

**PART IV of Attachment A**  
**INTERIOR DRAINAGE HYDROLOGY**

## PART IV – INTERIOR DRAINAGE HYDROLOGY

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## 1. Introduction

**1.1 Study Area.** The Truckee Meadows Flood Damage Reduction and Ecosystem Restoration Project has analyzed many alternatives over the years for both downtown Reno, the Truckee Meadows Reach, and the Lower River Reach. Recently, some of these alternatives are no longer being considered. This section, Part IV of Attachment A, describes the analysis that was performed to determine existing condition interior drainage hydrology for various tributary watersheds in the study area using rainfall-runoff models. This interior drainage analysis has two major purposes: (1) provide various frequency hydrographs for mapping residual floodplains caused by storms centered over tributaries or urban areas (this is flooding that is NOT the direct result of mainstem Truckee River overbank flows) (2) provide hydrology to analyze impacts of proposed alternatives to interior drainage floodplains. Some alternatives such as Huffaker Dam on Steamboat Creek are no longer being considered, yet this interior drainage study report will include a description of the hydrologic analysis for these locations since one or more of these alternatives might be considered again in the future. The main report, rather than this appendix, should be referenced for the latest information on alternatives.

Areas studied in this interior drainage report include North Truckee Drain, Steamboat Creek, Hidden Valley, south side of Truckee Meadows near the Airport, downtown Reno on south side of Truckee River, downtown Reno on north side of the Truckee River, and Long Valley Creek. This report also includes an estimated Probable Maximum Flood that was developed to size the spillway for the proposed Huffaker Dam.

As a reminder, the Truckee Meadows Flood Damage Reduction and Ecosystem Restoration Project is divided into three portions: (1) the reach of the Truckee River between Booth Street and U.S. Highway 395 (**Downtown Reno Reach**); (2) the reach of the Truckee River from Highway 395 to Vista, along with the tributaries of Boynton Slough, Steamboat Creek, and North Truckee Drain (**Truckee Meadows Reach**); and (3) the reach of the Truckee River between Vista and Pyramid Lake (**Lower River Reach**).

**1.2 Methodology.** All tributary watersheds to the Truckee River were analyzed with a rainfall-runoff model, either HEC-1 or HEC-HMS. On Steamboat Creek, which covers a couple of hundred square miles, the Corps utilized a 3-day general rain-on-snow event. A separate SNOW model was used to simulate the impact of rain melting the snow, precipitation falling as snow, or the snowpack absorbing and holding the rainfall. This is the only tributary high enough in elevation to require a snowpack analysis. A 3-day general rain was also used to model the 1% chance event on lower North Truckee Drain channel, a watershed of over 60 square miles that includes Spanish Springs Dam. For smaller areas like downtown Reno, both a general rain and a 3-hour cloudburst storm were modeled to see which dominated the flooding problem. Subsequent floodplain mapping performed by Hydraulic Design Section determined that general rain events determine the maximum amount of water that ponds against structures like levees because they have more volume than the cloudburst events. Cloudburst events have a flashy instantaneous peak but not much volume. For most locations the general rain storm was a 72-hour long event, except for downtown Reno and Long Valley Creek watershed, which used a 24-hour storm. Design storms were generally balanced to NOAA 14 Precipitation Frequency estimates published by the National Weather Service in 2003. For Steamboat Creek and North

Truckee Drain, soil loss rates were based on the Maricopa County, AZ Design Manual Green and Ampt Loss Rate Method (Ref. 22). For downtown Reno on the north side of the Truckee River, SCS curve numbers were used. For Long Valley Creek, initial/constant loss rates were utilized. The Corps initially used Curve Numbers for Steamboat Creek and North Truckee Drain; however, the method was later revised to Green and Ampt on the recommendation of an External Peer Review by David Ford Consulting Engineers, Inc. (Ref. 35). The External Peer Review was part of a new policy mandated by the Corps HQ in 2007, which required the team to consider certain technical portions of the study for outside review. The peer review for the North Truckee Drain hydrology was performed by Dr. Joe Devries of David Ford Consulting Engineers. Joe Devries is a former professor in the Civil Engineering Department at U.C. Davis. He also has many years experience in hydrology and hydraulics work, including a Probable Maximum Flood Analysis for Martis Creek Dam, which is in the Truckee River watershed. The result of the peer review indicated that the use of SCS Curve Numbers (CNs) should only be for storms with a duration of 24 hours. It was determined that is not appropriate to use CNs for durations longer or shorter than 24 hours. This conclusion was based in part on correspondence between Dr. Devries and Dr. Richard Hawkins of the University of Arizona (knowledgeable in the SCS methodology for drainage design). Since the Corps used a 72-hour storm for North Truckee Drain and Steamboat Creek watersheds, the use of CNs was deemed inappropriate for these watersheds. Instead, Dr. Devries recommended using the Green and Ampt loss rate method and the Corps followed this advice. SCS TR-60 does provide for adjustments to CNs for a 10-day storm. A linear interpolation for a 3-day event was tried but it did not prevent the loss rates from approaching zero inches on the 3<sup>rd</sup> day of the storm. As such, CNs were not used.

**1.3 Concurrent Flows Between Truckee River and Tributaries.** Whenever interior drainage is being modeled, there often needs to be an assumption of the concurrent tailwater condition in the mainstem of the river. The following rules were applied to hydraulic modeling of major tributaries or interior drainage areas next to the river.

**1.3.1 Interior Drainage Areas Behind Levees or Floodwalls.** For interior drainage areas behind levees or floodwalls in downtown Reno, the Meadows Reach, and Long Valley Creek, the concurrent event in the Truckee River was assumed to be approximately half the return period of the event occurring in the interior area. The timing assumption is that the interior area peaks at the same time the mainstem river peaks. The timing assumption is believed to be a little conservative.

<u>% Chance Exceedence Interior Area</u>	<u>Concurrent % Chance Exceedence on Truckee River</u>
10%	20%
2.0%	5.0%
1.0%	2.0%

**1.3.2 North Truckee Drain Channel.** The proposed North Truckee Drain channel design required the hydraulic modelers to look at two separate scenarios to see which was more critical. One scenario has a 1% event on the Truckee River with a concurrent event on the North Truckee Drain (i.e. 600 cfs on North Truckee Drain at the time of peak on the mainstem). The

second scenario is a 1% event on North Truckee Drain with a concurrent 2% chance event on the Truckee River mainstem (peak on peak). For the latter, the concurrent flow was based on the historic 1986 flood. In 1986, the North Truckee Drain incurred its highest recorded peak flow (estimated at 1850 cfs at Shadow Lane). The recorded peak flow at the Truckee River at Vista during that flood was a little rarer than a 2% chance event (1.54%).

Hydraulic Design has found that the 1% event on the Truckee River with a concurrent event on North Truckee Drain creates the highest stages on the lower end of the North Truckee Drain (all the way upstream to Prater Way). This is due to a high tailwater created by the Truckee River. This scenario is also critical for design of the proposed North Truckee Drain box culverts under the 80 freeway. The HEC-RAS model for this scenario includes both tributary and mainstem hydrographs with timing built into them so that the peak flow at the Vista gage equaled a 1% event. The concurrent flow on North Truckee Drain was approximately 600 cfs when the Truckee River reached its maximum stage.

**1.3.3 Steamboat Creek Channel.** Levees are proposed on Steamboat Creek to contain the backwater from the Truckee River. The Truckee River stages in the Meadows Reach will increase under with-project conditions. At one time, the locally preferred plan was to develop a project that contained the 117-year flood. This meant the levees on Steamboat Creek needed to be built all the way upstream to the Hidden Valley community. Because these levees could cause interior drainage problems for this community, an interior drainage analysis was performed on the Hidden Valley watershed. The analysis is described in Section 5.

The concurrent flow on the Truckee River when Steamboat Creek incurs an n-year flood was determined to be approximately half the return period of the event on Steamboat Creek. For example, during a 1% chance event (100-year) on Steamboat Creek, the concurrent event on the Truckee River is a 2% chance event (50-year). The timing is arranged in HEC-RAS such that Steamboat Creek and the Truckee River peak at the same time.

When simulating an n-year flood on the mainstem Truckee River, the concurrent Steamboat Creek and North Truckee Drain runoff, when added to the Truckee River flow creates a 1% chance peak flow at Vista under existing (without-project) conditions in the HEC-RAS model.

**1.3.4 Town of Lockwood and Long Valley Creek Channel.** The Corps must ensure that its with-project design does not induce flooding problems on the town of Lockwood near the mouth of Long Valley Creek. The Corps hydraulically modeled this area under two scenarios. The first scenario is a 1% chance event on the Truckee River with concurrent runoff in Long Valley Creek. The concurrent runoff had a built-in timing so that the peak flow downstream at the Tracy gage equaled a 1% peak. The concurrent runoff on Long Valley had a peak under 3,000 cfs for this scenario.

The other scenario that was modeled was a 1% chance event on Long Valley Creek with a 2% chance event on the Truckee River. The timing of the Long Valley hydrograph was adjusted in relation to the mainstem so that combined flows of the two did not exceed 21,500 cfs or the 1% chance peak flow at the Tracy gage on the Truckee River.

**1.3.5 Justification for Assumptions.** The January 1997 flood was slightly larger than a 1% chance event. Steamboat Creek, North Truckee Drain, and Long Valley Creek tributaries had smaller flows than the 1% chance event in their channels during this flood. The table below quantifies the percent chance event flows on tributaries and the Truckee River at Vista for three flood events.

Table 1. Comparison of Truckee River and Tributary Runoff for Historic Floods

	1997 Flood	1986 Flood	2005 Flood
Truckee R. at Vista (% chance event)	~ 0.95% (105-yr)	~ 1.54% (65-yr)	~ 2.2% (45-yr)
Steamboat Crk @ Steamboat gage (% chance event)	~ 4%	~ 2%	~ 2%
North Truckee Drain (% chance event)	< 10%	*Estimate > than 2%	< 10%
Long Valley Creek (% chance event)	< 10%	Between 5% and 2%	< 20%

\* In 1986, Spanish Springs Dam and other smaller detention basins within the watershed did not exist, so comparison to existing conditions rainfall runoff modeling results is difficult.

The question that needed answering for the interior drainage analysis was which frequency event should be used on the mainstem Truckee River when a tributary or interior drainage area is incurring an n-year event. For Steamboat Creek, the data shows that the Truckee River and Steamboat Creek can both have approximately 2% chance events at the same time (1986 and 2005). Regression plots of the Steamboat Creek versus Truckee River at Vista flows show a wide scatter. The results are not conclusive. Some floods have bigger return periods on the tributaries than the mainstem and vice versa. The historic data does not appear to justify having a 1% chance event on the mainstem and on the tributaries of Steamboat Creek and North Truckee Drain, all at the same time. To simplify the modeling of 1% chance events on Steamboat Creek and North Truckee Drain, it was decided to use the next lowest return period available in the dataset, a 2% chance event on the mainstem. The 1986 flood on North Truckee Drain also validates this decision. A similar decision was made for Long Valley Creek and interior drainage areas. The decision was to use the next lowest return period in the data set for the mainstem (approximately  $n\text{-year} \div 2$  when a tributary or interior drainage area incurs an n-year flood).

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## **2.0 North Truckee Drain**

**2.1 Study Area.** The geographic area covered in this analysis is the north side of the Truckee River between the downtown Reno streamgage and the Vista streamgage. A map of this area is shown in Chart 2-1. The farthest north portions of this area include Sun Valley (northwest side) and Spanish Springs Valley (northeast side). Sun Valley subbasins include #500 through #520. Spanish Springs Valley is a natural, bowl-shaped watershed to the north of downtown Sparks. Subbasins 1 – 29B (upper subbasins with thin black and yellow boundaries on Chart 2-1) constitute the Spanish Springs HEC-1 model. The whole Spanish Springs watershed is approximately 77 square miles in area, but Boneyard Flat, an approximately 11 square mile sink, does not contribute runoff. The remainder of the Spanish Springs HEC-1 model covers 66 square miles. Spanish Springs Dam, near the outlet of Spanish Springs Valley, controls 60 square miles of the upper watershed. The dam was designed by the locals to contain a 1% chance event. The Corps adopted the Spanish Springs HEC-1 model from the City of Sparks and modified it for its own use.

The runoff from the Spanish Springs HEC-1 model becomes input to the Corps' North Truckee Drain model, which covers 28 square miles of the lower portion of the watershed that includes the downtown area of the City of Sparks. Runoff within the city generally flows south or southeast and collects behind east-west running transportation barriers like McCarren Blvd, Highway 80, and the Southern Pacific Railroad before being conveyed to the Truckee River via surface channels like the North Truckee Drain. North Truckee Drain is an improved channel that begins at about the latitude of Shadow Lane in the foothills below Spanish Springs Dam. The channel flows in a generally southerly direction through downtown Sparks City. It enters the Truckee River on the north side of the river via an open channel. The upper end of the channel has a natural, earthen bottom, whereas some lower reaches are lined with concrete. Contributing runoff on the eastern side of the watershed are the Pah Rah and D'Andrea watersheds (subbasins 700 and 710, respectively). The drainage from Sun Valley (subbasins 500, 505, and 510) flows into the Sun Valley Detention Basin at subbasin 511. Approximately 220 cfs of the outflow from the dam is diverted into an underground pipe that runs east along the route of McCarran Blvd until it reaches the North Truckee Drain in a six foot diameter drain. The remainder of the runoff from Sun Valley flows south into subbasin 511 as surface runoff. Significant portions of the runoff from 511, 520, 515, and 516 reach the Sparks Marina, which is a large sink with lots of storage capacity. During a flood which would fill the Marina, Highway 80 blocks the overflow from moving southward; therefore, the lake continues to fill, eventually flooding homes to the north. The northern boundary of subbasin 517 is Highway 80. This subbasin also contains People's Ditch, which parallels Highway 80 on the south side of the freeway, and flows east until it dumps into North Truckee Drain. During a flood event that fills the Marina, approximately 30 cfs is pumped from the Marina into People's Ditch. Runoff from subbasins 404, 405, 410, 415, and 420 flows eastward, parallel with the Truckee River; however, the runoff does not enter the North Truckee Drain. These subbasins, as well as 425, are interior drainage areas behind proposed levees where pumping stations might be required.

The climate of the Truckee Meadows area is generally arid, characterized by dry summers and long winters with somewhat little precipitation. Mean annual precipitation on the valley floor is only about 7.5 inches, with over half of that occurring between December and March.

Temperatures can vary from sub-zero on the eastern slopes of the Sierra Nevada Mountains to over 100° Fahrenheit in the valley. While snowfall may occur at any elevation in the basin, it typically falls above 5,000 feet. The winter snow pack begins to melt around April. Local engineers have said there is no discernable snowpack in the North Truckee Drain watershed including Spanish Springs Valley.

**2.2 Historical Floods.** The North Truckee Drain watershed, unlike Steamboat Creek, does not have high enough elevations to have a significant snowpack; therefore, snowmelt impacts do not need to be modeled in this watershed. The flood of record in this watershed was a general rain event that occurred in February 1986. Cloudburst storms can cause localized flooding; however, a general rain event is believed to be the dominant event which will cause a 1% chance exceedence peak on the lower North Truckee Drain. Two streamgages exist on North Truckee Drain. They include USGS gage #10348245 (N Truckee Drain at Spanish Springs Road nr Sparks) and #10348300 (N Truckee Drain at Kleppe Lane nr Sparks). The locations of the two gages are shown on Plate 1. The upper gage at Spanish Springs Road consists of only 10 years of broken record between 1986 and 2006. The lower gage consists of 12 years of broken record between 1993 and 2006. When the Truckee River incurs high stages, it causes a backwater effect on the Kleppe Lane gage, which makes its readings unreliable during large Truckee River floods. Below are the listed peak flows from the two USGS gages, #10348245 and #10348300.

Table 2-1. Peak Flows for USGS Gage #10348245, North Truckee Drain at Spanish Springs Rd. nr Sparks, NV, Drainage Area = 80.0 Square Miles

Date of Peak	Discharge in CFS
02/17/86	1850
07/14/93	34
09/28/94	31
03/11/95	27
01/02/97	15
08/01/02	43
06/20/03	20
06/26/04	23
06/24/05	115
12/31/05	286

Table 2-2. Peak Flows for USGS Gage #10348300, North Truckee Drain at Kleppe Ln nr Sparks, NV, Drainage Area Not Determined

Date of Peak	Discharge in CFS
01/20/93	106
05/19/94	153
05/02/95	455
05/18/96	670
08/07/99	120
08/03/00	228
04/20/01	92
08/01/02	282
12/16/02	120
02/25/04	220
06/24/05	312
02/28/06	561

During the flood of February 1986, there was a record amount of runoff from the tributaries in this area. Both North Truckee Drain and Steamboat Creek had their highest flows on record during this event. North Truckee Drain incurred an estimated peak of 1,850 cfs near Shadow Lane, while lower Steamboat Creek incurred an estimated peak of 5,600 cfs. The 1986 storm occurred prior to the existence of the North Spanish Springs Detention Basin and Spanish Springs Dam. The recorded flow at the Truckee River at Vista gage was 16,100 cfs. During the December 31<sup>st</sup>, 2005 storm event, North Truckee Drain flows were estimated to be about 300 cfs, while the Truckee River at Vista gage recorded a peak of 13,700 cfs.

**2.3 HEC-1 Models.** The City of Sparks has developed an HEC-1 model of Spanish Springs watershed over the years. The model covers the Spanish Springs Valley, which is a 66 square mile area. Residential developers have been mandated to mitigate for land-use change, which often takes the form of detention storage. Dozens of small detention facilities exist in the watershed. A number of A-E's firms have modified the model over the years as development occurs, so the area has been split into over one hundred subbasins. Near the end of the valley is Spanish Springs Dam, which was originally built by the local government to control a 1% chance event. The model follows some NRCS TR-55 guidelines including the use of SCS unit hydrographs for rainfall to runoff transform, SCS Curve Numbers for soil loss rates, and a 24-hour storm based on an unofficial, draft version of NOAA 14 Precipitation Frequency. Spanish Springs Dam is specified as having 1,503 acre-feet of flood control storage. It is a dry dam with no permanent storage. It has an ungated outlet and an uncontrolled spillway. Due to the complexity of modeling this urbanized valley, the Corps adopted the City of Sparks model and modified it for its own use. Modifications made by the Corps include: 1) creation of a 72-hour design storm using NOAA 14 Precipitation Frequency, 2) replacing SCS Curve Numbers with Green and Ampt loss rates, and 3) using a 5-minute computation interval. The Spanish Spring Model covers 66 square miles, which is the upper portion of the North Truckee Drain watershed.

A separate HEC-1 model for the lower North Truckee Drain watershed was created by the Corps. This model covers approximately 28 square miles of the lower watershed, some of which drains

directly to the Truckee River and does not enter North Truckee Drain. This model uses S-graphs developed in the 1980 Study (Ref. 5) for creating subbasin unit hydrographs, and Green and Ampt soil loss rates. Routing methods include Muskingum, Muskingum Cunge, and Modified Puls.

**2.4 Flood Control Development.** The largest flood control or detention storage structures include North Spanish Springs Detention Facility (sub #10B), Spanish Springs Dam (sub #26), Sun Valley Detention Basin (sub #510), Paradise Park (sub #605), the Sparks City Marina, and an in-line series of storage areas called the D'Andreas Dams (sub #710).

## **2.5 Runoff Model Development**

As mentioned earlier, the Corps adopted the Spanish Springs model that was developed by the City of Sparks. This model covers about 66 square miles. The model includes the drainage above Spanish Springs Dam and about 6 square miles of local area below the dam. The model is based on methods defined in TR-55 including SCS Curve Numbers (CNs) for loss rates and a SCS Unit Hydrograph for rainfall to runoff transform. The Corps modified the precipitation used in the model to a three-day storm using NOAA Atlas 14 Precipitation Frequency. The remainder of the watershed model, including Sun Valley and lower North Truckee Drain, was developed by the Corps and its Contractor (MWH, Inc.). The North Truckee Drain model uses the Green & Ampt soil loss method. The Green & Ampt loss parameters for the Spanish Springs and NTD models were developed by HDR, Inc. using the Maricopa County, AZ Design Manual guidance.

