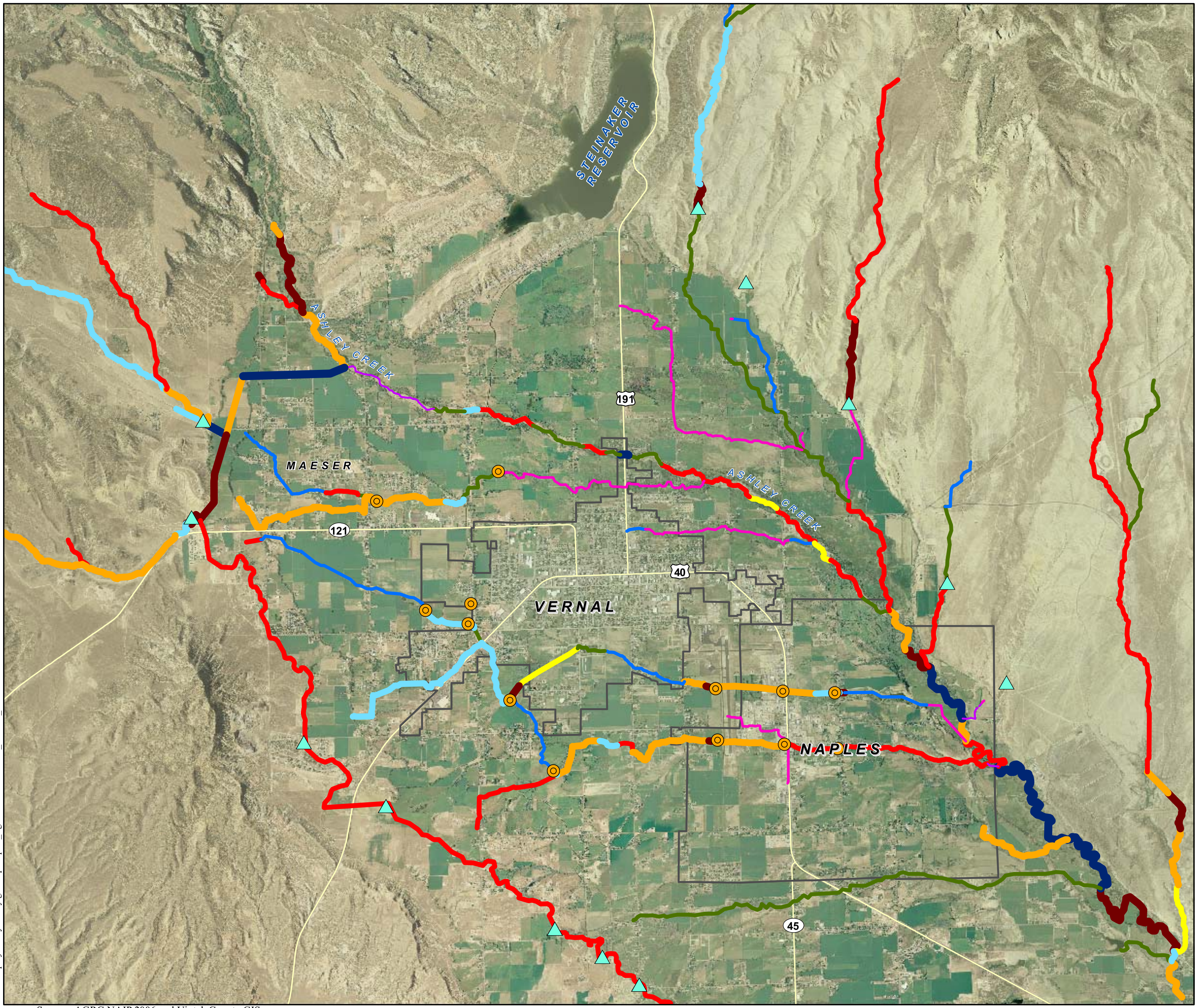
JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

FIGURE 7-10
ESTIMATED FUTURE MODIFIED 25-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008



Source: AGRC NAIP 2006 and Uintah County GIS.