**2.5.1 Subbasin Delineation.** Subbasin boundaries in the Spanish Springs Valley HEC-1 model were adopted from the existing model. In the Corps North Truckee Drain HEC-1 model, the watersheds were divided into numerous subbasins using topographic data and available information about existing drainage systems. Topographic data in digital format were collected for watersheds tributary to the Truckee River between Highway 395 and Vista. These data consisted of the following:

- List data from the USACE in the Truckee Meadows Project study area
- List data from Washoe County
- USGS 30-meter digital elevation models

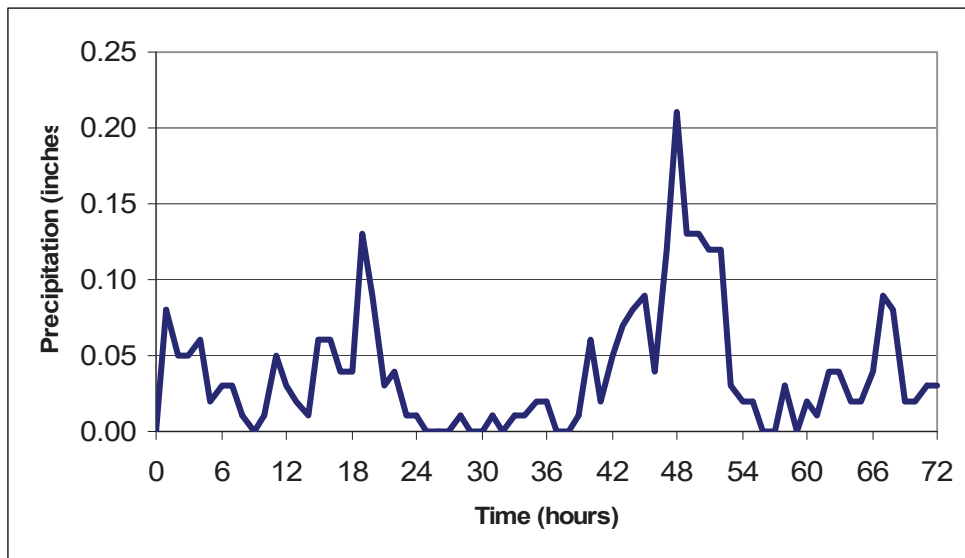
The topographic layers were merged to form a continuous digital land surface. Subbasins were then automatically generated in a GIS environment using the digital land surface in combination with hydrographic line data. The subbasins were manually refined or adjusted, where necessary, to effectively represent the watersheds and account for drainage features not apparent on the digital land surface.

**2.5.2 General Rainflood Storm Centered at Spanish Springs Dam.** This storm was derived to determine the existing condition peak flows for various frequency events on lower North Truckee Drain. The hydrographs generated will be used to determine residual damages from interior floods and to design the North Truckee Drain Re-Alignment (a proposed project feature). Six flood frequencies were evaluated in this study: the 20%, 10%, 2%, 1%, 0.5%, and 0.2%

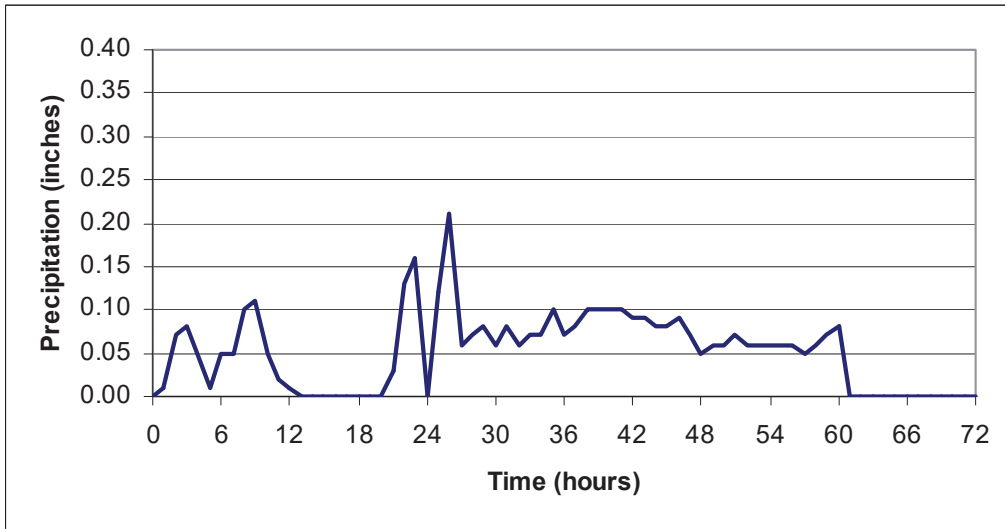
exceedence events. Precipitation data used in the HEC-1 model are based on partial duration NOAA Atlas 14 Depth Duration Precipitation Study which was completed in 2003. This study provides the most current annual, all-season precipitation frequencies for storm durations from 5 minutes to 60 days, and percent exceedences from 50% to 0.1%. Spatial data files (point-rainfall isohyetal maps) were downloaded from the National Weather Service's Precipitation Frequency Data Server as geographic shapefiles. The peak flow of record for North Truckee Drain is the 1986 flood, which was a general rain event.

**a. 72-Hour General Rainflood.** A historic storm pattern from 1986 for Reno Airport (AP) was used to develop 72-hour general rainflood precipitation for each subbasin. It was felt that inflow volume to the Spanish Springs Dam was an important factor in the analysis of the North Truckee Drain peak flows, and a historical look at past floods shows that more than 24-hours of precipitation can occur during one storm. Therefore, a longer duration than 24-hours was chosen for the design storm. The 1986 storm pattern and some other historic events are shown in the figures below.

**FIGURE 2-1. FEBRUARY 1986 STORM PATTERN FOR RENO AIRPORT, ADOPTED FOR USE IN GENERAL RAINFLOOD EVENT MODELING**



**FIGURE 2-2. RENO AIRPORT PRECIPITATION DISTRIBUTION FOR 22 – 24 DECEMBER 1955 STORM**



**FIGURE 2-3. RENO AIRPORT PRECIPITATION DISTRIBUTION FOR 1 – 3 JANUARY 1997 STORM**

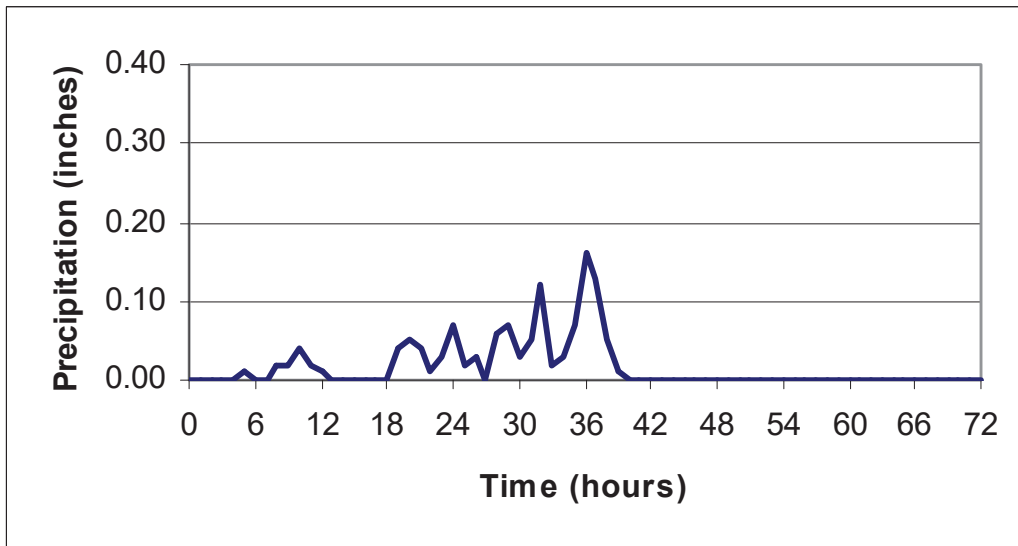


Table 2-3 shows the storm totals for various durations for the 1986 storm.

Table 2-3. 1986 Total Storm Depths Recorded at the Reno Airport Gage

	3-hr	6-hr	12-hr	24-hr	48-hr	72-hr
Total Depth	0.47	0.83	1.19	1.45	1.89	2.85
% of 24-hr depth	32%	57%	82%	100%	130%	197%

Table 2-4 shows the 1% chance exceedence NOAA 14 depths for the Subbasin named “SUNRIS” (tiny subbasin immediately downstream of Spanish Springs Dam; this is the bullseye of the general rain).

Table 2-4. NOAA 14 1% Chance Exceedence Storm Totals for Subbasin SUNRIS

	3-hr	6-hr	12-hr	24-hr	48-hr	72-hr
Total Depth	1.27	1.58	2.18	2.89	3.55	Not Available
% of 24-hr depth	44%	55%	75%	100%	123%	Not Available

Initially, the hourly ordinates of the 72-hour, 1986 storm pattern were multiplied by a ratio that would balance the maximum 24-hours to the 1% chance, 24-hour depth in NOAA 14. This caused the 12- and 48-hour durations to become more rare than the 1% event. Since this was not desired, the 12- and 48-hour depths were balanced to NOAA 14 as well. Since NOAA 14 does not provide a 72-hour depth, it was determined that the 72-hour depth should be a ratio of 1.07 of the 48-hour depth (based upon the ratio of the 1%, 3-day depth to the 1%, 2-day value at the Reno Airport for a Winter Season Frequency Curve). For subbasin SUNRIS, this depth was 131% of the 24-hour depth. As a result the following procedure was used to determine the design storm for each subbasin.

1. Determine the 24-hour Precipitation Frequency value in NOAA 14 for each subbasin using GIS.
2. Apply the following ratios to the 24-hour depth to obtain the other duration depths for each subbasin.

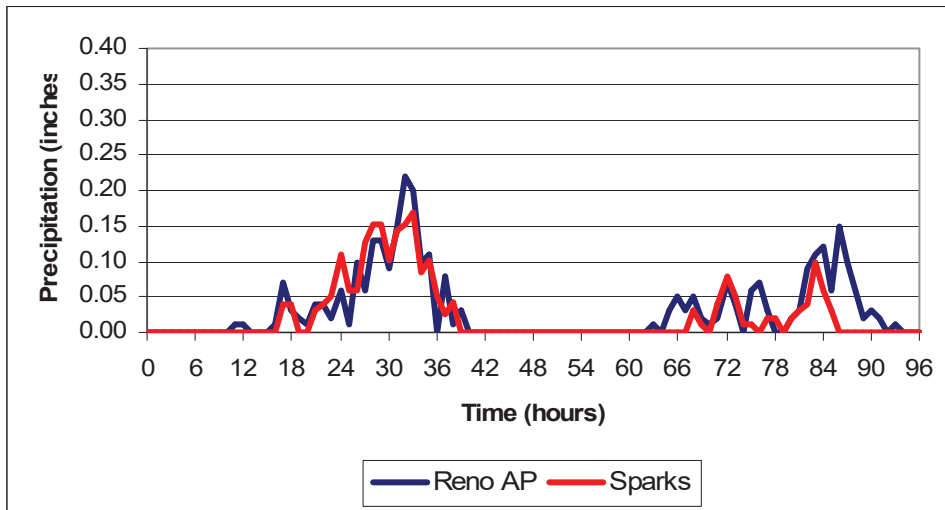
	3-hr	6-hr	12-hr	24-hr	48-hr	72-hr
% of 24-hr depth	32%	57%	75%	100%	122%	131%

3. Balance the 1986 Storm Pattern ordinates to the depth-duration values obtained in Step 2 above.
4. Apply the proper areal reduction factor to each precipitation depth based upon the subbasin’s proximity to the storm bullseye.

The Reno Airport gage storm pattern was considered appropriate for application to the North Truckee Drain watershed, based upon a comparison of two precipitation gages during the New Years Eve 2005 Flood Event, which was about a 2% chance flood in downtown Reno. The two gages were the Reno Airport gage and one near Spanish Springs Dam named KNVSPARK4. Figure 2-4 shows a comparison of the rainfall at the two gages during this event. Figure 2-5

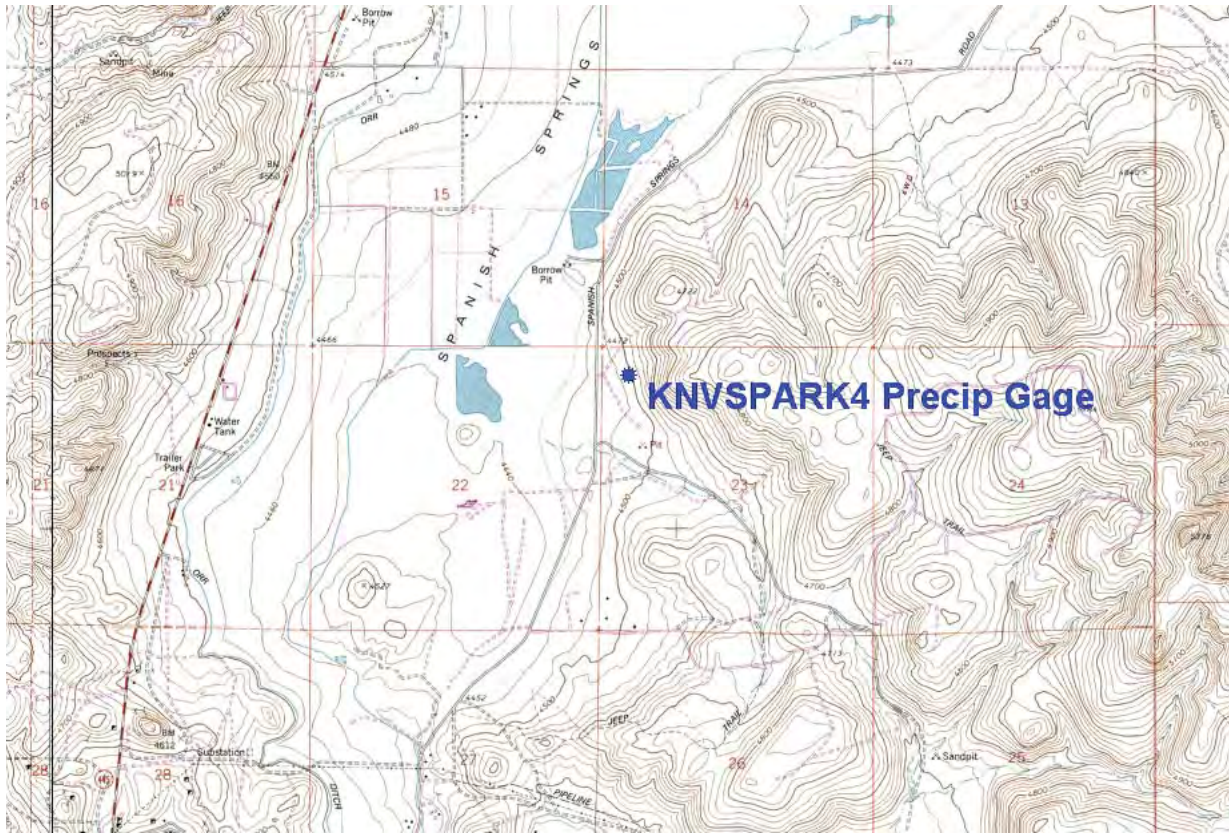
shows the location of the Sparks gage. The Sparks gage was not available during the 1986 storm event.

**FIGURE 2-4. RENO AP AND SPARKS PRECIPITATION DISTRIBUTION FOR 30 DECEMBER 2005 – 2 JANUARY 2006 STORM**



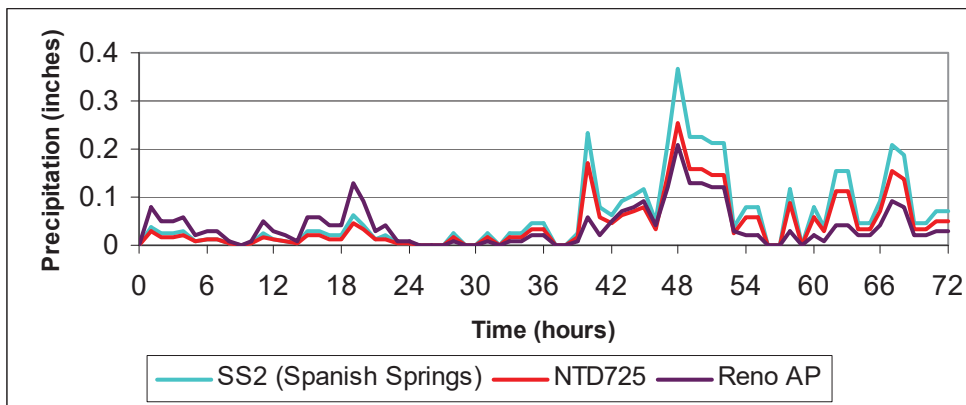
**Reno Airport and KNVSPARK4 (Vistas Off Los Altos Way) Recorded Precipitation for the 30 December 2005 to 2 January 2006 Storm (51 Hours of Actual Precipitation over a 96-Hour Period at Reno).**

**FIGURE 2-5. MAP SHOWING LOCATION OF KNVSPARK4 PRECIP GAGE**



The figure below shows the resulting design storm for two subbasins compared with the original 1986 storm at the Reno Airport.

**FIGURE 2-6. 1% GENERAL RAINFLOOD PRECIPITATION DISTRIBUTION, BASED ON FEBRUARY 1986 STORM**



**Reno Airport Precipitation for 17 – 19 February 1986 and Design Storm Hyetographs for a Spanish Springs Subbasin and a North Truckee Drain Subbasin.**

**Table 2-5** displays the 1% point rainfall depths found in NOAA Atlas 14 for a general rain event compared with the 1% chance point precipitation used in the HEC-1 model for two subbasins (one in Spanish Springs and one in North Truckee Drain). In addition, the depth-area reduced rainfall used in the HEC-1 model is also shown as a percentage of the NOAA 14 point precipitation depths. The adjustment for Drainage Area Reduction Factors (DARF) is discussed later in this report.

**TABLE 2-5  
DISTRIBUTION OF PEAK PERIOD RAINFALL  
FOR 1% GENERAL RAINFLOOD BASED ON 1986 EVENT  
Peak Period for 3-, 6-, 12-, 24-, 48-, and 72-hour Durations**

<b>Peak Period</b>	<b>3- hour</b>	<b>6- hour</b>	<b>12- hour</b>	<b>24- hour</b>	<b>48- hour</b>	<b>72- hour</b>
<b>Time of Peak (hours)</b>	48 to 50	47 to 52	42 to 53	40 to 63	25 to 72	1 to 72
<b>Spanish Springs Subbasin #2 (GriffithCanyon) (Drainage Area 4.38 sq.mi.)</b>						
NOAA 14 depths for subbasin (in.)	1.56	1.97	2.88	4.17	5.60	5.99
Actual point precipitation used in model (in.)	1.33	2.37	3.12	4.17	5.08	5.46
% of fully balanced storm (point precip) used in model	86%	121%	108%	100%	91%	91%
DARF-adjusted precipitation used in model (in.)	0.82	1.46	1.922	2.932	3.953	4.406
DARF-adjusted % of fully balanced storm (point precip) in model	53%	74%	67%	70%	71%	74%
<b>North Truckee Drain Subbasin #725 near Prater Way (Drainage Area 1.09 sq.mi.)</b>						
NOAA 14 fully balanced storm depths (in.)	1.28	1.61	2.26	3.07	3.85	4.12
Actual point precipitation used in model (in.)	0.98	1.75	2.30	3.07	3.75	4.03
% of fully balanced storm (point precip) used in model	77%	109%	102%	100%	97%	98%
DARF-adjusted precipitation used in model (in.)	0.57	1.01	1.33	2.07	2.83	3.17
DARF-adjusted % of fully balanced storm (point precip) in model	44%	63%	59%	67%	73%	77%

**b. Depth-Area Reduction Factors.** The precipitation data obtained from NOAA 14 is point precipitation, but, since it is being applied to each subbasin area, it is necessary to use depth area reduction factors (DARFs) to reduce the intensity of the precipitation based on storm duration and the area over which it is being applied. The storm shape chosen is a circular pattern. Those subbasins farthest from the bullseye were given the greatest amount of areal reduction. DARFs were taken from the Washoe County Flood Control Master Plan Meteorological Analysis, prepared by Hydmet, dated April 1994 (Reference 14). Hydmet is owned by Jack Humphrey, PhD, P.E. Hydmet analyzed precipitation gages in this region to determine precipitation frequency values and areal reduction factors for Washoe County. The equations are shown in Table 2-6, although not all were used. The equations are based upon precipitation studies in Utah and Arizona. Example reduction factors are given for 1, 5, 10, and 100 square miles and for durations from 5 minutes to 3 days. Note from the table that areal

reductions for shorter durations are more severe than for longer durations. For example, for an area of 5 square miles, the 5-minute duration DARF is 0.61, while for the 3-day duration the DARF is 0.97, or over fifty-percent less reduction. **TABLE 2-6**

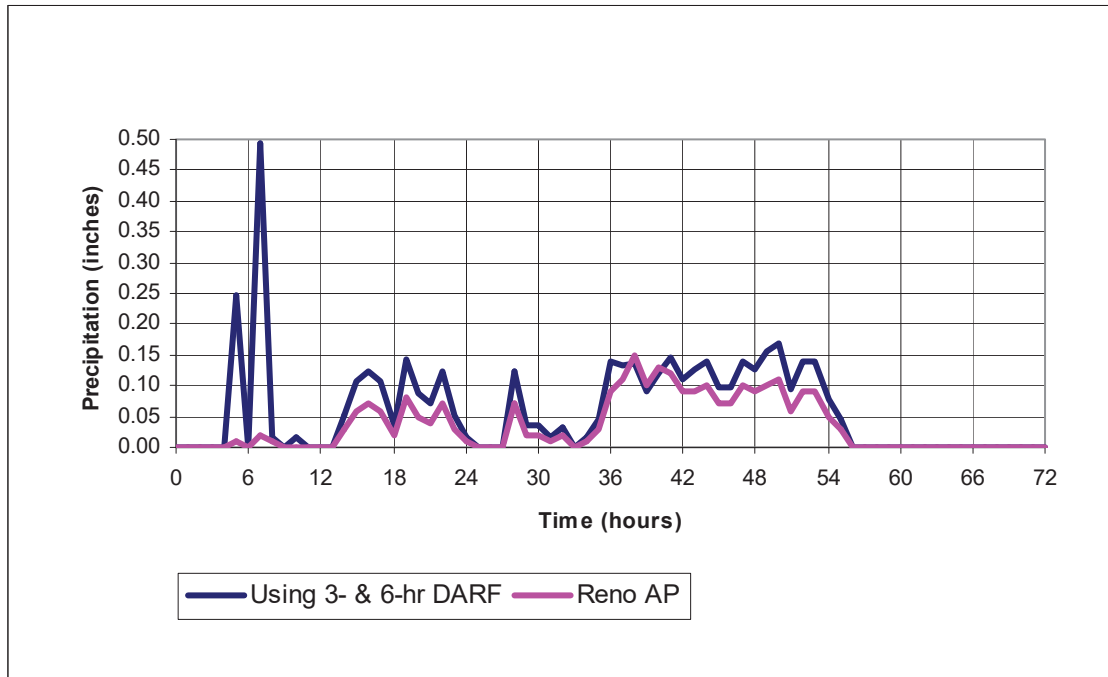
**DEPTH AREA REDUCTION FACTORS**

Rainfall Duration	DARF Equation	DARF 1 square mile	DARF 5 square miles	DARF 10 square miles	DARF 100 square miles
*5-min	.01*(100-18.5*Area <sup>.46</sup> )	0.82	0.61	0.47	N/A
*10-min	.01*(100-14.2*Area <sup>.46</sup> )	0.86	0.70	0.59	N/A
*15-min	.01*(100-12.0*Area <sup>.46</sup> )	0.88	0.75	0.65	N/A
*30-min	.01*(100-9.2*Area <sup>.46</sup> )	0.91	0.81	0.73	N/A
*1-hour	.01*(100-7.0*Area <sup>.46</sup> )	0.93	0.85	0.80	N/A
*2-hour	.01*(100-5.3*Area <sup>.46</sup> )	0.95	0.89	0.85	N/A
*3-hour	.01*(100-4.5*Area <sup>.46</sup> )	0.96	0.91	0.87	0.63
*6-hour	.01*(100-3.5*Area <sup>.46</sup> )	0.97	0.93	0.90	0.71
12-hour	.01*(100-2.6*Area <sup>.46</sup> )	0.97	0.95	0.93	0.78
1-day	.01*(100-2.0*Area <sup>.46</sup> )	0.98	0.96	0.94	0.83
2-day	.01*(100-1.5*Area <sup>.46</sup> )	0.99	0.97	0.96	0.88
3-day	.01*(100-1.3*Area <sup>.46</sup> )	0.99	0.97	0.96	0.89

\*Note: Durations shaded were not considered applicable for general rain, only cloudburst.

Note that it was found that using the 3- and 6-hour DARF factors for the outlying subbasins (furthest from the storm center), resulted in the maximum 3- and 6-hour rainfall depths being reduced to unusually small amounts. The reduction was so drastic that the rainfall for these durations was smaller than for the longer durations. The original pattern was significantly altered. The figure below illustrates the problem.

**FIGURE 2-7. 1963 STORM PATTERN COMPARED WITH SUBBASIN #405 BALANCED HYETOGRAPH AFTER APPLYING 3-, 6-, 12-, 24-, 48-, AND 72-HOUR DARF FACTORS**



Notice that rainfall around hours 3 through 9 get pushed extraordinarily high compared to the original pattern. This is due to the short duration rainfall being pushed down too far by the areal reduction factors when compared to the longer durations. Further analysis determined that the 3- and 6-hour DARF factors, found in Reference 14 (Washoe County Flood Control Master Plan Meteorological Analysis), were more appropriate for use in a cloudburst storm, not a general rain distribution. To solve this, the areal reduction factors for the 3-hour and 6-hour durations were not used. Instead, only the 12-hour DARF factor was applied to the maximum 3-, 6-, and 12-hours of precipitation in the design storm.

**c. Storm Centerings:** Storms were based on a circular pattern with no maximum storm size. The storm was expanded outward until all subbasins in the HEC-1 model were covered. The centering or bullseye of the general rainstorm for the North Truckee Drain was in subbasin #1160, which is located in the Spanish Springs Valley, just below Spanish Springs Dam. The purpose of the centering was to create high peak flows on the North Truckee Drain channel. Chart 2-2 shows the bullseye as a red circle for the North Truckee Drain General Rainstorm.

**d. Rain-on-Snow:** As mentioned earlier, local experts say there is no discernable snowpack in the North Truckee Drain watershed. As a test, the Corps used a snowpack model and antecedent snowpack criteria developed for Steamboat Creek on the North Truckee Drain watershed. There are no subbasins above 5,500 feet elevation in the Spanish Springs Valley. Only six subbasins in North Truckee Drain model had elevations above 5,500 feet, with the potential of snowmelt contribution. The actual area above 5,500 feet was a very small percentage of each subbasin area. As a test, the SNOW program was used with these six subbasins to determine the net change in runoff due to snowmelt contribution. The result of running a rain-on-snow model simulation for the 1% chance event only changed the flow on

North Truckee Drain by 50 cfs. This basically verified the opinion of local experts. The final design storm for this watershed did not include snowpack modeling.

### **2.5.3 Soil Loss Parameters**

**a. Soil Type, Land Use, and Cover.** The Spanish Springs HEC-1 model adopted from the City of Sparks uses SCS Curve Numbers for soil loss rates. A Corps' contractor also derived curve numbers for the North Truckee Drain model. Initially, this type of loss rate was used for several HEC-1 model runs using several different storm patterns. It became apparent that something might be wrong with using Curve Numbers for a 3-day storm event, as the Curve Numbers seemed to have unusually low loss rates as the storm progressed beyond the initial 24-hours. An External Peer Review was performed by Dr. Joe Devries of David Ford Consulting Engineers, Inc. His analysis confirmed that the curve number loss rates were approaching almost zero inches per hour on the third day of the storm event (Reference 35). Instead, the reviewer suggested using the Green and Ampt method as described in the Maricopa County, AZ Design Manual. Consequently, the Corps converted both the Spanish Springs and the North Truckee Drain manual to Green and Ampt soil loss rate method. A comparison of results from the two soil loss rate methods is shown in Table 2-10.

**b. Green & Ampt Loss Parameters.** The Green & Ampt loss method uses the following parameters in the HEC-1 model:

IA – initial loss in inches, primarily a function of land-use and surface cover (see Table 2-8). Zero rainfall excess is generated until the initial loss is satisfied.

DTHETA – volumetric measure of the soil moisture storage capacity available at the start of the storm (dry, normal, or saturated soil conditions).

PSIF – Wetting front suction in inches.

XKSAT – hydraulic conductivity at natural saturation in inches per hour.

RTIMP – percent impervious (see Table 3-4).

The parameters, DTHETA, PSIF, and XKSAT, are functions of soil characteristics, ground surface characteristics, and land management practices. The primary soil surface characteristics are vegetation canopy cover, ground cover, and soil crusting. Land management practices effect soil porosity. The parameters for bare ground are dependent on soil type, as tabulated in Table 2-7, below. The Corps used the DTHETA values for a normal antecedent soil moisture condition. Vegetation cover in each subbasin was accounted for in the process to derive the final loss rate.

**TABLE 2-7  
GREEN & AMPT LOSS RATE PARAMETER VALUES FOR BARE  
GROUND**

Soil Classification	XKSAT (in/hr)	PSIF (in.)	DTHETA		
			Dry	Normal	Saturated
A1 loamy sand & sand	1.20	2.4	0.35	0.30	0.00
A2 sandy loam	0.40	4.3	0.35	0.25	0.00
B1 loam	0.25	3.5	0.35	0.25	0.00
B2 silty loam	0.15	6.6	0.40	0.25	0.00
B3 silt	0.10	7.5	0.35	0.15	0.00
C sandy clay loam	0.06	8.6	0.25	0.15	0.00
D1 clay loam	0.04	8.2	0.25	0.15	0.00
D2 silty clay loam	0.04	10.8	0.30	0.15	0.00
D3 sandy clay	0.02	9.4	0.20	0.10	0.00
D4 silty clay	0.02	11.5	0.20	0.10	0.00
D5 clay & rocky outcrop	0.01	12.4	0.15	0.05	0.00

The drainages in Washoe County are a mixture of several different soil types, so composite values for the Green & Ampt parameters need to be determined for each subbasin. The procedure to determine the subbasin composite value of XKSAT is to average the area-weighted logarithms of the XKSAT values of each soil type in that subbasin. The corresponding subbasin composite values for PSIF and DTHETA parameters are based on the subbasin XKSAT as determined from a graph (Figure 4.3 in the Maricopa County Design Manual). A final adjustment of the composite XKSAT is made based on subbasin percent vegetation cover.

Land use data, provided by Washoe County, were available for the study area. Land use information provided by Washoe County was rectified with the Maricopa County Design Manual land-uses. The consolidated land use categories with related percent impervious, basin n, and vegetation cover, are listed in **Table 2-8**. Basin n values are used in the calculation of lag (Los Angeles S-graph method) as described in the next section. Unit hydrographs derived from S-graphs were used in the North Truckee Drain model. These basin n values were adopted from the *Sacramento City/County Drainage Manual*, 1994. The A-E firm that derived the n values agreed that these values could be adopted for the Reno area. Acreage of each land use and soil type was extracted for each subbasin using ArcView. Composite values for initial loss, percent impervious, vegetation cover, and basin n value were determined for each subbasin using the ArcView land use data.

**TABLE 2-8  
LAND USE CATEGORIES AND CORRESPONDING DATA**

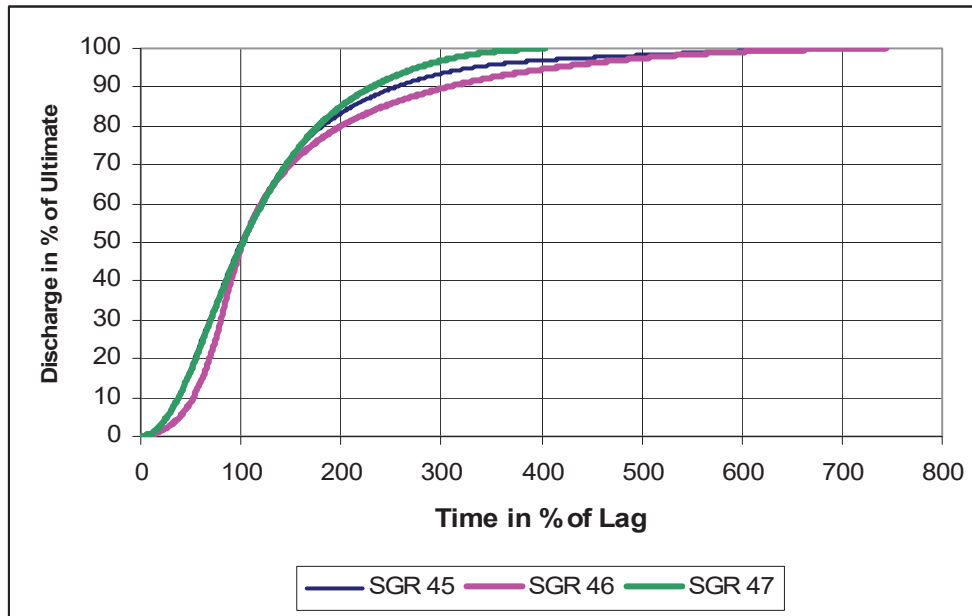
Description of Land Use	Percent Imper-vious	N value	Initial Loss (IA)	Vegetation Cover
			(inches)	in %
Pasture - heavily grazed	5%	0.075	0.30	30%
Parks - fair grass cover	5%	0.070	0.30	60%
Parks - good grass cover	5%	0.070	0.30	80%
Major Highways	90%	0.030	0.10	0%
Commercial – Business	85%	0.031	0.10	75%
Industrial	72%	0.032	0.12	70%
Residential - 1/8 acre lots	65%	0.037	0.20	65%
Residentail - 1/4 acre lots	38%	0.042	0.25	50%
Residentail - 1/2 acre lots	25%	0.060	0.25	50%
Residential - 1 acre lots	20%	0.073	0.30	50%
Water Bodies	100%	-	0.00	0%

**2.5.4 Hydrograph Development.** The computer program SACPRE, a pre-processor to HEC-1, was used to develop unit hydrographs using the Corps S-graph method. An S-graph is not a unit hydrograph; it represents the hydrologic response of a basin to a rainfall event of infinite duration. The curve has a rising limb with a characteristic S-shape that eventually reaches a constant discharge rate equal to the constant excess rainfall rate multiplied by the area of the basin. An S-graph (shown in the figure below) is constructed from a series of unit hydrographs of length T by the process of successive lagging by T and summing the discharge values (ordinates). The beauty of the S-graph method is that one can derive unit hydrographs for events of any desired duration from them. The Corps Sacramento District developed three S-curves, a mountain general rain S-graph (SGR 45), a mountain cloudburst S-graph (SGR 47) and a valley general rain and cloudburst S-graph (SGR 46), specifically for use in the Truckee Meadows area. The S-graphs were developed for the 1980 Truckee River Study (Ref. 5). The S-curve procedure was not used in the Spanish Springs HEC-1 model. The Spanish Springs model adopted from Washoe County used the SCS unit hydrograph method.

**TABLE 2-9. S-CURVES FOR THE TRUCKEE MEADOWS AREA**

SACPRE Description	Curve No.
Truckee Meadows: Mountain General Rain	45
Truckee Meadows: Valley General Rain & Cloudburst	46
Truckee Meadows: Mountain Cloudburst	47

**FIGURE 2-8. TRUCKEE MEADOWS AREA S-GRAPHS**



The computer program SACPRE, originally developed by the Corps Los Angeles District as LAPRE-1, was used as a pre-processor to HEC-1 to produce unit hydrographs for each subbasin based on the drainage area, basin lag, and chosen S-graph.

**2.5.5 Routing Parameters.** Once rainfall has turned into runoff and flowed to the outlet point for a subbasin, it is necessary to combine this flow with upstream subbasin flows. The combined flow hydrograph is then routed through stream and drainage channels to the next subbasin outlet. There are a number of hydrologic routing techniques available in the HEC-1 program, such as Muskingum, Modified Puls, Muskingum-Cunge, and Kinematic Wave. While any of these techniques may be used, routing techniques were generally applied to this study as described below, based on a number of sources:

- 1983 Corps HEC-1 model (Muskingum and Modified Puls)
- Information from local government and their consultants (Muskingum and Muskingum-Cunge)
- Determination of travel time and attenuation developed for this study (Muskingum)
- Channel routing information from Corps HEC-RAS models of North Truckee Drain

**2.5.6 Special Facilities and Model Features.**

a. Diversions and Flow Splits. Diversions and flow splits were included in the model routing of subbasin flows where indicated from previous work by others. Diversion and flow split parameters were entered directly from existing models, where available.

b. Detention Basins. The major detention facilities include the North Spanish Springs Detention Facility (584 ac-ft, subbasin 1910), and Spanish Springs Dam (1500 ac-ft, subbasin

1500) within the Spanish Springs HEC-1 model. Detention facilities within the North Truckee Drain HEC-1 model include Sun Valley Detention Basin (subbasin 510), Paradise Park Detention Pond (subbasin 605), the Pah Rah Dams (subbasin 700), the D'Andreas Dams (subbasin 710) and the Sparks Marina next to Highway 80.

**2.5.7 Model Verification.** It is difficult to verify the computer models for Spanish Springs and North Truckee Drain. The upper gage has 10 years of broken record and the lower gage only has 12 years of record and is influenced by backwater from the Truckee River. Historic precipitation data are not available to calibrate a rainfall runoff model for this watershed. The 2005 flood event was too small on North Truckee Drain to be useful.

**2.5.8 Model Runs** As mentioned earlier, initial model runs were done using SCS Curve Numbers for loss rates. After completion of the External Peer Review on the North Truckee Drain HEC-1 model, the Corps changed to the Green and Ampt loss rate method. A comparison of the results between the two methods is shown below for the inflow and outflow for Spanish Springs Detention Basin for the 1% chance event.

**TABLE 2-10. Model Run Comparison at Spanish Springs Dam**

Loss Rate Method	Design Storm	Spanish Springs Det. Basin		
		Inflow (cfs)	Outflow (cfs)	Notes
SCS Curve Numbers	72 Hour Event, 1986 Pattern	2,950	1,910	Run #8: 86 Reno Airport pattern, 72-hr storm, model run based on 15-minute time computation steps
Green & Ampt	72 Hour Event, 1986 Pattern	1,040	530	Adopted model for this study, model run based on 5-minute time computation steps. Same storm as used above.

**TABLE 2-11. Model Run Comparison on North Truckee Drain Near Prater Way**

Loss Rate Method	Design Storm	North Truckee Drain at Prater Way	
		Peak Flow (cfs)	Notes
SCS Curve Numbers	72 Hour Event, 1986 Pattern	2,470 cfs	Run #8: 1986 Reno Airport pattern, 72-hr storm, model run based on 15-minute time computation steps
Green & Ampt	72 Hour Event, 1986 Pattern	1,910 cfs	Adopted model for this study, model run based on 5-minute time computation steps. Same storm as used above.