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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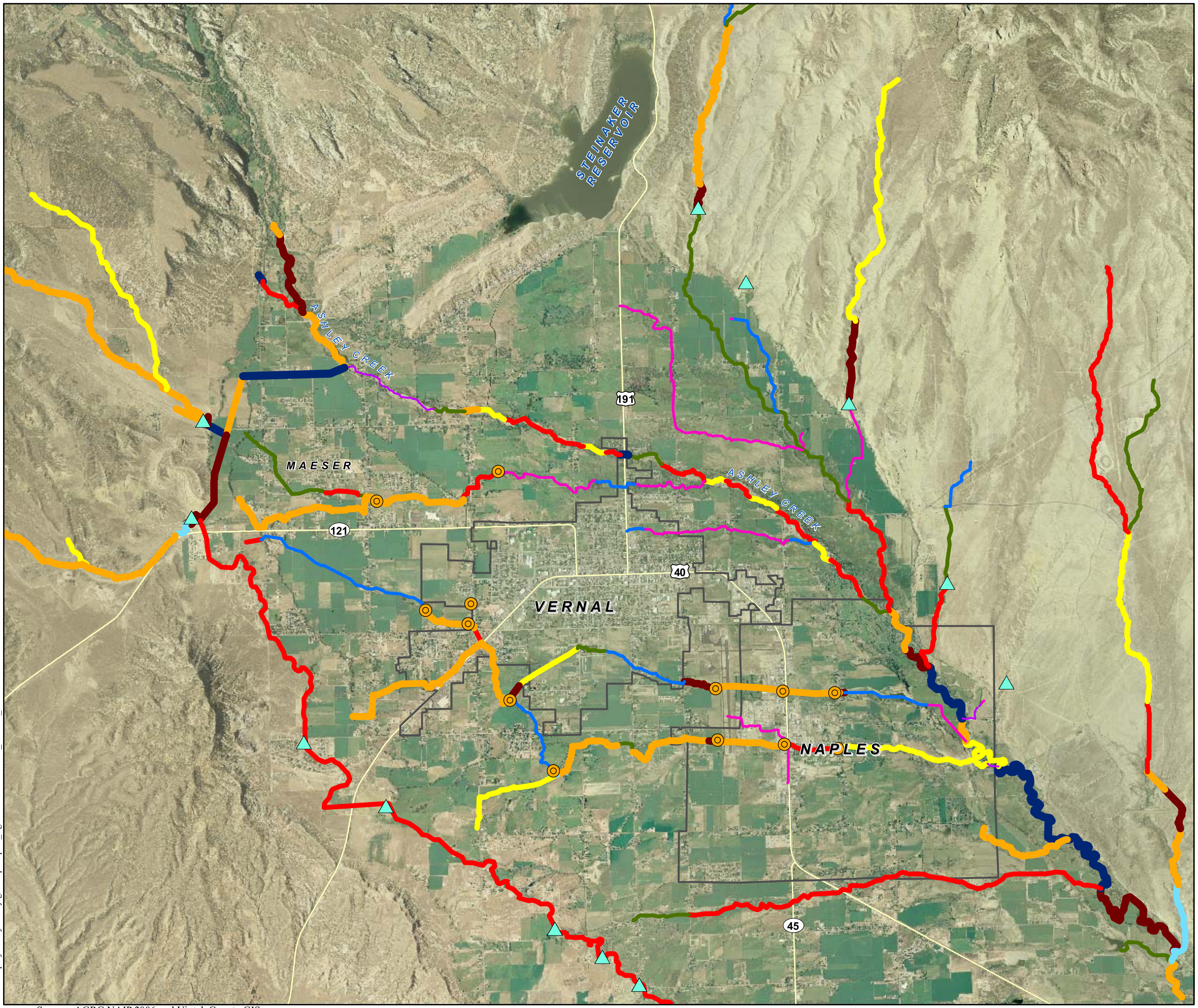

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 7-11
ESTIMATED FUTURE MODIFIED 50-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N W E S