The North Truckee Drain at Spanish Springs Road streamgage record is shown below.

Table 2-12. USGS Gage #10348245 Peak Flows

Date of Peak	Discharge in CFS
02/17/86	1850
07/14/93	34
09/28/94	31
03/11/95	27
01/02/97	15
08/01/02	43
06/20/03	20
06/26/04	23
06/24/05	115
12/31/05	286

For purposes of comparison, the 1986 peak flow at Spanish Springs Road can be compared to the Spanish Springs inflow as modeled in HEC-1, although the gage site has an additional 6 square miles of drainage area. Of note is that the 1986 peak flow of 1,850 cfs exceeds the peak inflow for the 1% chance exceedence HEC-1 model run (1,040 cfs). A number of possible explanations exist: 1) The HEC-1 model has the North Spanish Springs Detention Facility (storage of over 500 acre-feet) which did not exist in 1986, 2) the HEC-1 model storm bullseye was located to maximize peak flow on the lower North Truckee Drain rather than to maximize inflow to Spanish Springs Dam, 3) the streamgage has an additional 6 square miles of drainage area as compared to Spanish Springs Dam 4), although significant development has occurred in the Spanish Springs watershed since 1986, developers have been mitigating for urbanization by installing numerous detention basins throughout the area, 5) the 1986 peak discharge was not measured directly but rather by an indirect estimate based on high water marks.

Unfortunately, precipitation within the North Truckee Drain watershed was not measured during the 1986 flood. Several precipitation gages in the region were checked for durations from 1 hour to 3 days to determine the % chance exceedence that occurred. See the table below.

**TABLE 2-13**  
**% CHANCE EXCEEDENCE FOR PRECIPITATION AMOUNTS**  
**FOR 5 STATIONS FOR VARIOUS DURATIONS FOR THE**  
**FEBRUARY 1986 STORM EVENT**

Duration	Reno AP	Minden AP	Mt Rose Xmas Tree	Carson City	Virginia City
1-Hour	<20%	<20%	20%	N/A	N/A
2-Hour	<20%	<20%	20%	N/A	N/A
3-Hour	<20%	20%	10%	N/A	N/A
6-Hour	20%	20%	<10%	N/A	N/A
12-Hour	20%	<20%	<10%	N/A	N/A
24-Hour	<10%	20%	<4%	<20%	20%
2-Day	<10%	20%	<4%	20%	10%
3-Day	4%	<4%	10%	10%	10%

"<" means more frequent than...

N/A = not available

**2.6 Summary of Results.** Table 2-14 summarizes simulated peak flows (in cubic feet per second) for the general rain flood with a bullseye at the outlet of the Spanish Springs Detention Basin.

**TABLE 2-14  
SUMMARY OF SIMULATED PEAK FLOWS  
FOR THE GENERAL RAIN FLOOD SERIES**

Location	PEAK FLOWS (CFS)				
	Exceedence Frequency				
		10%	2%	1%	
Spanish Springs Detention Basin					
Inflow		420	800	1,050	
Outflow		380	490	530	
NTD @ Prater Road		960	1,610	1910	

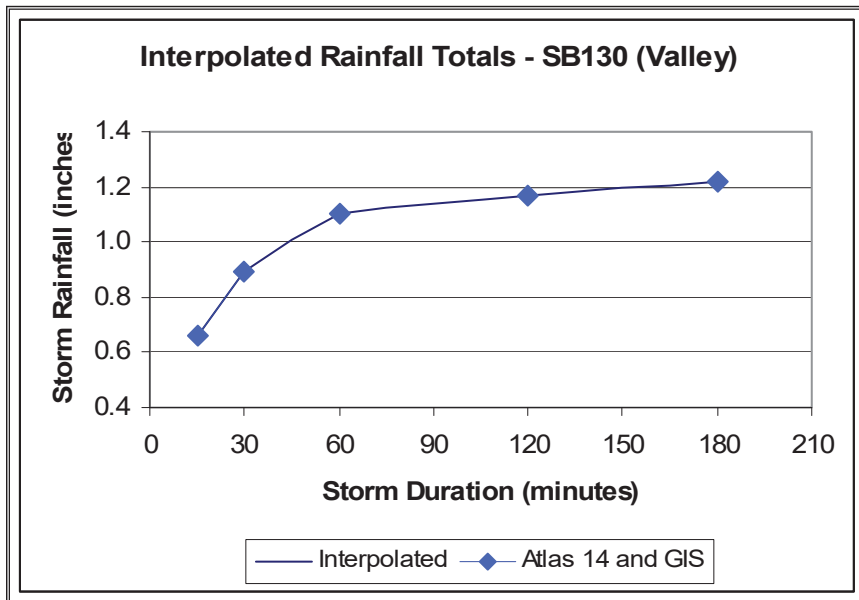
**2.7 Hydrology for Areas Behind Levees.** Planned project features include a levee on the north side of the Truckee River in the Truckee Meadows Reach. To evaluate interior flooding induced by the levee, two types of storms were evaluated: a 72-hour general rain event centered near the levees and a 3-hour cloudburst centered next to the levees. A 3 hour duration cloudburst was based on local observations of typical events. These cells tend to be isolated and cover at most about 5 square miles. Both cloudburst storms and general rainstorms need to be analyzed to see which will cause the most critical flood conditions for these interior areas. The subbasins of concern are 405, 410, 415, 420, 425, and 517. It was determined (Memorandum: Review FLO-2D Model, Jan 2012, Ford Consulting) using FLO-2D that runoff from subbasins north of Hwy 80 do not cross the highway but flow southeast to the Sparks Marina. At this time, the HEC-1 model was subsequently revised to reflect this routing. For interior drainage floodplain mapping on the south side of Hwy 80, the individual subbasin hydrographs were divided into smaller identical pieces and input into FLO-2D cells at the discretion of the hydraulic modeler. The HEC-1 model routing in this area was not used and should not be considered accurate since overland flow direction changes for different frequency events.

**2.7.1 3-Hour Cloudburst for Subbasins 404, 405, 410, 420, 425 and 517.**

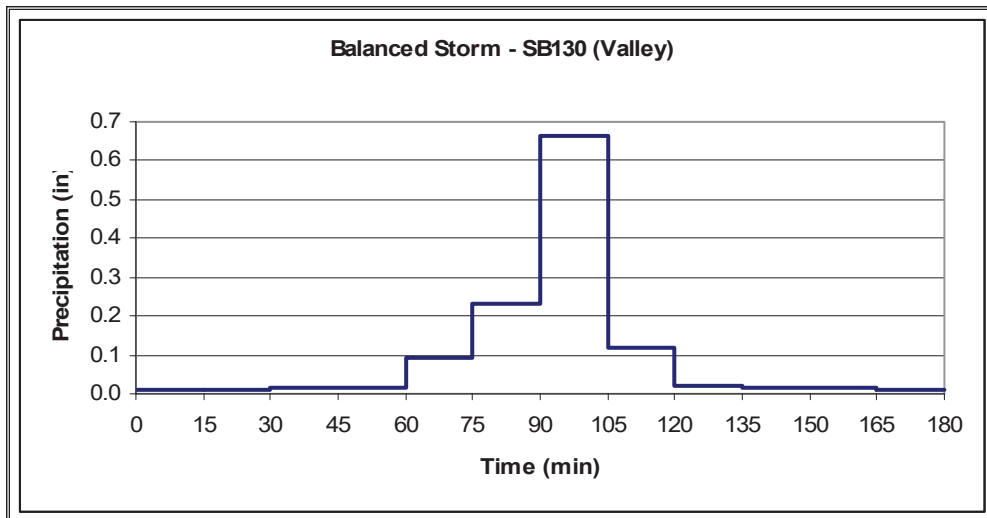
Subbasin precipitation for the 3-hour cloudburst event was developed in 15-minute increments using a balanced hydrograph approach. The HEC-1 model is run in 5-minute time steps, which means each 15-minute precipitation depth provided in the dssfile is divided evenly into three additional increments by HEC-1. Consequently, if one 15 minute increment of rainfall is 0.15 inches, the model splits the precipitation into three five minute increments of 0.05 inches. As with the general rainflood, six cloudburst flood frequencies were modeled: the 20%, 10%, 2%, 1%, 0.5%, and 0.2% exceedence events. NOAA Atlas 14 provided the rainfall data for 15-, 30-, 60-, 12-, and 180-minute durations for each subbasin analyzed for interior drainage flooding. The 45-, 75-, 90-, 105-, 135-, 150-, and 165-minute precipitation totals were then estimated using

log-log interpolation. An example of the development of the 1% chance cloudburst event for a subbasin is shown in Figures 2-9 and 2-10.

**FIGURE 2-9. INTERPOLATED CLOUDBURST RAINFALL TOTALS - SB130 (VALLEY)**



**FIGURE 2-10. BALANCED CLOUDBURST STORM – SB130 (VALLEY)**



### **2.7.2 General Rain**

A 72-hour general rain storm based on the 1986 storm pattern was used for interior drainage for subbasins 405, 410, 415, 420, 425 and 517. The design storm was created in 1-hour increments of rainfall, although the HEC-1 model was run in a 5-minute timestep.

### **2.7.3 Depth Area Reduction for Both Cloudburst and General Rain Storms**

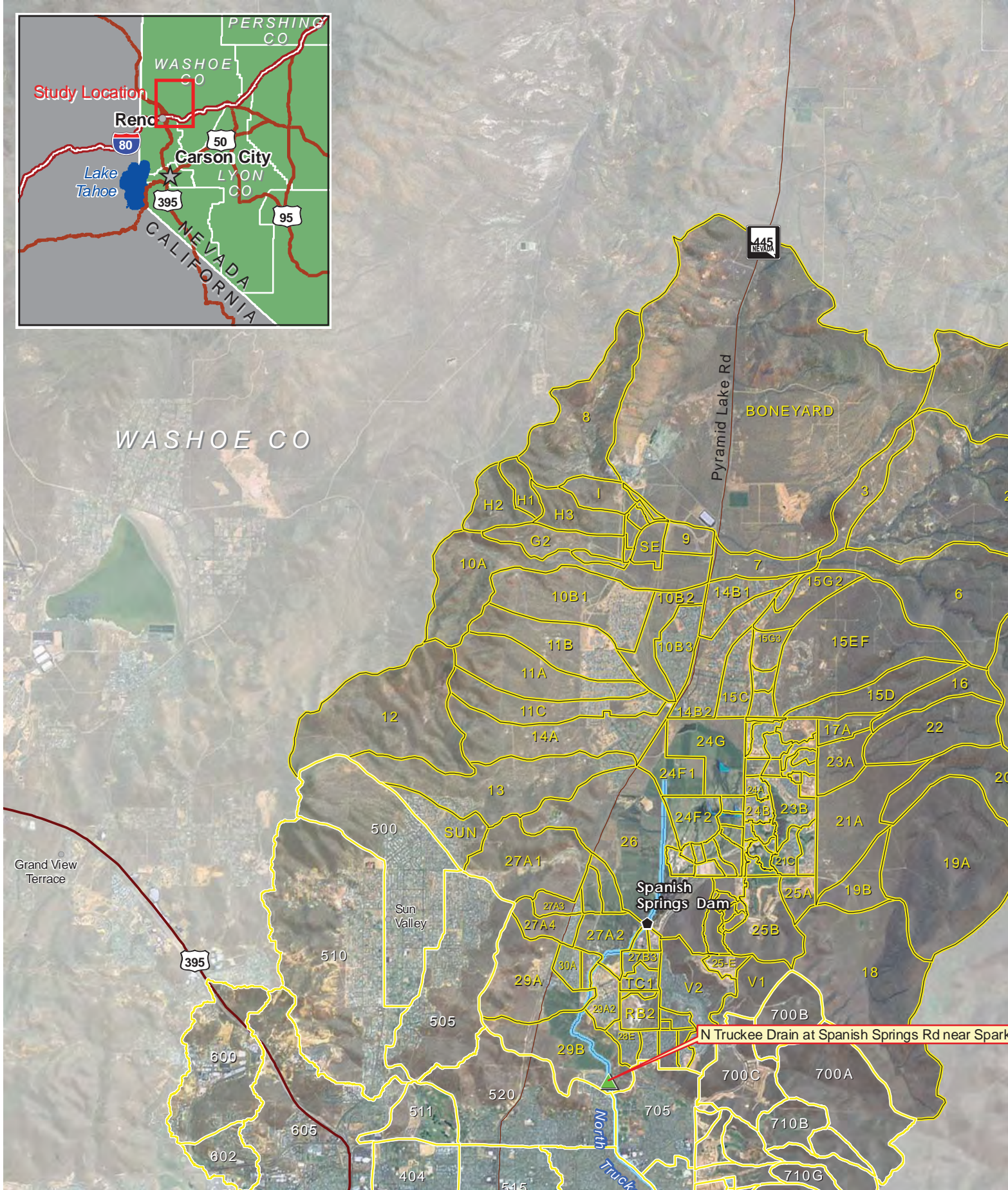
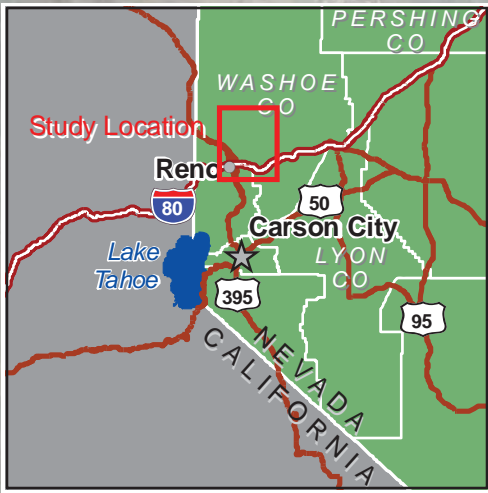
Point precipitation for subbasin 405 was adjusted with a depth area reduction factor for approximately 3 square miles (originally it was believed 404 flowed into subbasin 405). As this adjustment only decreases the point rainfall by 3%, the model was not re-run to use point precipitation. Point precipitation was used for subbasins 410, 415, 420 and 425. A minor ridge exists on the south side of subbasin 517 which confines the water to flow northwest towards People's Drain. HEC-1 model routings were not used on the south side of the Hwy 80. Hydraulic Design's FLO2D model covers this entire area. The hydrographs for subbasins downstream of the freeway were split into smaller hydrographs and input into appropriate FLO2D cells for routing to model the floodplain. See the Hydraulic Appendix for results.

### **2.7.4 Results**

Table 2-15 summarizes the peak flows and volumes for the five interior drainage subbasins for the cloudburst and general rainflood series. In all cases, the cloudburst flood peak flows are significantly larger than the general rainflood peak flows. Volumes for the general rainflood series are larger than the cloudburst volumes.

**Table 2-15**  
**Interior Drainage Results for Subbasins Next to Levees**

Subbasin Abbrev.	405		410		415		420		425	
Drainage Area	2.93 sq.mi.		0.55 sq.mi.		0.69 sq.mi.		0.24 sq.mi.		0.21 sq.mi.	
Cloudburst Flood Series	Pk (cfs)	6-Hr Vol. (ac-ft)	Pk (cfs)	6-Hr Vol. (ac-ft)	Pk (cfs)	6-Hr Vol. (ac-ft)	Pk (cfs)	6-Hr Vol. (ac-ft)	Pk (cfs)	6-Hr Vol. (ac-ft)
20%	399	83	164	17	253	22	104	8	91	6
10%	488	96	205	19	313	25	129	9	115	7
2%	792	131	333	26	502	35	205	13	194	10
1%	966	151	405	31	611	41	251	15	240	12
0.50%	1,162	174	486	35	731	47	298	17	289	14
0.20%	1,494	220	624	44	935	59	381	22	373	18
Subbasin Abbrev.	405		410		415		420		425	
General Rainflood Series	Pk (cfs)	3-Day Vol. (ac-ft)	Pk (cfs)	3-Day Vol. (ac-ft)	Pk (cfs)	3-Day Vol. (ac-ft)	Pk (cfs)	3-Day Vol. (ac-ft)	Pk (cfs)	3-Day Vol. (ac-ft)
20%	127	92	51	30	68	42	27	18	24	18
10%	150	108	59	54	83	66	33	24	28	22
2%	206	149	82	72	120	96	47	36	39	30
1%	232	168	92	84	138	108	55	42	45	34
0.50%	260	188	103	90	154	120	60	48	49	37
0.20%	297	216	120	102	178	138	69	54	58	43



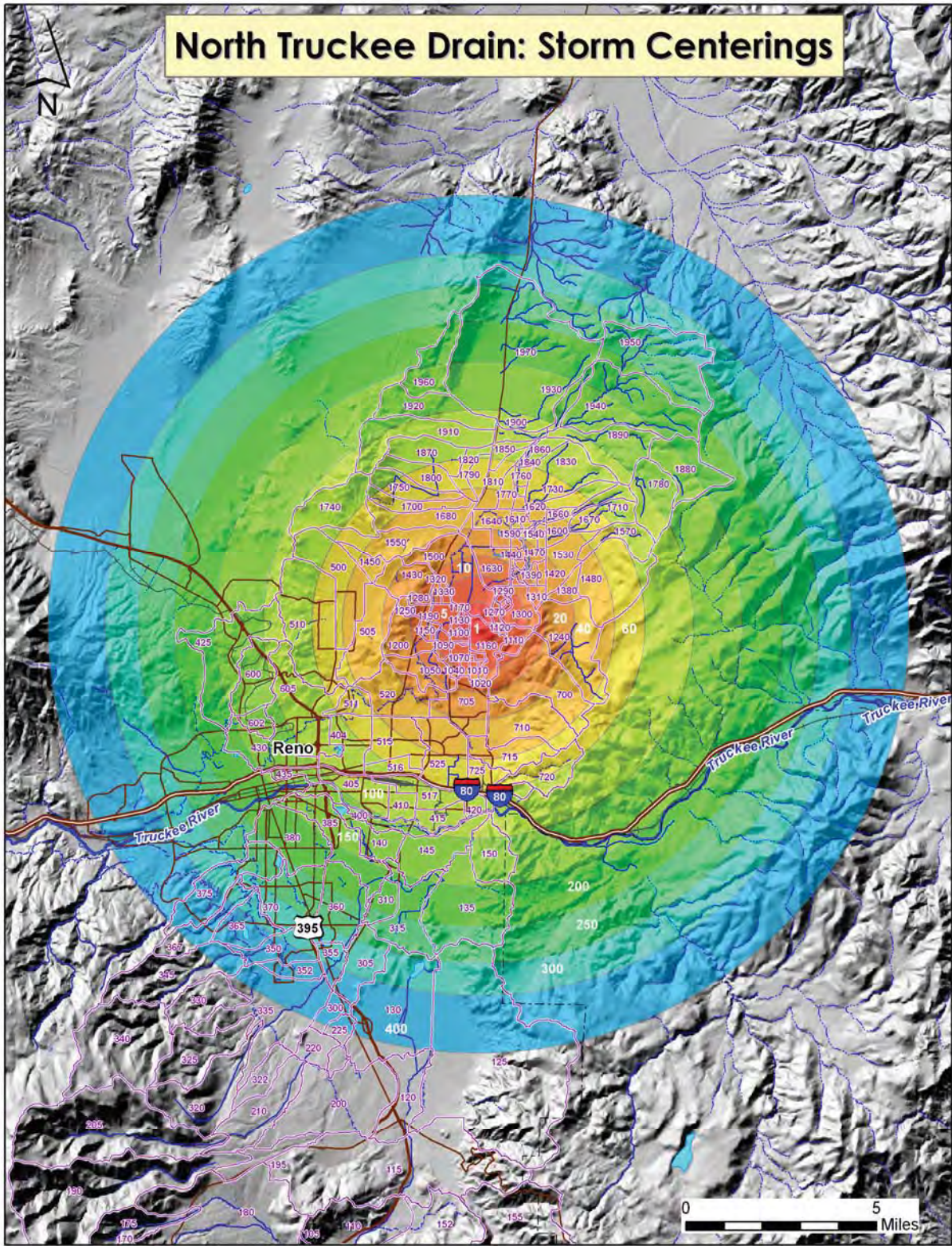


Chart 2-2

### 3.0 Steamboat Creek

**3.1 Study Area.** This memorandum specifically addresses the runoff characteristics of the south side of the river in the Truckee Meadows Reach. The Steamboat Creek drainage is bounded by the Carson Range to the west and by the Virginia Range to the east. Steamboat Creek generally runs from south to north towards the Truckee River. Washoe Lake and Washoe Valley, an 85-square-mile watershed in the southernmost part of the watershed rarely contributes flow to Steamboat Creek. The large lake surface normally absorbs all the runoff during flood events and outflow is confined to a small weir-like structure with flashboards. The lake has to develop a significant head surface over the weir to be able to produce much outflow. The contribution from this part of the watershed during a 1% chance exceedence event on Steamboat is estimated to only be a couple hundred cfs. As in the 1980 Corps Study, this portion of the watershed was not modeled.

The southernmost tributaries on Steamboat Creek include Browns, Galena, and Bailey Canyon creeks. Flowing in from a more westerly direction are Whites and Thomas Creeks. Thomas Creek splits into two parts downstream of Last Chance Ditch. The majority of flow continues northeast and joins Steamboat Creek near the Huffaker Hills while the other split flows in a more northerly direction and connects with Boynton Slough. Dry and Evans Creeks are on the far northwest side of the watershed and once combined, form Boynton Slough. Boynton Slough passes through the airport area and connects with Steamboat Creek near the mouth of the entire watershed. The locally preferred plan includes a proposed dam at Huffaker Hills. Three streamgages exist in the watershed (Steamboat Creek at Steamboat, Steamboat Creek at Short Lane, and Steamboat Creek at Clean Water Way). A watershed map and the location of the proposed dam and existing gages are shown on Chart 3-1. The entire watershed extends up the slope of Mt Rose, which rises above 10,000 feet elevation and which has a significant winter snowpack. The summit of Mount Rose, elevation 10,776 feet, is the highest point in the watershed.

**3.2 Historical Floods.** The three largest floods of record on Steamboat Creek occurred in February 17<sup>th</sup>, 1986, January 1<sup>st</sup>, 1997, and December 31<sup>st</sup>, 2005. The flow records for these three events are shown below.

#### Peak Flows of Record on Steamboat Creek

Date of Peak	Steamboat Crk @ Steamboat (cfs)	Steamboat Crk @ Short Lane	Steamboat Crk@ Clean Water Way
02/17/86	3,600	5,600 <sup>1</sup>	Not available
01/01/97	2,090	Not available	Not available
12/31/05	3,600	3,180	3,870

<sup>1</sup> This was an estimate based on slope-area measurements made by local officials

### **3.3 HEC-1 Models**

The Steamboat Creek HEC-1 computer model was developed by MWH (Montgomery Watson Harza) for the U.S. Army Corps of Engineers. Subbasin unit hydrographs in the model were developed from S-curves derived from an analysis of the December 1955 and January-February 1963 flood events, presented in the Truckee River Hydrology Office Report dated February 1980 (Reference 5). The Steamboat Creek model does not include the Washoe Valley basin to the south, as Washoe Lake produces minimal outflow during large events. The original model also used SCS Curve Numbers for soil loss rates. The Corps attempted to calibrate the HEC-1 model to observed peak flows during the December 31<sup>st</sup>, 2005 flood event, but the resulting peaks were too high (at least by a factor of two). After the External Peer Review of the North Truckee Drain HEC-1 model, the Corps decided to change the model to the Green and Ampt Loss rate based on procedures found in the Maricopa County, AZ design manual. The model calibration worked better after this conversion.

**3.4 Subbasin Delineation**. The Steamboat Creek watershed was divided into numerous sub-watersheds or subbasins using topographic data and available information about existing drainage systems. Topographic data in digital format were collected for watersheds tributary to the Truckee River between Highway 395 and Vista. These data consisted of the following:

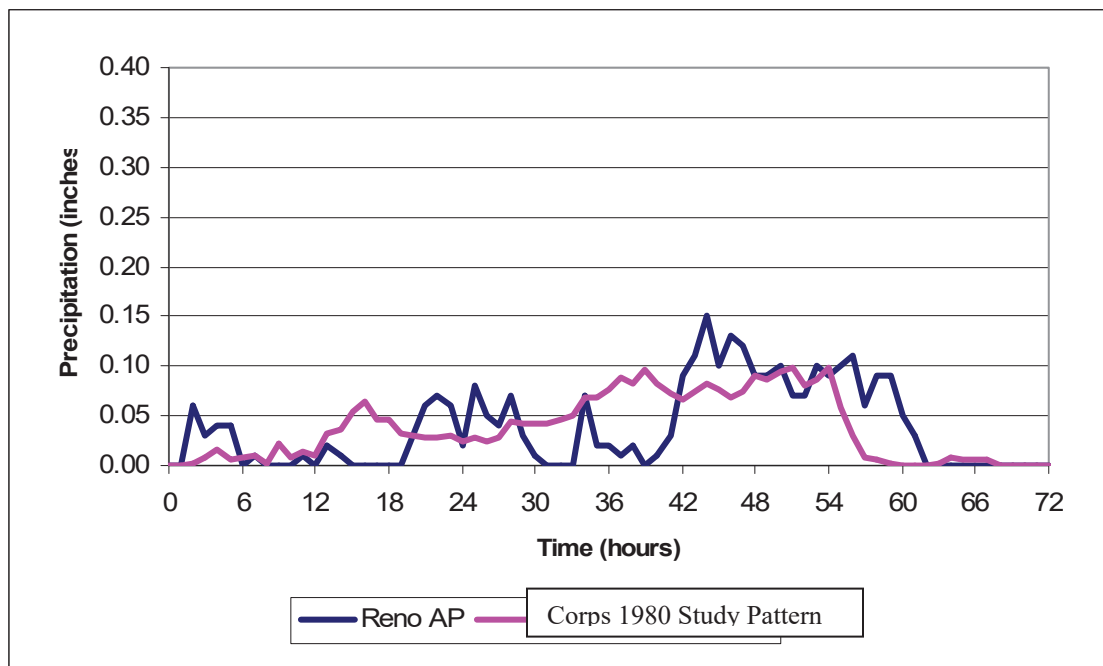
- List data from the USACE in the Truckee Meadows Project study area
- List data from Washoe County
- USGS 30-meter digital elevation models

The topographic layers were merged to form a continuous digital land surface. Subbasins were then automatically generated in a GIS environment using the digital land surface in combination with hydrographic line data. The subbasins were manually refined or adjusted, where necessary, to effectively represent the watersheds and account for drainage features not apparent on the digital land surface. Subbasin boundaries are shown in Chart 3-1.

**3.5 General Rainflood**. Six flood frequencies were evaluated in this study: the 20%, 10%, 2%, 1%, 0.5%, and 0.2% exceedence events. Precipitation data used in the HEC-1 model are based on partial duration NOAA Atlas 14 Depth Duration Precipitation Study which was completed in 2003. This study provides the most current annual, all-season precipitation frequency for this region. Spatial data files (point-rainfall isohyetal maps) were downloaded from the National Weather Service's Precipitation Frequency Data Server as geographic shapefiles. A major storm occurred in late January and early February 1963 on the upper Truckee River watershed. In the 1980 rainfall-runoff study of the Truckee Meadows tributaries (Reference 5), the Corps developed a design storm pattern for Steamboat Creek watershed based on the 1963 event. The Corps 1980 Study pattern could not be attributed to any specific gage, although it is believed to be a hybrid pattern based on a mix of mountain and valley precipitation gages. The 1980 Study precipitation pattern was adopted for this latest analysis. The pattern is illustrated in **Figure 3-1**, and the hourly rainfall distribution is tabulated in **Table 3-1**. The pattern storm was balanced to the 6-, 12-, 24-, and 48-hour depth duration frequencies in NOAA 14. The 3-hour depth found in NOAA 14 could not be used since cloudburst storms dominate this duration in the Reno area. Cloudburst storms have much higher intensities than winter general rains in this region.

Reference 14 was investigated for this reason. It was found that at higher elevations, cloudburst storms and winter storms depths did not differ as much as compared to lowland valleys; therefore, two separate ratios were derived. The analysis found that factors of 0.88 and 0.73 for mountain and valley, respectively, should be used to reduce the 3-hour cloudburst depths to an appropriate depth for general rainfloods. These ratios were derived by comparing “winter seasonal” against “annual” flow frequency depths found in Reference 14. The adjusted 3-hour depth from NOAA 14 was used to balance the design storm. Finally, a 72-hour depth was applied to the design storm pattern using a ratio of 1.04 times the 48-hour depth.

**FIGURE 3-1 – JANUARY THRU FEBRUARY 1963 STORM PRECIPITATION DISTRIBUTION COMPARED WITH CORPS 1980 STUDY DESIGN STORM**



**Reno Airport Precipitation for the 29 January to 1 February 1963 Storm (46 Total Hours of Actual Precipitation).**

Figure 3-1 above shows the Corps 1980 Study design storm, in violet color, compared with the observed rainfall recorded at the Reno Airport, in dark blue, for the 1963 storm event. Both patterns shown below have the same total rainfall, ~ 2.76 inches. Reno Airport had several periods of zero precipitation during the storm, while the design storm has more hours of continuous rainfall, appropriate for a watershed with higher elevation terrain and orographic effects.

**3.6 Depth-Area Reduction Factors.** The precipitation data obtained from NOAA is point precipitation, but since it is being applied to each subbasin area, it is necessary to use depth area reduction factors (DARFs) to reduce the intensity of the precipitation based on storm duration and the area over which it is being applied. The DARFs were applied to the general rainflood and cloudburst precipitation patterns for each subbasin prior to running the snowmelt and runoff models. Depth area reduction factors were taken from the Washoe County Flood Control Master Plan Meteorological Analysis (Hydmet, 1994). The DARF factors for durations 6-hour or

smaller in the Washoe County Report were found to be more applicable to cloudburst events; therefore, they were not used (see discussion on North Truckee Drain DARF). Therefore, only the 12-hour DARF was applied to the maximum 3-, 6-, and 12-hours of rainfall in the design storm.

**TABLE 3-1  
DEPTH AREA REDUCTION FACTORS**

<b>Rainfall Duration</b>	<b>DARF Equation</b>	<b>DARF 1 square mile</b>	<b>DARF 5 square miles</b>	<b>DARF 10 square miles</b>	<b>DARF 100 square miles</b>
12-hour	$.01*(100-2.6*Area^{.46})$	0.97	0.95	0.93	0.78
1-day	$.01*(100-2.0*Area^{.46})$	0.98	0.96	0.94	0.83
2-day	$.01*(100-1.5*Area^{.46})$	0.99	0.97	0.96	0.88
3-day	$.01*(100-1.3*Area^{.46})$	0.99	0.97	0.96	0.89

**3.7 Storm Centerings:** Storms were based on a circular pattern with no maximum storm size. The storm was expanded outward until all subbasins in the HEC-1 model were covered. Two different storm centerings, or bullseyes, of the general rainstorm were developed for Steamboat Creek. One is the Huffaker Hills centering, with the bullseye in subbasin 200. The other centering, for Boynton – Steamboat, has its bullseye at the outlet of subbasin 345. The purpose of the Huffaker Hills centering is to create high peak flows for inflow to the potential Huffaker Hills flood detention facility. The purpose of the Boynton Slough centering is to create high peak flows on the Boynton Slough tributary to Steamboat Creek. Chart 3-2 graphically shows the storm bullseye for the Boynton Slough Centering. Chart 3-3 shows the location of the bullseye for the Huffaker Hill centering. The subbasins that were the bullseye for each centering are listed below.

**STORM CENTERINGS**

<b>Storm Type</b>	<b>Centering Name</b>	<b>Abbreviation</b>	<b>Subbasin on which Storm is Centered</b>	<b>Corresponding Flow Output Node (HEC-1 Concentration Point)</b>
	Boynton Slough	BOY	Intersection of 335, 345, 350, and 352	C360
	Huffaker Hills	HH	200	C130B

**3.8 Snowmelt Analysis.** Snowmelt on the eastern slopes of the Sierra Nevada Mountains can have a significant effect on runoff during winter and spring storm events. For this reason, precipitation for the general rainflood was processed prior to use in the HEC-1 model using a separate computer program to incorporate runoff from rain-on-snow events. The Corps Sacramento District developed the SNOW model for the purpose of calculating runoff for rain-on-snow conditions. The model performs a water-budget analysis to track water in an antecedent snowpack until it is released, using the concept of threshold density to estimate the shrinkage of

the snowpack as rainfall occurs. The SNOW model output for each subbasin is used as the precipitation input for the HEC-1 model. The SNOW model produces rainfall output at the computation timestep that is required for input to the HEC-1 model. As happens in nature, the SNOW model may determine a net loss from the initial precipitation falling on a subbasin due to retention by the snowpack. On the other hand, the computed net rainfall can significantly increase in some subbasins, with the addition of “melt” from a ripe snowpack. Precipitation is computed for different elevation zones according to EM 1110-2-1406 using the following input:

- Basin exposure and canopy cover
- Mean basin precipitation
- Average air temperature and wind velocity per time interval
- Initial snow depth and density
- Threshold density.

Development of precipitation input for the SNOW model included the following steps:

1. 72-hour general rainflood precipitation events were constructed for each flood event and subbasin, as discussed previously in this chapter.
2. In a GIS environment, each subbasin was divided into 1000-foot elevation band areas.
3. For each elevation band in each subbasin, 48-hour precipitation totals were calculated in GIS using the NOAA Atlas 14 spatial data.
4. 72-hour precipitation totals were estimated by multiplying the 48-hour precipitation total by a scaling factor of 1.04.
5. Wind from the Reno Airport gage for the 1963 storm was extrapolated by adding 2 mph in wind speed for each 1000-foot increase in elevation band, such that each elevation band in the watershed was accounted for.
6. The temperature lapse rate was set to -2.7 degrees F per 1000-foot elevation change. The program default is -3 degrees F. This lapse rate was derived by comparing the difference in daily minimum temperature (night-time low) recorded at Bald Mountain and the Reno Airport for the two heaviest days of precipitation during the 1963 storm.

DARFs were applied to the precipitation patterns prior to being input to the SNOW program. The revised general rainflood precipitation events calculated by SNOW for each subbasin were output to DSS, ready for direct input to the HEC-1 model.

The SNOW model requires specification of initial conditions for snow depth and snow water content by elevation zone. To develop this data, an investigation was performed to determine appropriate snowpack parameters for the 1% chance winter flood for watersheds in Washoe County. Historic data indicated that the largest winter floods resulting from high-elevation rain-on-snow occurred from late December through mid-February. All of these events were associated with strong southwest tropical moisture influx ("the pineapple express") with resulting warm temperatures and freezing levels above 10,000 ft. However, watershed response and storm

event snowpack data indicated that there were minimal runoff contributions above 7,000-7,500 feet elevation because existing snowpack melted slower and remained intact after the maximum precipitation period of the storms. Further, existing snowpacks significantly delayed and attenuated runoff due to transitory snowpack liquid water storage and because they impeded vertical and overland flow. The critical elevation zone for flood runoff was identified between 5,000 and 7,000 feet, where snowpacks melted nearly entirely during the flood events. The most critical initial snowpack condition for the design flood would likely occur during the earliest dates (i.e., January 1st), when the least snowpack would be expected. The NRCS database was accessed for all snow survey and snow sensor records in western Nevada and the Tahoe region. Two types of records were available:

- (1) Monthly snow surveys for all sites on or near March 1st and April 1<sup>st</sup>, with more sporadic measurements for February 1st and January 1<sup>st</sup>, and
- (2) Continuous measurements of snow water content for the entire winter and spring using telemetered snow pillows.

**Table 3-2** lists long-term average snow survey data for sites in western Nevada and the Truckee River/ Carson River drainages.

The snowpack data were graphed with site elevation compared to snow water content. Relationships were estimated from the graphs for average snow depth and snow water content for January 1, February 1, March 1 and April 1. **Table 3-3** shows relationships for the Carson Range and relationships for the remainder of the study area. The February 1 relationships were used in the SNOW model as the most likely critical conditions for a winter rain-on-snow event. Modeled subbasins were assigned initial snow conditions for either the Carson Range or Washoe Valley depending upon their location in the watershed and maximum basin elevation.

In the South Area, model simulations indicated that the snowpack in the higher watershed elevations absorbed rainfall, decreasing rather than increasing runoff from these subbasins. As anticipated, the most significant contribution to runoff from rain-on-snow occurred in the 6,500 – 7,500 foot elevation range, where the snowpack was moderate and temperatures warm enough to allow snowmelt to occur. Overall, snowmelt contributed very little to runoff in the upper watersheds.

**TABLE 3-2  
LONG-TERM AVERAGE SNOW SURVEY DATA**

(Note: SWE = Snow Water Equivalent, the amount of water contained within the snowpack)

Station Name	Elevation	Latitude Degrees	Longitude Degrees	January 1st		February 1st		March 1st		April 1st	
				Depth	SWE	Depth	SWE	Depth	SWE	Depth	SWE
<b>Nevada Snow Data (Carson Range)</b>											
Bald Mt.	6720	41.78	-119.62			10	2.8	11	3.3	8	2.7
Big Meadow Snotel	8300	39.45	-119.95		7.4		12.1		18.2		21.2
Big Meadows	8300	39.45	-119.95	32	8.4	45	13.1	60	21.3	65	28.4
Clear Creek	7300	39.12	-119.90	10	1.8	27	7.4	34	10.9	29	10.9
Forty Nine Mtn.	6000	41.57	-119.82			13	3.4	13	3.6	8	2.7
Glenbrook #2	6900	39.08	-119.92	15	3.7	27	7.7	34	10.5	34	11.9
Hays Canyon	6400	41.30	-119.82			11	2.7	11	3.2	6	2
Little Bally Mt - Aerial	6000	41.57	-119.82			10	2.7	11	3.3	7	2.5
Little Valley	6300	39.25	-119.88		1.3	18	4.1	21	7.4	17	6.4
Marlette Lake	8000	39.15	-119.90	40	10.5	46	13.8	58	20.1	57	22.6
Mount Rose	9000	39.35	-119.88	37	11.7	51	17.7	75	28	79	33.6
Sheldon Snotel	5860	41.90	-119.45		0.5		0.7		0.9		0
<b>Nevada Snow Data (117W-119W)</b>											
Buckskin, Lower Snotel	6700	41.75	-117.53		4.1		6.3		8.3		8.2
Buckskin, Upper	7200	41.78	-117.53			18	6	27	9.2	27	10.5
Disaster Peak	6500	41.97	-118.20			25	8.6	37	13.7	28	11.2
Disaster Peak Snotel	6500	41.97	-118.20		4.7		7.3		10.8		8.3
Golconda #2	6560	40.90	-117.57	21	2.4	22	5.7	19	5.7	10	3.7
Granite Peak	7800	41.65	-117.40			38	11.4	38	12.6	44	15.3
Granite Peak Snotel	7800	41.65	-117.53		9.5		14.3		16.9		20.8
Martin Creek	6700	41.68	-117.53		6.1	26	7.1	28	8.8	25	9
Quinn Ridge AM	6300	41.00	-117.00		1.8	8	1.9	7	2.3	3	0.9
<b>California Snow Data (Tahoe Region)</b>											
Heavenly Valley	8800	38.92	-119.92	38	11.2	55	17.6	72	25.1	73	28.1
Freel Bench	7300	38.50	-119.95	20	5.2	29	8.9	33	11.5	27	10.7
Hegans meadow snotel	8000	38.85	-119.93		6.4		10.8		15.2		15.8
Boca # 2	5900	39.38	-120.09			19	5.2	20	6.8	11	4.5
Tahoe City Cross Snotel	6750	39.17	-120.15		6.1		10.3		13.4		10.4
Brockway Summit	7100	39.26	-120.07	22	6.5	39	12.1	45	15.6	43	16
Independence Creek	6500	39.49	-120.29	15	4.4	29	8.5	35	11.7	33	12.9
Fallen Leaf Snotel	6300	38.93	-120.05		3		5.1		6.4		2.9

**TABLE 3-3**  
**DESIGN FLOOD INITIAL SNOWPACK CONDITIONS**  
 (Note: SWE = Snow Water Equivalent, the amount of water contained within the snowpack)

<b>CARSON RANGE, WASHOE COUNTY, NEVADA</b>								
<b>Elev. (feet)</b>	<b>Average Snowpack Conditions (inches)</b>							
	<b>Jan 1st</b>		<b>Feb 1st</b>		<b>Mar 1st</b>		<b>Apr 1st</b>	
	Depth	SWE	Depth	SWE	Depth	SWE	Depth	SWE
5500	2	0.4	5	1	5	1	2	0.5
6500	11	2.5	17	3.5	19	5	15	5
7500	21	5	33	8	37	11	32	12
8500	32	9	48	14	61	21	60	23
9500	46	14	69	22	90	35	97	42
<b>WASHOE VALLEY EXCLUDING CARSON RANGE</b>								
<b>Elev. (feet)</b>	<b>Average Snowpack Conditions (inches)</b>							
	<b>Jan 1st</b>		<b>Feb 1st</b>		<b>Mar 1st</b>		<b>Apr 1st</b>	
	Depth	SWE	Depth	SWE	Depth	SWE	Depth	SWE
5500	0	0	0	0	0	0	0	0
6500	2	0.4	2	0.4	2	0.4	2	0.4
7500	11	2.5	12	3	18	4	20	5
8500	23	5.5	27	6.5	35	8	32	9
9500	42	12	42	12	60	15	64	20

### 3.9 Soil Loss and Land Use Parameters.

**3.9.1 Soil Type, Land Use, and Cover.** Digital soil data were downloaded from an NRCS website. Land use data, provided by Washoe County, covered the entire study area. The land use data included numerous development categories, which were consolidated into eleven cover categories for the purpose of this study. Land uses were altered to reflect current development in the study area as indicated by aerial photography from 2000. The consolidated land use categories, with associated percent impervious, initial loss, and vegetation cover, are listed in **Table 3-4**. The derived XKSAT values (constant loss rate) for some subbasins were adjusted in later model calibration runs. A discussion of this calibration is discussed in Section 3.12, Model Verification.