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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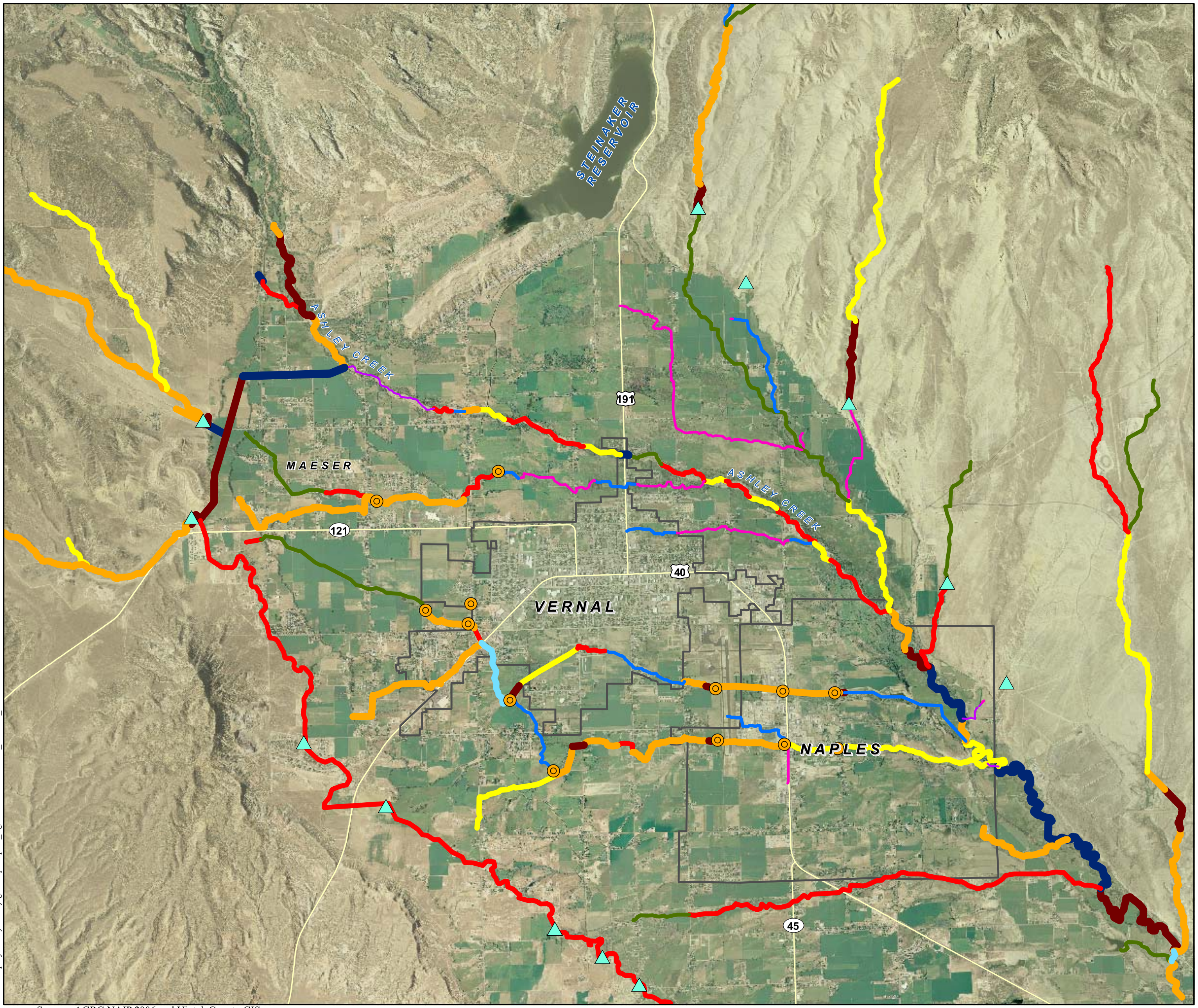

Source: AGRC NAIP 2006 and Uintah County GIS.