Basin n values, used in the calculation of lag (Los Angeles S-graph method), are also shown in **Table 3-4**. These basin n values were adopted from the *Sacramento City/County Drainage Manual*, 1994 (Reference 31). Acreage of each land use was extracted for each subbasin using ArcView. Composite values for initial loss, percent impervious, vegetation cover, and basin n value were determined for each subbasin using the ArcView land use data.

**TABLE 3-4  
LAND USE CATEGORIES AND CORRESPONDING DATA**

Description of Land Use	Percent Imper-Vious	N value	Initial Loss (IA)	Vegetation Cover
			(inches)	in %
Pasture - heavily grazed	5%	0.075	0.30	30%
Parks - fair grass cover	5%	0.070	0.30	60%
Parks – good grass cover	5%	0.070	0.30	80%
Major Highways	90%	0.030	0.10	0%
Commercial – Business	85%	0.031	0.10	75%
Industrial	72%	0.032	0.12	70%
Residential - 1/8 acre lots	65%	0.037	0.20	65%
Residential - 1/4 acre lots	38%	0.042	0.25	50%
Residential - 1/2 acre lots	25%	0.060	0.25	50%
Residential - 1 acre lots	20%	0.073	0.30	50%
Water Bodies	100%	-	0.00	0%

**3.9.2 Green & Ampt Loss Parameters.** The Green & Ampt loss method uses the following parameters in the HEC-1 model:

IA – initial loss in inches, primarily a function of land-use and surface cover (see Table 3-4). Zero rainfall excess is generated until the initial loss is satisfied.

DTHETA – volumetric measure of the soil moisture storage capacity available at the start of the storm (dry, normal, or saturated soil conditions).

PSIF – Wetting front suction in inches.

XKSAT – hydraulic conductivity at natural saturation in inches per hour.

RTIMP – percent impervious (see Table 3-4).

The parameters, DTHETA, PSIF, and XKSAT, are functions of soil characteristics, ground surface characteristics, and land management practices. The primary soil surface characteristics are vegetation canopy cover, ground cover, and soil crusting. Land management practices effect soil porosity. The parameters for bare ground are dependent on soil type, as tabulated in Table

3-5, below. The Corps used the DTHETA values for a normal antecedent soil moisture condition.

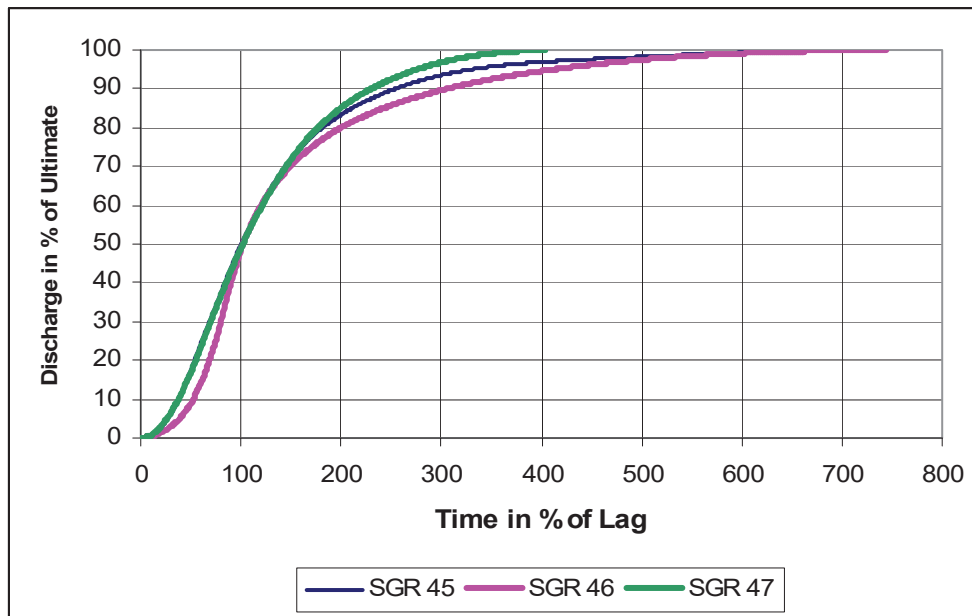
**TABLE 3-5  
GREEN & AMPT LOSS RATE PARAMETER VALUES FOR BARE  
GROUND**

Soil Classification	XKSAT (in/hr)	PSIF (in.)	DTHETA		
			Dry	Normal	Saturated
A1 loamy sand & sand	1.20	2.4	0.35	0.30	0.00
A2 sandy loam	0.40	4.3	0.35	0.25	0.00
B1 loam	0.25	3.5	0.35	0.25	0.00
B2 silty loam	0.15	6.6	0.40	0.25	0.00
B3 silt	0.10	7.5	0.35	0.15	0.00
C sandy clay loam	0.06	8.6	0.25	0.15	0.00
D1 clay loam	0.04	8.2	0.25	0.15	0.00
D2 silty clay loam	0.04	10.8	0.30	0.15	0.00
D3 sandy clay	0.02	9.4	0.20	0.10	0.00
D4 silty clay	0.02	11.5	0.20	0.10	0.00
D5 clay & rocky outcrop	0.01	12.4	0.15	0.05	0.00

The drainages in Washoe County are a mixture of several different soil types, so composite values for the Green & Ampt parameters need to be determined for each subbasin. The procedure to determine the subbasin composite value of XKSAT is to average the area-weighted logarithms of the XKSAT values of each soil type in that subbasin. The corresponding subbasin composite values for PSIF and DTHETA parameters are based on the subbasin XKSAT as determined from a graph (Figure 4.3 in the Maricopa County Design Manual). A final adjustment of the composite XKSAT is made based on subbasin percent vegetation cover.

**3.10 Hydrograph Development.** The computer program SACPRE, a pre-processor to HEC-1, was used to develop unit hydrographs using the Corps S-graph method. An S-graph is not a unit hydrograph; it represents the hydrologic response of a basin to a rainfall event of infinite duration. The curve has a rising limb with a characteristic S-shape that eventually reaches a constant discharge rate equal to the constant excess rainfall rate multiplied by the area of the basin. An S-graph (**Figure 3-2**) is constructed from a series of unit hydrographs of length T by the process of successive lagging by T and summing the discharge values (ordinates). The beauty of the S-graph method is that one can derive unit hydrographs for events of any desired duration from them. The Corps Sacramento District developed three S-curves a mountain general rain S-graph (SGR 45), a mountain cloudburst S-graph (SGR 47) and a valley general rain and cloudburst S-graph (SGR 46), specifically for use in the Truckee Meadows area (**Table 3-6**). The S-graphs were developed for the 1980 Truckee River Study (Ref. 5).

**FIGURE 3-2 – TRUCKEE MEADOWS AREA S-GRAPHS**



The computer program SACPRE, originally developed by the Corps Los Angeles District as LAPRE-1, was used as a pre-processor to HEC-1 to produce unit hydrographs for each subbasin based on the drainage area, basin lag, and chosen S-graph.

**TABLE 3-6  
S-CURVES FOR THE TRUCKEE MEADOWS AREA**

SACPRE Description	Curve No.
Truckee Meadows: Mountain General Rain	45
Truckee Meadows: Valley General Rain & Cloudburst	46
Truckee Meadows: Mountain Cloudburst	47

**3.11 Routing Parameters.** Once rainfall has turned into runoff and flowed to the outlet point for a subbasin, it is necessary to combine this flow with upstream subbasin flows. The combined flow hydrograph is then routed through stream and drainage channels to the next subbasin outlet. There are a number of hydrologic routing techniques available in the HEC-1 program, such as Muskingum, Modified Puls, Muskingum-Cunge, and Kinematic Wave. While any of these techniques may be used, routing techniques were generally applied to this study as described below, based on a number of sources:

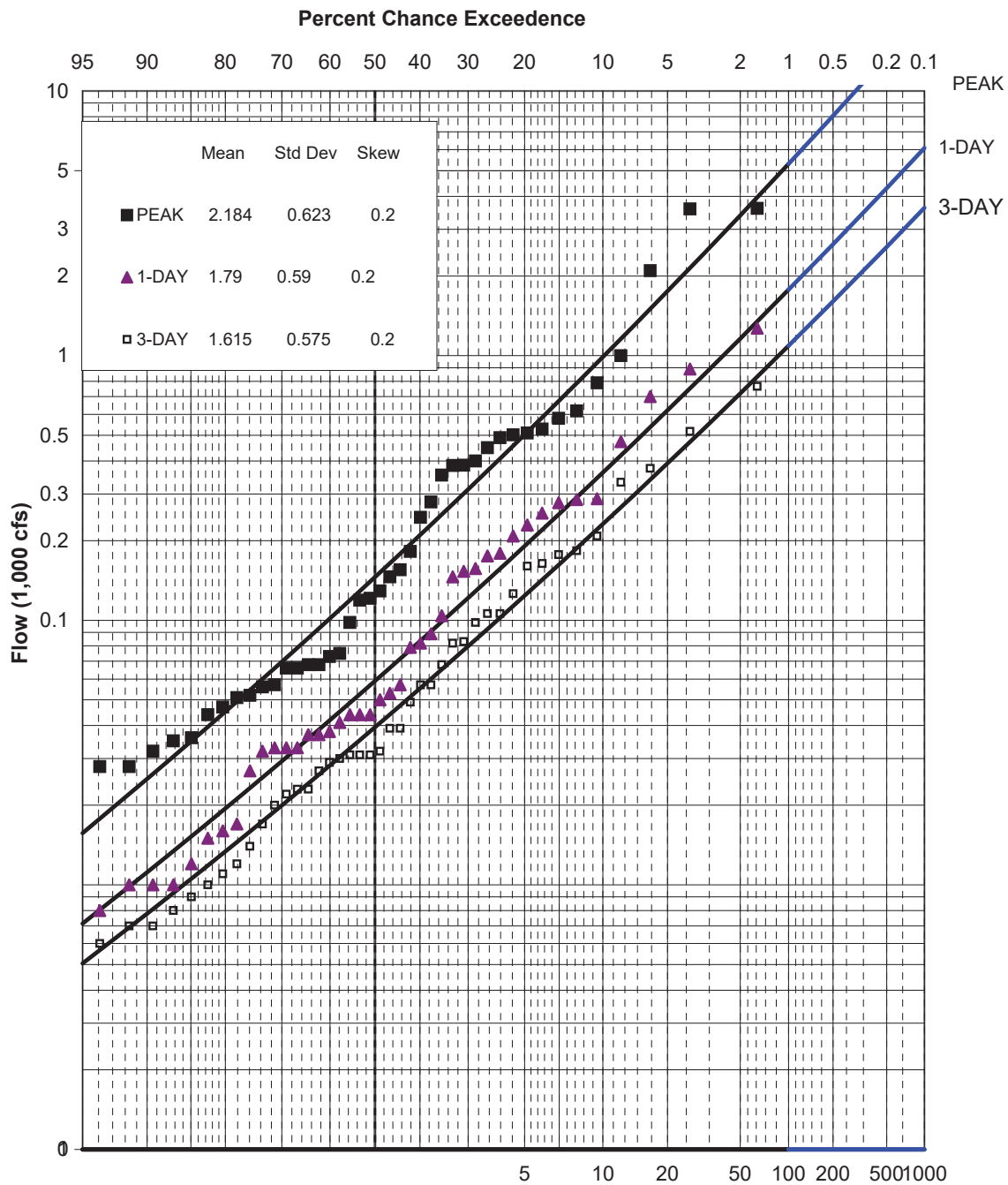
- 1983 Corps HEC-1 model (Muskingum and Modified Puls)
- Information from local government and their consultants (Muskingum and Muskingum-

Cunge)

- Determination of travel time and attenuation developed for this study (Muskingum)
- Channel routing information from Corps HEC-RAS models of Steamboat Creek

**3.12 Model Verification.** An attempt was made to calibrate the HEC-1 model to the 2005 flood event, but the results were not conclusive due to the fact that the runoff at the Geiger Grade and Short Lane locations spread out laterally into the overbanks and away from the main channel profile, potentially causing the flow to run around the gage and making the flow measurements too low. It is also notable that the peak flow upstream at Steamboat was 3,600 cfs while the peak flows recorded farther downstream at Geiger Grade and Short Lane were around 3,000 cfs, despite a significant increase in drainage area. To calibrate the HEC-1 above the Steamboat Creek at Steamboat gaging station, a 1% chance design storm was centered above the gage. The XKSAT in the HEC-1 model was adjusted uniformly by a ratio until the resulting hydrographs had a good match with the 1% chance exceedence peak flow frequency curve. To achieve this fit, the original XKSAT parameters in each subbasin were reduced to 29% of their original values. Figure 3-3 shows the Steamboat at Steamboat Creek frequency curve. The gage record is 44 years in length. The statistics were derived by running the period of record data through the Regional Frequency Program (RegFreq).

During the 1986 flood event, the peak flow at Steamboat Creek at Steamboat was recorded as 3,600 cfs. The estimated peak flow at the Steamboat Creek at Short Lane (site of the proposed Huffaker Dam) was 5,600 cfs. This results in a downstream to upstream ratio of approximately 1.6 times the upper gage runoff. The 1986 storm event provides a good indication of the runoff potential from the lower part of the watershed. The XKSAT values for the subbasins between the Steamboat Creek at Steamboat gage and the Huffaker damsite were adjusted until the upstream to downstream peak flow attained a ratio of 1.62 for the 1% chance, storm centered above Huffaker Dam. This caused the XKSAT values to be adjusted to 36% of their original values that were based on soil characteristics. As no other information was available, the subbasins that contribute runoff downstream of the Huffaker Dam site were calibrated for the December 31<sup>st</sup>, 2005 event based on the additional flow runoff that occurred between the Steamboat Creek at Short Lane and Steamboat Creek at Clean Water Way gage. This resulted in using 90% of the original XKSAT values for the subbasins downstream of the Huffaker Dam site.



**NOTES:**

1. WY1962 -2005, 44 years = record length
2. Median plotting positions
3. Drainage area 123 sq. mi.
4. USGS Gage No. 10349300
5. Snowmelt & rainflood combined, stats similar to rainflood only analysis
6. FFA runs determined no low outliers
7. Adopted Stats from Regfreq output

FLOW FREQUENCY CURVES
<b>STEAMBOAT CRK AT STEAMBOAT, NV TRUCKEE RIVER</b>
WATER YEARS 1962-2005
U.S ARMY CORPS OF ENGINEERS SACRAMENTO DISTRICT

**TABLE 3-7  
COMPARISON OF STEAMBOAT AND SHORT LANE FLOWS FOR 1986 AND 2006  
FLOODS AND THE 1% GENERAL RAINSTORM HUFFAKER HILLS CENTERING**

Event	"At Steamboat" flows (cfs)		Short Lane Computed Flows (cfs)	Flow Ratio: Short Lane to "at Steamboat"
	Computed Flows (cfs)	Frequency Curve Flows (cfs)		
1% General Rain Huffaker Hills Centering				
1% peak flow	3,990	5,310	6,450	1.62
1% 1-day volume	1,710	1,780	3,500	2.05
1% 3-day volume	660	1,090	1,410	2.14
Feb 1986 flood	"At Steamboat"		Short Lane	
peak flow (obs. or est.)	3,600		5,600	1.56
Dec 2005 flood	"At Steamboat"		Short Lane	
peak flow (observed)	3,600		3,180	0.88
1-day volume (observed)	860		1,450	1.69
2-day volume (observed)	480		930	1.94

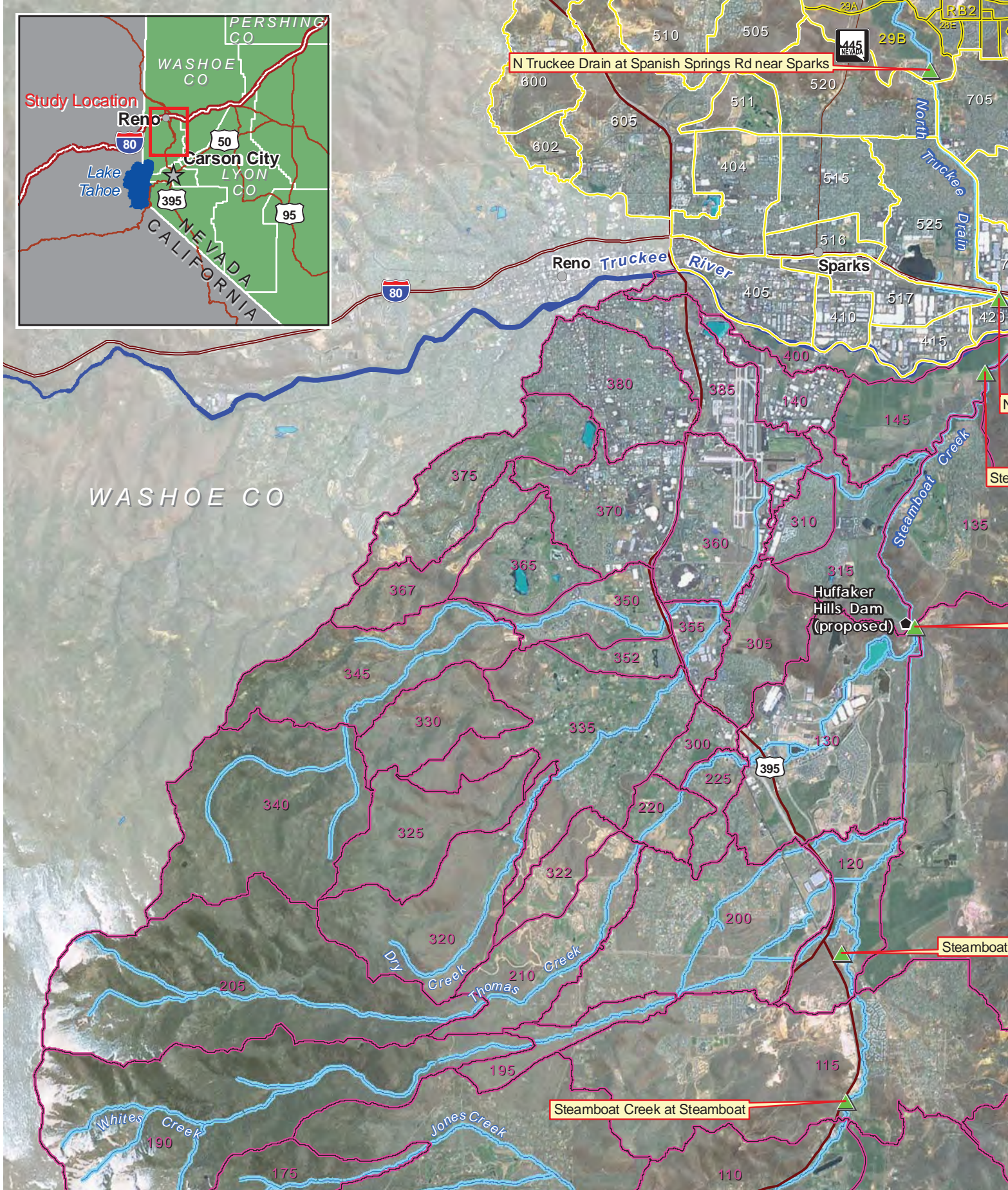
Note: Volumes are in day-cfs

**3.13 Summary of Results.** Table 3-8 summarizes simulated peak flows (in cubic feet per second) for the general rain flood series for the Boynton Slough and the Huffaker Hills centerings. The model assumes that the hydrographs on Steamboat Creek remain in-channel as they move downstream.

**TABLE 3-8  
SUMMARY OF SIMULATED PEAK FLOWS  
FOR THE GENERAL RAIN FLOOD SERIES**

Location	PEAK FLOWS (CFS)					
	Exceedence Frequency					
	20%	10%	2%	1%	0.50%	0.20%
BOYNTON SLOUGH CENTERING						
Steamboat Creek:						
At Steamboat	300	700	2,100	2,690	3,710	5,160
At Short Lane	740	1,230	3,720	4,500	5,600	7,600
At Mouth	1,730	2,180	5,170	6,480	8,280	10,500
Boynton Sl. at mouth	1,170	1,370	2,180	2,660	3,580	4,770
HUFFAKER HILLS CENTERING						
Steamboat Creek:						
At Steamboat	510	1,080	3,260	3,990	5,230	7,020
At Short Lane	980	1,780	5,200	6,450	8,260	10,700
At Mouth	1,770	2,540	6,340	7,710	9,950	13,100
Boynton Sl. at mouth	1,010	1,190	1,890	2,300	3,120	4,200

Note: Steamboat Creek at Short Lane is the same location as Huffaker Hills Damsite



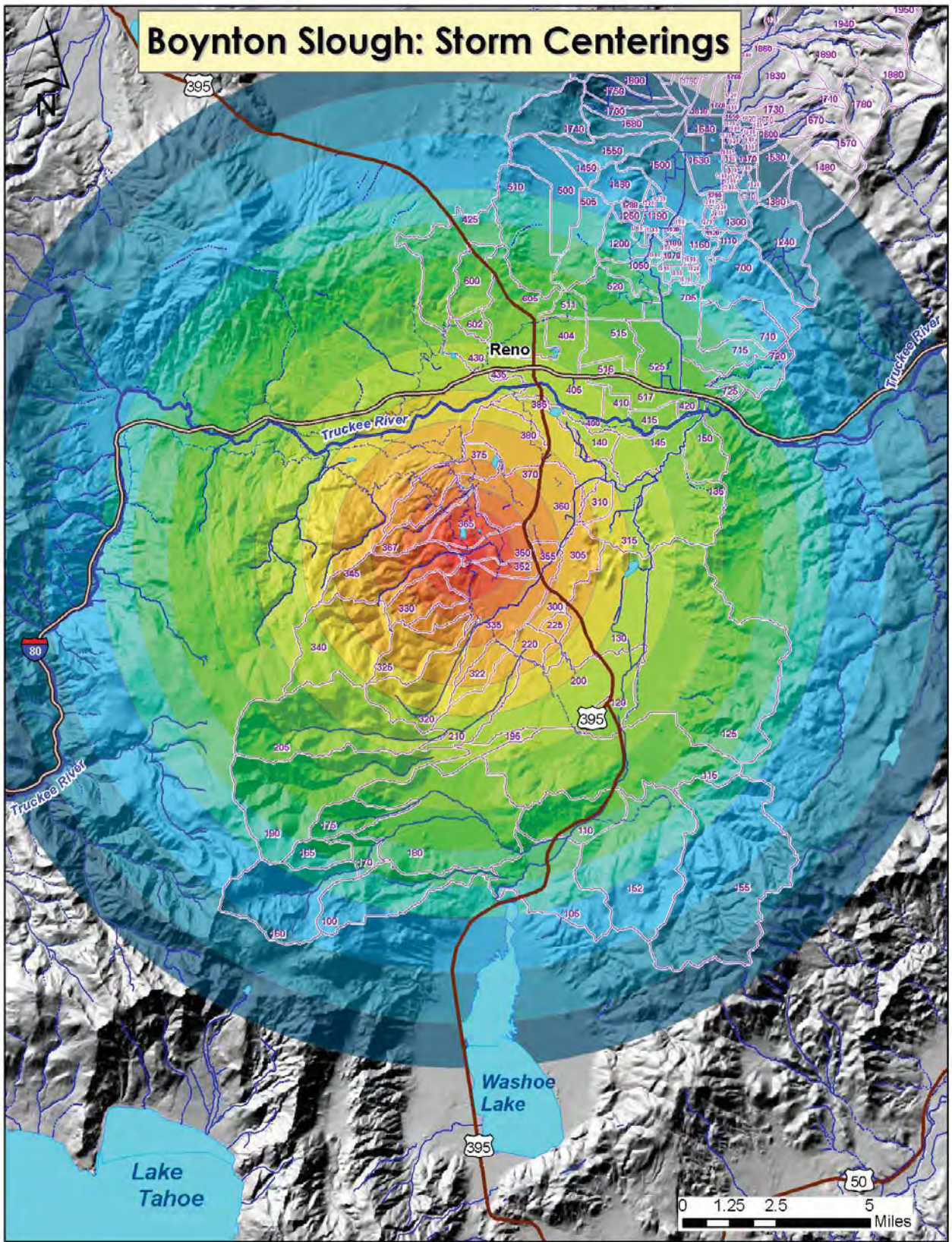


Chart 3-2

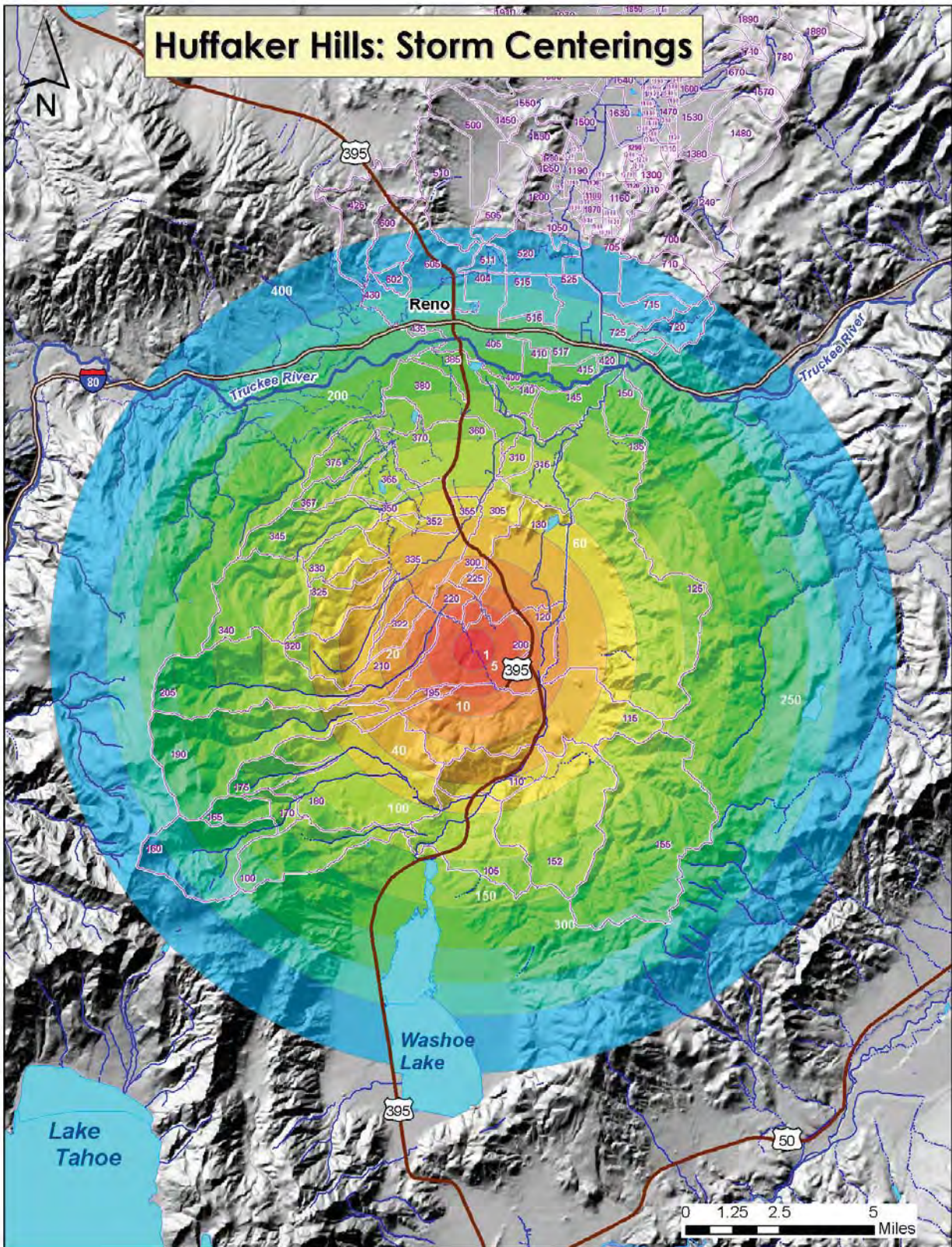


Chart 3-3



## 4.0 Probable Maximum Flood for Huffaker Dam

**4.1 Introduction.** The proposed Huffaker Dam was designed to decrease the flow downstream of Vista for rare events. Following Corps criteria, the dam needs a spillway that can safely pass a Probable Maximum Flood (PMF) without overtopping. Another goal is to minimize flooding behind the dam for rare floods approaching the PMF event. This section describes the development of the probable maximum flood for this dam. Due to time constraints, a few shortcuts were taken in following HMR 49 guidelines; however, it is believed that the product developed is sufficient for a Feasibility Level analysis of cost and performance. If this study should go into the P.E.D. phase with Huffaker Dam as part of the approved project, the PMF flood analysis should be refined to completely follow HMR 49 guidelines. The actual modeling and routing of the PMF through the proposed dam structure was accomplished by Hydraulic Design Section. Please see Attachment B-Hydraulics for details on the routing and resulting water surface elevations. The criteria for this region is covered in Hydrometeorological Report No. 49 (HMR 49) dated September 1997. It applies to the Colorado River and Great Basin drainages. The entire watershed above Huffaker Hills, including Washoe Lake is 194 square miles. The drainage area above the Washoe Lake outlet is 84 square miles. Washoe Lake itself covers approximately 25% of the watershed above the outlet of the dam. The outlet to Washoe Lake is a small weir with optional flashboards, which provides an enormous amount of surcharge space in the lake due to the minimal outflow capacity. Outflow capacity is directly related to the depth of the water (head) over the weir. Two separate HEC-1 models were used to model the watershed above and below Washoe Lake. Chart 3-1 shows a basin map, with the location for the proposed Huffaker Hills Dam.

**4.2 Probable Maximum Precipitation (PMP).** A general rain event was used for this watershed, since that is the type of event that will create the largest PMF hydrograph. Based on guidance in HMR 49, the general rain storm PMP is 72 hours long and has two components, orographic (precipitation affected by terrain) and convergence (precipitation from atmospheric processes not affected by terrain). The PMP was computed for each component for the month of interest, January. The storm has two components. Each component (orographic and convergence precipitation) was adjusted for drainage area and intermediate time durations (6-, 12-, 18-, 24-, 48-, and 72-hours). When completed, the separate components were added together to create a winter general-storm PMP. The resulting Washoe basin 72-hour storm total is 13.51 inches of precipitation, while the lower Steamboat watershed between the outlet of Washoe Lake and the Huffaker Dam site is 12.72 inches. The table below provides the values for various depth-durations. Figures 4-1 and 4-2 show the hyetograph used for the upper and lower portions of the watershed, based on HMR 49 guidelines.

Table 4-1. PMF Storm Precipitation in HEC-1 Models

HEC-1 Model	Incremental Increase in Precipitation Per Duration						
	6-hr	12-hr	18-hr	24-hr	48-hr	72-hr	Storm Total
Washoe Lake	3.25	2.11	1.50	1.22	3.51	1.91	13.51
Lower Steamboat Crk	3.07	1.99	1.41	1.14	3.29	1.81	12.72

The goal of this effort was to produce a conservatively large PMF that would not likely be exceeded. A full analysis of storm distribution patterns, including those for different seasons of the year, was not completed due to time constraints; however, it is believed that the January PMF will result in the highest flows.

Normally, precipitation is spatially distributed according to elevation and orographics. Instead, one hyetograph was developed for all subbasins in the Washoe Lake HEC-1 model, and another hyetograph was developed for all subbasins in the lower Steamboat Creek model. The storm depth of the PMP hyetograph used in this analysis was designed to be an approximate average for the whole watershed area being modeled.

Two hyetograph patterns were tested, although ultimately, only one was used. One test used the pattern for the all-season PMF (winter months) for French Meadows Dam on the middle fork of American River watershed. The watershed ranges from 5,200 to 9,000 feet elevation, whereas Steamboat Creek ranges from 4,400 to a little over 10,000 feet. The watershed is directly west of the Truckee River watershed, on the west slope of the Sierras. The resulting PMF had a smaller peak and volume than the final adopted one. The adopted pattern was based on Standard Project Flood found in the report Standard Project Rain-flood Criteria, Sacramento-San Joaquin Valley, California, 1972. The most intense 72-hours of this 96-hour storm were utilized for the pattern. The final hyetographs used for each HEC-1 model are shown in Figures 4-1 and 4-2. Since the largest historic floods on the Truckee River are the result of pacific storms moving east over the Sierra Mountains, the adoption of this pattern seemed reasonable.

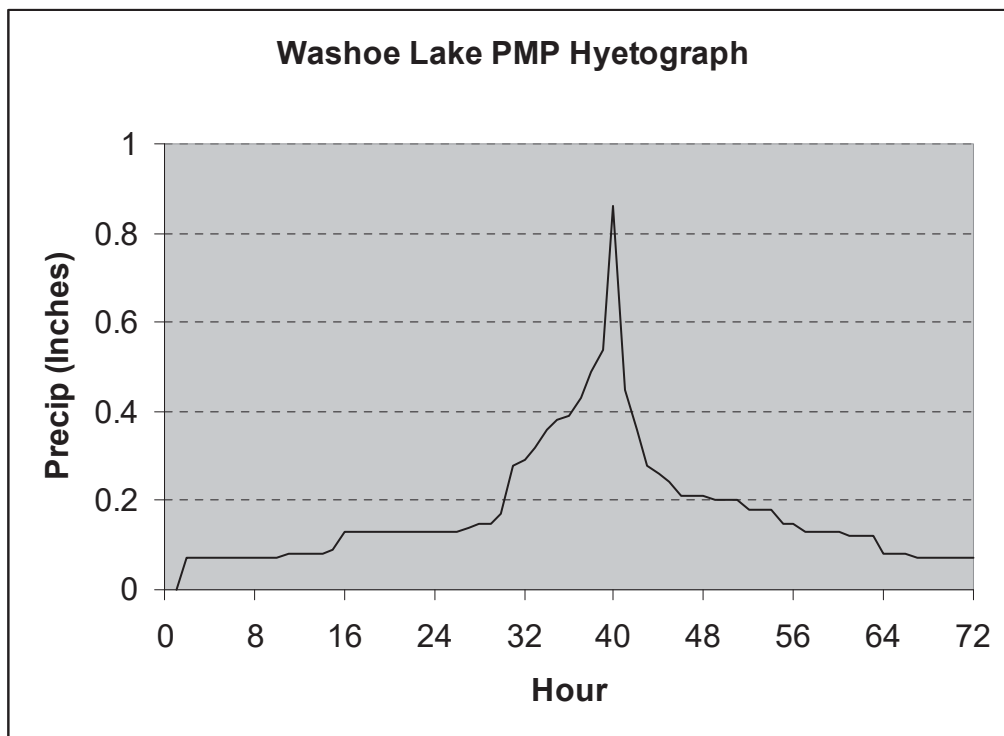


Figure 4-1. Washoe Lake PMP Hyetograph

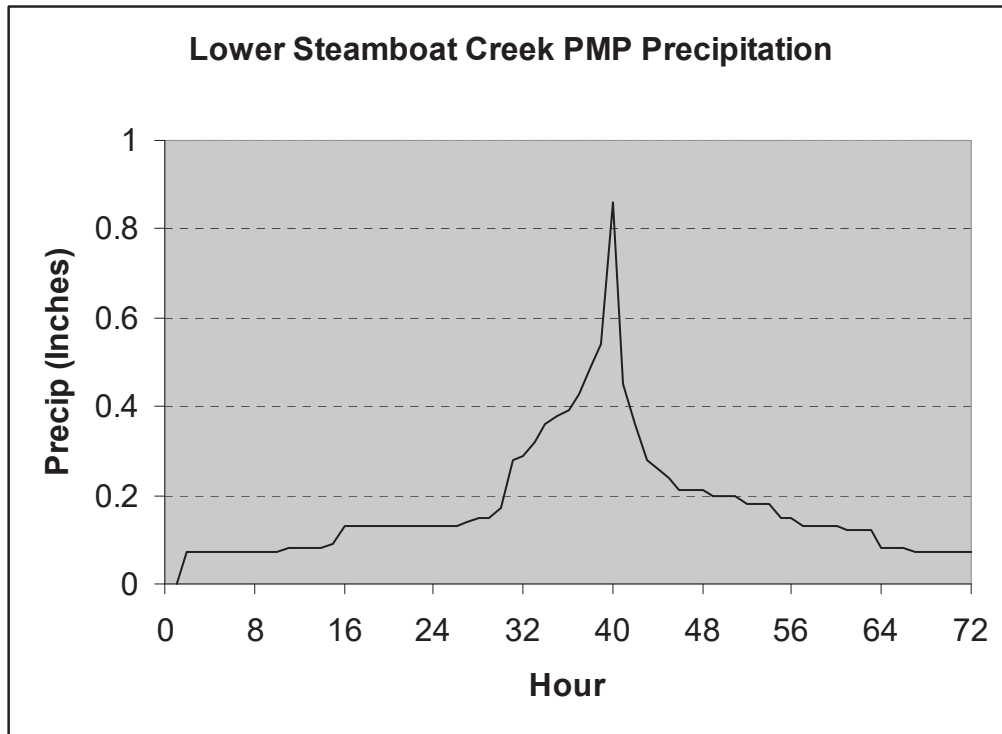


Figure 4-2. Lower Steamboat Creek PMP Hyetograph

**4.3 Snowmelt Contribution.** Snowmelt models had previously been developed for lower Steamboat Creek, below Washoe Lake, as discussed in Part IV, Section 3.8 of this report. The models include an initial snowpack, one for the higher elevation east-facing subbasins of the Carson Range and one for the west-facing subbasins of the Virginia Range. This snowmelt model, including initial snowpack conditions, was adopted to simulate snowpack melt during the PMF on lower Steamboat Creek. The initial snowpack conditions represent an average snowpack for the basin, based on an average of all 1<sup>st</sup> of February snowpacks for the period of record of the snow gages. To save time, the winds and temperatures used in the SNOW Model for the Steamboat Creek PMF were adopted from the PMF analysis for French Meadows Dam on the west slope of the Sierras. Both higher winds and higher temperatures increase the amount of snowmelt. The winds and temperatures used in this analysis are expected to be conservatively high for Steamboat Creek.