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FIGURE 7-12
ESTIMATED FUTURE MODIFIED 100-YEAR CHANNEL VELOCITIES
ASHLEY VALLEY
UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N S E W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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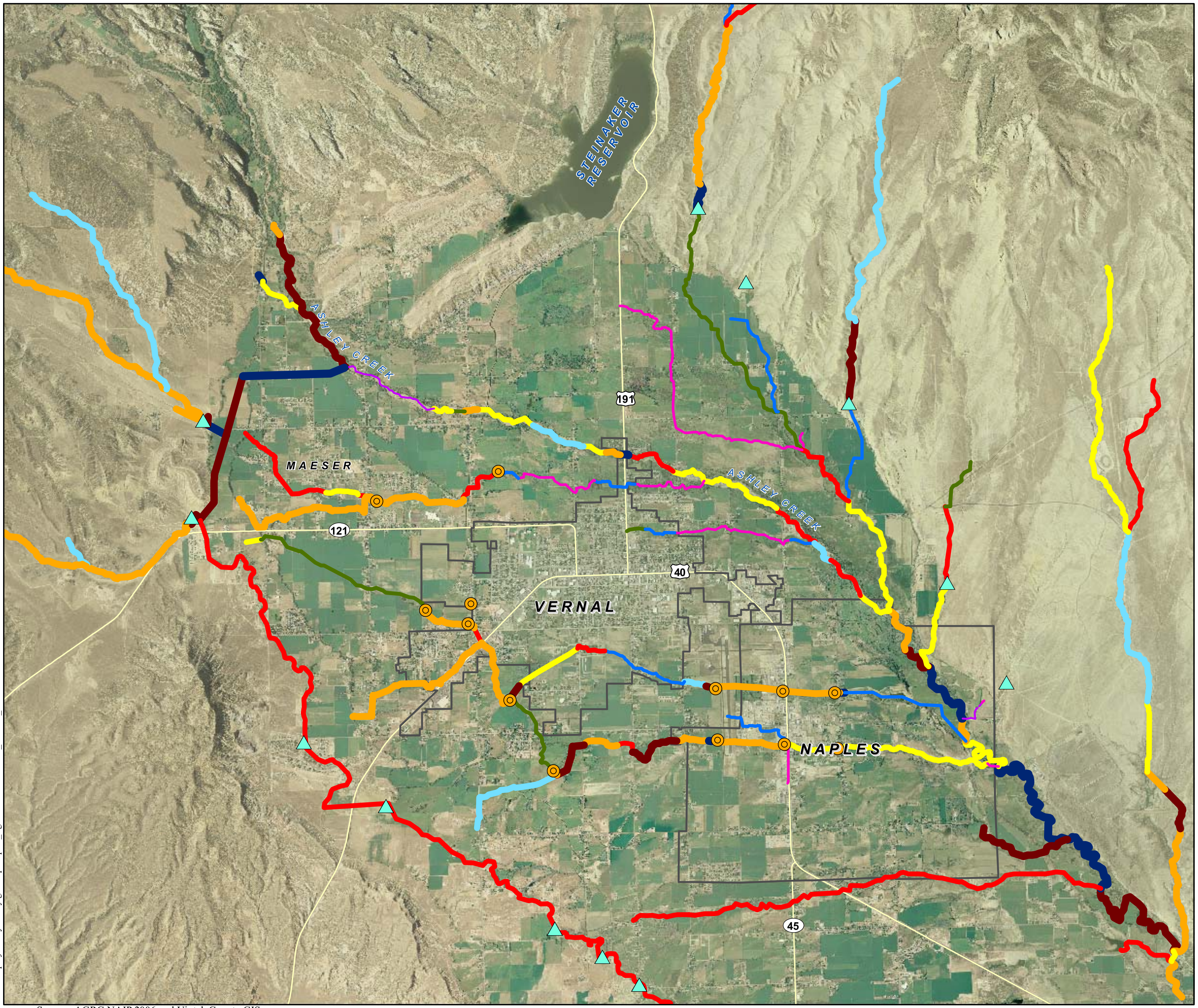

FIGURE 7-13
ESTIMATED FUTURE MODIFIED 500-YEAR CHANNEL VELOCITIES ASHLEY VALLEY UINTAH COUNTY, UTAH

JUNE 2008

Legend

- Debris Basin
- Detention Pond
- Major Road
- Municipal Boundary

Storm Event Maximum Velocity (ft/s)

- < 1.0
- 1.1 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 12.0
- 12.1 - 20.0
- > 20

N E S W

0 5,000 10,000 Feet

1 INCH = 5,000 FEET

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Source: AGRC NAIP 2006 and Uintah County GIS.

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Chapter 8 PERMITTING REQUIREMENTS AND FUTURE REGULATIONS

8.1 STORM WATER PERMITTING

Storm water permitting dates back to 1972 when the federal Water Pollution Control Act (also known as the Clean Water Act [CWA]) was amended to provide that the discharge of pollutants to waters of the United States from any point source is unlawful unless the discharge is in compliance with a National Pollutant Discharge Elimination System (NPDES) permit. The 1987 amendments to the CWA added section 402(p), which established a framework for regulating storm water discharges under the NPDES Program. Subsequently, in 1990, the U.S. Environmental Protection Agency (EPA) promulgated regulations for permitting storm water discharges from industrial sites (including construction sites that disturb five acres or more) and from Municipal Separate Storm Sewer Systems (MS4s) serving a population of 100,000 people or more. These regulations, known as the Phase I regulations, require operators of medium and large MS4s to obtain storm water permits from the EPA or State, where equivalent State regulations are adopted. On December 8, 1999, the EPA promulgated regulations, known as Phase II, requiring similar permits for storm water discharges from Small MS4s and from construction sites disturbing between one and five acres of land.

An “MS4” is a conveyance or system of conveyances (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, man-made channels, or storm drains): (i) designed or used for collecting or conveying storm water; (ii) which is not a combined sewer; and (iii) which is not part of a Publicly Owned Treatment Works (POTW). [See Title 40, Code of Federal Regulations (40 CFR) §122.26(b)(8).]

A “Small MS4” is an MS4 that is not permitted under the municipal Phase I regulations, and which is “owned or operated by the United States, a State, City, Town, borough, County, Parish, District, association, or other public body (created by or pursuant to State law) having jurisdiction over disposal of sewage, industrial wastes, storm water, or other wastes, including special districts under State law such as a sewer district, flood control district or drainage district, or similar entity…” (40 CFR §122.26(b)(16)).

The State of Utah has adopted the NPDES permitting requirements through the ratification of the Utah Water Quality Act in 1994. This act created the Utah Pollutant Discharge Elimination System (UPDES) as an equivalent to the NPDES. The UPDES is operated by the State Division of Water Quality (DWQ) of the Department of Environmental Quality.

Federal and State regulations allow two permitting options for storm water discharges (individual permits and general permits). The State has elected to adopt a statewide general permit for Small MS4s in order to efficiently regulate numerous storm water discharges under a single permit. When governmental entities within the Valley conduct improvement projects involving storm drains and/or surface improvements that have the potential to affect State receiving waters, a Notice of Intent (NOI) to comply with the terms of this general permit should be submitted.

Activities involving storm drains within the Valley should fall under one of two types of permits: a construction permit or a Small Municipal Separate Storm Sewer Systems (MS4) General Permit.

8.1.1 CONSTRUCTION PERMIT

A construction permit must be secured prior to breaking ground on construction that will disturb more than one acre of land. The UPDES General Permit for Storm Water Discharges Associated with Construction Activity (General Construction Permit) requires all dischargers where construction activity disturbs one acre or more to:

1. Develop and implement a Storm Water Pollution Prevention Plan (SWPPP) which specifies Best Management Practices (BMPs) that will prevent all construction pollutants from contacting storm water and with the intent of keeping all products of erosion from moving off-site into receiving waters.
2. Eliminate or reduce non-storm water discharges to storm sewer systems and other waters of the U.S.
3. Develop and implement a monitoring program.
4. Perform inspections of all BMPs.

8.1.2 SMALL MS4 GENERAL PERMIT

According to the General Construction Permit, the SWPPP shall emphasize the use of appropriately selected, correctly installed and maintained pollution reduction BMPs. All dischargers are required to prepare and implement a SWPPP prior to disturbing a site, and the SWPPP shall remain on the site at all times and shall be implemented to protect water quality at all times throughout the life of the project.

The SWPPP has two major objectives: (1) to help identify the potential sources of sediment and other pollutants that affect the quality of storm water discharges and (2) to describe and ensure the implementation of BMPs to reduce or eliminate sediment and other pollutants from storm water, as well as non-storm water, discharges.

The SWPPP shall include BMPs which address source control and, if necessary, shall also include BMPs which address pollutant control.

The following elements are required in a SWPPP:

1. Site description addressing the elements and characteristics specific to the site;
2. Descriptions of BMPs for erosion and sediment controls;
3. BMPs for construction waste handling and disposal;
4. Implementation of approved local plans;
5. Proposed post-construction controls, including description of local post-construction erosion and sediment control requirements;
6. Non-storm water management.

8.1.3 MONITORING PROGRAM

The General Construction Permit requires development and implementation of a monitoring program. Dischargers are required to inspect the construction site prior to anticipated storm events and after actual storm events. During extended storm events, inspections must be made during each

24-hour period. Inspections will identify areas contributing to a storm water discharge and evaluate whether measures to reduce pollutant loadings identified in the SWPPP are adequate and properly installed and functioning in accordance with the terms of the General Permit. In addition, inspections will determine whether additional control practices or corrective maintenance activities are needed.

8.2 Small MS4 General Permit

Upon completion of development, or at an appropriate time as determined through communications with State DWQ staff, the local governing body will likely require a municipal permit. Small MS4s may be identified through the following methods:

1. Automatically designated by U.S. EPA pursuant to 40 CFR section 122.32(a)(1) because it is located within an urbanized area defined by the Bureau of the Census.
2. Traditional Small MS4s that serve Cities, Counties, and unincorporated areas that are designated by DWQ after consideration of the following factors:
    a. High population density – an area with greater than 1,000 residents per square mile, potentially created by a non-residential population, such as tourists or commuters.
    b. High growth or growth potential – Growth of more than 25 percent between 1990 and 2000, or anticipated growth of more than 25 percent over a 10-year period ending prior to the end of the first permit term.
    c. Significant contributor of pollutants to an interconnected permitted MS4.
    d. Discharge to sensitive water bodies.
    e. Significant contributor of pollutants to waters of the U.S.

Based on the above criteria, portions of the Valley are likely subject to MS4 permit regulations. As development occurs, additional portions of the Valley will also be expected to conform. It is recommended that all governing bodies adopt these criteria in the near future regardless of their current designation under the MS4 discharge permit.

The MS4 permit requires dischargers to develop and implement a Storm Water Management Program (SWMP) that describes the best management practices, measurable goals, and time schedules of implementation as well as assigns responsibility of each task. Also, as required by the Small MS4 General Permit, the SWMP must be available for public review and must be approved by the State prior to permit coverage commencing. This information is provided to facilitate the process of an MS4 obtaining Small MS4 General Permit coverage. The Storm Water Management Plan is completed as a separate document and can be obtained from the City by the public for review.

8.2.1 Storm Water Management Plan

The General Permit requires permittees to develop and implement a SWMP designed to reduce the discharge of pollutants through their MS4s to the Maximum Extent Practicable (MEP). The General Permit requires the SWMP to be fully implemented by the end of the permit term (or five years after designation for those designated subsequent to General Permit adoption). Once DWQ staff has reviewed a SWMP and, in light of meeting the MEP standard, recommends approval of coverage, the public may review the SWMP and request a public hearing if necessary. The SWMP will be made available for public review for a minimum of 60 days.

Federal and State regulations require operators of MS4s to develop a five-year work plan with associated performance measures and budgeting to address six Minimum Control Measures (MCMs). The MCMs to be addressed include:

1. Public Outreach and Education;
2. Public Participation and Involvement;
3. Illicit Discharge Elimination;
4. Construction Site BMPs Over One Acre;
5. Post-Construction BMPs; and
6. Municipal Activities.

For each MCM, measurable BMPs should be developed, and a schedule and budget provided for completion of the BMP.

8.2.2 Storm Water Pollution Prevention Regulation

To ensure BMPs are followed, each entity should implement a storm water pollution prevention ordinance. The ordinance should describe the BMPs described in this section and as well as other relevant BMPs as the entity deems necessary or prudent. The ordinances should be worded such that most of the physical BMPs for new construction are a requirement of approval to ensure they will be properly constructed and maintained.

8.3 Best Management Practices

The best management practices for the following types of potential contamination sources are described below. Additional detail on each of the proposed BMPs can be found in the Appendix of this report.

8.3.1 New Construction

All new construction projects in excess of one acre or those projects which pose a potential risk to storm water pollutants should be required to submit a Storm Water Pollution Prevention Plan (SWPPP). At a minimum, the SWPPP should include the following components:

1. Site description addressing the elements and characteristics specific to the site;
2. Descriptions of BMPs for erosion and sediment controls;
3. BMPs for construction waste handling and disposal;
4. Implementation of approved local plans;
5. Proposed post-construction controls, including description of local post-construction erosion and sediment control requirements;
6. Non-storm water management.

Examples of BMPs that may be part of a SWPPP include:

1) Straw bales or gravel bags around inlets and along new ditches.
2) Detention or settling ponds prior to discharge off-site.
3) Phased construction to minimize exposed sediment.

It is recommended that all new development and large construction projects be required to submit and follow a SWPPP plan prior to commencing work.

8.3.2 Existing Development

Controlling the quality of storm water runoff from existing development generally requires public involvement and education. Informing the community of the importance of clean water and ways to avoid or minimize behaviors that typically cause polluted storm water is imperative to maintaining reasonably clean runoff from existing developments. Three typical sources of pollution are Oil and grease, fertilizer, and trash. Oil and grease as well as fertilizer often affect water quality throughout the region. Through education and proper management these contaminates can be minimized and the water quality of the region preserved. Trash can also affect water quality but more often than not it clogs key storm culverts and grates and diminishes capacity. Education and street cleaning to prevent trash from entering the storm water system can prevent or minimize flooding during major rainfall events.

Since not all pollution from existing developments can be eliminated through public education it is also important to provide treatment of storm water through detention basin, screening manholes, or oil water separators throughout existing communities whenever practical.

8.3.3 Roadway Maintenance

Roadways in general have high potential to contribute large amount of pollutants into storm water for a variety of reasons, including:

1) Roadways cover a large portion of the land, and often are constructed through sensitive areas.
2) Curbs and gutters catch and store debris, fuel, oil and grease from automobiles.
3) Winter operations introduce salts and sands throughout the roadway network.
4) Pavement design creates high runoff volumes, while increasing contact time between storm water and contaminates.

As regions such as the Valley continue to develop, additional roadways and the associated storm water pollutant potential will increase. The pollutants generated from roadways can be mitigated by implementing best management practices. There are a series of best management practices to reduce roadway generated polluted storm water.

The following BMPs are recommended for use within the Valley:

- ***Develop roadway salting and sanding protocols to minimize the use of salt and sand on the roadways throughout the winter. Consider using alternative de-icing formulas throughout the Valley and especially near environmentally sensitive areas.***
- ***Site future O&M facilities, such as sand storage, away from natural water ways and storm channels.***
- ***Store winter salt and sand piles under cover to prevent contact with wind and precipitation. Construct evaporation ponds for storm water in and around these sites where possible.***
- ***Divert all existing storm water and require that future storm water runoff from roadways be treated prior to discharge into natural channels. Treatment may include grassy swales, settling ponds, and oil water separators.***

Chapter 9 REFERENCES

- SewerGEMs 8.1, Bentley Systems, Inc., 2007.
- Magna Water Company Water and Sewer Master Plan 1999, Hansen, Allen, Luce Inc. and Epic Engineering, P.C., August 1999
- Syracuse City Storm Drain Master Plan, Epic Engineering P.C., June 2007
- Introduction to Environmental Engineering third edition, McGraw-Hill ISBN 0-07-015918-1, 1998
- Summit County Storm Water Pollution & Erosion Control Ordinance 381-A, Adopted December 2004
- Urban Storm Water Management BMP Performance Monitoring, US EPA & American Society of Civil Engineers EPA-821-B-02-001, April 2002
- Ashley Creek Stabilization / Restoration Report, Franson Noble & Associates, Inc and Mussetter Engineering Inc. May 2000.
- Geotechnical / Geological Feasibility Evaluation Proposed Spring Creek Reservoir, Kleinfelder, Inc. May 2003.