There is currently no snowmelt model for the watershed above Washoe Lake and time was not available to develop one. To account for potential snowmelt, the soil loss rates for the pervious areas in the Washoe Lake HEC-1 model were modified to 0.02 inches per hour constant loss, as compared to 0.05 inches per hour for lower Steamboat Creek. Washoe Lake itself was considered impervious (zero losses) since it is covered with water. A full 25% of the Washoe Lake watershed is covered by water. The computed PMF outflow from Washoe Lake for this analysis was found to be only 340 cfs, even though the inflow hydrograph had a peak of 29,000 cfs and a 3-day volume of over 54,000 ac-feet. The outflow from Washoe Lake is rather insensitive to the volume of inflow that occurs during a 72-hour period. This is due to the huge

storage capacity on Lake Washoe and its small outflow capacity. As such, not using a snowmelt model for this part of the watershed is not much of a concern.

**4.4 Washoe Lake Basin HEC-1 Model.** The 84-square-mile Washoe basin was treated as a single subbasin for the computer runoff model. Franktown Creek, the longest tributary to Washoe Lake, was used for the longest flow path into Washoe Lake. The average mountain Truckee Meadows general rain S-curve (SGR45.dat) was used to create the basin unit hydrograph.

A storage-elevation relationship was presented in the USGS gaging station description for stage gage 10348700, Washoe Lake near Carson City, NV. Using GIS software and topographic maps, the stage-storage relationship was extrapolated to an elevation higher than the road surface of Highway 395, which runs along the west side of the lake. The outflow from Washoe Lake moves west through a culvert under Highway 395, before turning southeast down Steamboat Creek. The outflow from the lake is constrained by the culvert capacity until the lake overtops Highway 395. Once this happens, the highway itself becomes a weir, controlling outflow from the lake. A hydraulic stage-discharge rating for the Highway 395 embankment was created by Hydraulic Design Section and input into the HEC-1 model for Washoe Lake. The starting water surface elevation at Washoe Lake for the PMF simulation was chosen to be the highest water surface ever recorded during the period of record at the lake stage gage. It is 5032.62 feet, which occurred during the January 1997 flood. During the HEC-1 simulation of the PMF, the Highway embankment was barely overtopped, which is why the peak outflow only reached 340 cfs!

**4.5 Huffaker Hills HEC-1 Model.** The Washoe basin HEC-1 model outflow hydrograph with a peak of 340 cfs was input as a dssfile into subbasin 105 of the Lower Steamboat Creek HEC-1 model. As mentioned earlier, the SNOW program was used to model snowmelt contribution to Steamboat Creek from the east-facing subbasins of the Carson Range and west-facing subbasins of the Virginia Range. The resulting net precipitation from the SNOW model became the precipitation input into the Steamboat Creek HEC-1 model. The rainfall excess in the HEC-1 model plus the outflow from Washoe Lake were combined to make the PMF inflow hydrograph to the proposed Huffaker Dam site. Precipitation and runoff results are tabulated below.

Table 4-2. Huffaker Hills PMF Summary of Results

Watershed	drainage area (sq.mi.)	PMP (inches)	Peak Flow (cfs)	72-Hour Volume (ac-ft)
Washoe basin inflow (25% impervious)	83.8	13.44	29,300	54,700
Washoe basin outflow			340	2,000
Steamboat Cr. At Huffaker Hills Damsite including Washoe Lake Outflow	194.55 (including Washoe Lake)	12.72	28,000	57,600

## 5.0 Hidden Valley Interior Drainage

**5.1 Introduction.** Hidden Valley is a small community located on the eastern side of Steamboat Creek near its confluence with Boynton Slough. During the January 1997 flood event, some homes in this community were flooded by a combination of runoff from the Truckee River and Steamboat Creek. The Corps proposes to build a combination of floodwalls and levees along Steamboat Creek to protect this community from backwater inundation from the Truckee River. These structures could potentially cause an interior flooding problem by blocking the normal drainage path of runoff into Steamboat Creek; therefore, interior hydrology needs to be analyzed. A watershed map of Hidden Valley is shown on Chart 5-1. Both general rain and cloudburst events were analyzed. Frequencies evaluated were the 10%, 2%, 1%, 0.5%, and 0.2% chance events. The Corps obtained an HEC-HMS model that was built in late 2007 by the Reno office of the engineering firm HDR, Inc. The Corps obtained and modified the model for its own use as follows: 1) the soil loss rate method was changed from SCS Curve Numbers to Green & Ampt, 2) the design storm was modified to use NOAA 14 precipitation frequency, 3) Subbasin 9 of the original model was split into two subbasins (9a and 9b) to match existing conditions. Subbasin 9a flows into subbasin hv20. Subbasin hv9b flows into subbasin hv13.

**5.2 General Rain Design Storm.** A 72-hour General Rain event was developed. NOAA 14 precipitation frequency values for durations for the 3-, 6-, 12-, 24-, and 48-hour depths were extracted from the NOAA 14 website for the centroid of each subbasin. The 72-hour depth was estimated since NOAA 14 does not provide values for this duration. It was estimated as the 48-hour depth times 1.04, which is based on the ratio of the 72-hour depth to the 48-hour depth at the Reno Airport for the 1% chance event (Reference 14). In the Reno area, cloudburst storms that occur in the summer dominate the maximum, short duration rainfall depths recorded at precipitation gages. Since a general rain is being modeled, the 3-hour depth must be reduced to be realistic of a winter type storm. The 3-hour depth was multiplied by 0.73, derived by taking the 1% chance, 3-hour depth from the “winter only” frequency curve divided by the 3-hour depth for the annual frequency curve for the Reno Airport gage. NOAA 14 is an annual analysis and is therefore biased towards cloudburst events for short durations. A 1963 hourly storm pattern used in the 1980 Study of this area was balanced to the 3-, 6-, 24-, 48-, and 72-hour depths derived above. The rainfall runoff model was run on a 1-minute time step calculation.

Three areas were analyzed for interior drainage in this watershed. The first was the outlet for subbasins hv12a, b, and c. The Corps later decided to remove the proposed levee along this portion of Hidden Valley and, as such, the hydrograph for this area was not used in the interior flooding analysis. The second area was the outlet for hv13 and the third area was the outlets for subbasins hv19 and hv21. To maximize runoff into each of these areas, the entire Hidden Valley watershed was broken up into three zones and a storm developed for each one. Areal reduction factors found in Reference 14 were applied based on the area of each zone. In other words, a storm was centered over each zone to maximize runoff. The table below shows the subbasins found in each zone.

Table 5-1: Hidden Valley Watershed Storm Centering Zones

Zone	Subbasins
South zone	Hv6, hv7, hv10, hv11, hv12a, hv12b, hv12c
Central zone	Hv1, hv2, hv9b, hv13
North zone	Hv3, hv4, hv5, hv9a, hv14, hv15, hv16, hv17, hv18, hv19, hv20, hv21, hv22

**5.3 HMS Model Components.** The HMS model runs on a 1-minute time step increment. It uses SCS unit hydrographs for the rainfall to runoff transformation. The routings between basins were done using Muskingum-Cunge. Green and Ampt loss rates were based on soil types found in each subbasin. The Green & Ampt parameters were based on the Maricopa County, AZ design manual (see ref. 22). The results of the model runs are shown below for the 1% chance event storm.

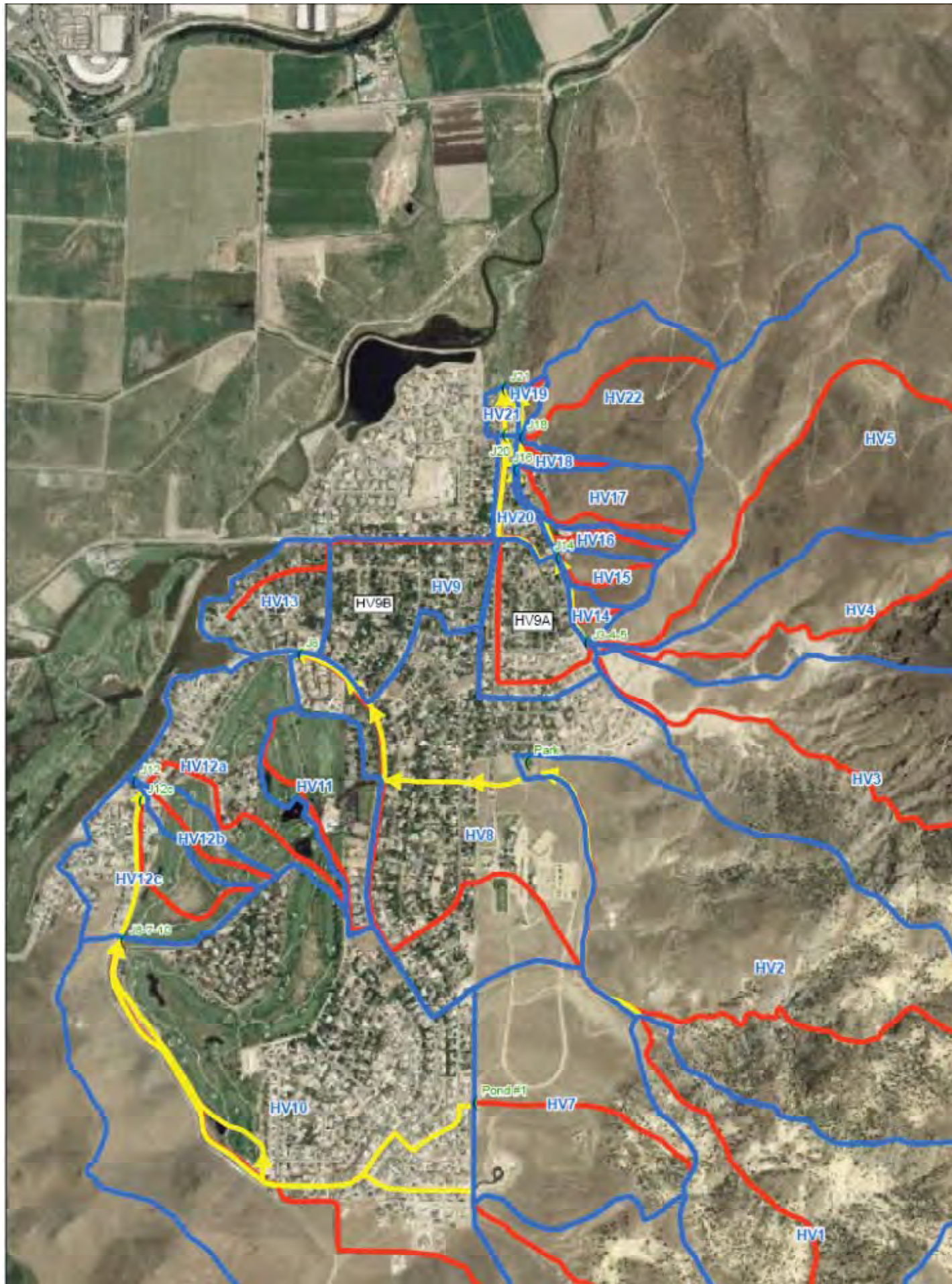
Table 5-2 . General Rain Model Results

Location	1% Chance Peak Flow in CFS
Outlet of HV12 (J12)	51
Outlet of HV13 (J13)	30
Outlet of HV19 (J19)	46
Outlet of HV21 (J21)	20

**5.4 Cloudburst Design Storm.** A cloudburst storm lasting 3 hours was created for the HMS model. NOAA 14 depth duration frequency values for the 15-, 20-, 60-, 120- and 180-minute durations were obtained for each subbasin in the model. The storm depths were applied to a triangular-shaped, balanced storm whereby the highest intensity value is placed in the center of the 3-hour period, the 2<sup>nd</sup> most intense to the right of the center, the third most intense to the left of center, and so on. The model was run on a 1-minute time step calculation. The only change in the HMS model from general rain to cloudburst was the design storm. The results of the modeling were as follows:

Table 5-3. Cloudburst Model Results

Location	1% Chance Peak Flow in CFS
Outlet of HV12 (J12)	300
Outlet of HV13 (J13)	200
Outlet of HV19 (J19)	280
Outlet of HV21 (J21)	160



## Meadows Reach near Airport Interior Drainage

Levees will be placed on the south side of the Truckee River in the Truckee Meadows Reach near the Reno structures could inhibit interior drainage, a hydrologic analysis was needed. Chart 6-1 shows the three areas that yellow lines are the footprint of the levees, while the gold and purple shaded areas show the contributing drainage of subbasin 400b, respectively. After the hydrology was completed, the levee related to subbasin 400b was moved to its present location shown in Chart 6-1. This made the contributing area smaller, which is why the purple shaded area extends to the river side rather than the location determined in the HEC-RAS model that water flowed away (south) from the levee for subbasin 400B; the model was not re-done. Notice that McCarren Boulevard forms the west edge of the small interior area protected by the levee. This area was modeled in HEC-1 as well. This section describes the hydrologic analysis for these three interior

**Design Storms.** 3-hour duration cloudburst events for the 10%, 2%, 1%, 0.5%, and 0.2% chance events were modeled with no areal reduction was applied to each subbasin because of their small size. The drainage areas of subbasin 400a, subbasin 400b, and the UNR Ring Levee are 0.26, 0.19, and 0.027 square miles, respectively. The cloudburst hyetograph was modeled in 5-minute increments, while the HEC-1 model runs in 5 minute time steps. The HEC-1 model automatically divides each event of rainfall into three even 5-minute parts. For example, a 15-minute value of 1.5 inches will be divided into three events of 0.5 inches. The 3-hour cloudburst event was balanced to the 15-, 30-, 60-, 120-, and 180-minute durations listed in NOAA Atlas 14.

**Design Storms.** 72-hour duration general rain was modeled for the 10%, 2%, 1%, 0.5%, and 0.2% chance events with no areal reduction was applied to each subbasin because of their small size. The general rain hyetograph was modeled in 15-minute increments, while the HEC-1 model runs in 15 minute time steps. A 1963 hourly storm hyetograph pattern was used for 6-, 12-, 24-, 48- and 72-hour depths. The 3-hour depth was taken from NOAA 14 and multiplied by a ratio of 0.73 characteristic of a general rain rather than a cloudburst event. The derivation of the ratios is described in more detail in the Appendix. The 6-, 12-, 24-, and 48-hour depths were taken from NOAA Atlas 14 without adjustment, while the 72-hour depth was 1.04 times the 48-hour value (based on an analysis of the Reno Airport gage).

The HEC-1 model for these three subbasins uses unit hydrographs derived from the Truckee Meadows Valley S-1000. The unit hydrographs are based on the Green and Ampt method as dictated in the Maricopa County, AZ Design Manual (see ref. 1). The unit hydrographs were based on soil types, but the soil-based XKSAT value was multiplied by a ratio of 0.90 based on

el to the December 31<sup>st</sup>, 2005 storm event on Boynton Slough and Steamboat Creek. A percent impervious  
 or each subbasin. The following table provides results of the model runs for the 1% chance event.

Interior Drainage Modeling Results for Southside of Truckee Meadows

Subbasin 400A		Subbasin 400B		UNR Ring Levee	
(cfs)	5-hour Volume (ac-ft)	Peak (cfs)	5-Hour Volume (ac-ft)	Peak (cfs)	5-Hour Volume (ac-ft)
	8	12	1.3	13	2.4
	12	38	3.2	24	3.5
	14	54	4.4	30	4.2
	16	73	6	37	4.8
	20	105	8	49	6

Interior Drainage Modeling Results for Southside of Truckee Meadows

Subbasin 400A		Subbasin 400B		UNR Ring Levee	
(cfs)	3-day Volume (ac-ft)	Peak (cfs)	3-day Volume (ac-ft)	Peak (cfs)	3-day Volume (ac-ft)
	24	3	3.2	2	1.8
	33	4.5	4.5	2.7	2.5
	37	5	5	3	2.8
	42	10	6	4	3.2
	47	20	10	5.4	3.8



Chart 6-1

## 7.0 Interior Drainage Behind Huffaker Dam

**7.1 Introduction.** The proposed Huffaker Dam will create a large lake behind it which poses an inundation threat to some homes and buildings. Two floodwalls are proposed to protect these areas. The floodwalls create the potential for interior drainage problems since they will block normal drainage patterns. Chart 7-1 shows the proposed floodwall footprints as dark red lines. One floodwall protects four warehouses, while the other protects residential homes. Chart 7-2 displays the HEC-1 subbasin map for this area. Subbasin #1306 is the small drainage area within the floodwall protecting the four warehouses. Subbasins #1301 and 1302 contribute runoff to the channel that runs on the north side of the warehouse floodwall. The runoff from Thomas and Whites Creeks flows north through subbasin #1304 before passing between a neckwall that leads to the lake behind Huffaker Dam. The neckwall is designed to protect both the warehouses and the residential area from backwater from the lake that moves up the channel. Subbasin #1305 is the interior area behind the floodwall that protects the residential area. These areas were modeled in HEC-1 for both cloudburst and general rain events for the 10%, 2%, 1%, 0.5% and 0.2% chance exceedence frequency events. The following describes this analysis.

**7.2 Cloudburst over Whites and Thomas Creeks.** A circular-shaped, 10 square mile cloudburst with a bullseye on subbasin #1304 was created to maximize the peak flow that would pass through the neckwall at the outlet of this subbasin. Although these hydrographs were not used for the Feasibility Study, they may be used in P.E.D. to determine if the peak flow from a cloudburst would outflank or overtop the upstream end of the neckwall. Figure 7-3 shows the circular outline of the storm over the subbasins with a bullseye in subbasin #1304. The storm duration is 3-hours. Frequencies modeled include the 10%, 2%, 1%, 0.5%, and 0.2% chance events. Areal reduction was applied to each subbasin based on its proximity to the bullseye. The cloudburst hyetograph was created in 15 minute increments. The 3-hour cloudburst event was balanced to the 15-, 30-, 60-, 120-, and 180-minute precipitation depths found in NOAA Atlas 14. Loss rates were based on Green and Ampt. Unit hydrographs were created from the Truckee Meadows Valley S-graph. The drainage area for subbasin #1304 is 2.26 square miles. The contributing drainage area for cloudburst storms, including upstream subbasins #225 and #1303, is 4.32 square miles. The following table provides results of this analysis.

Table 7-1. Results for Thomas and Whites Creek 10 Square Mile Cloudburst Centered on Subbasin 1304

Exceedence Frequency	Peak Discharge (cfs) for Outlet Subbasin 1304 Contributing d.a. (4.32 sq.mi.)
10%	560
2%	1,200
1%	1,500
0.5%	2,000
0.2%	2,700

**7.3 Cloudburst over Subbasins 1301, 1306, and 1305.** A point rainfall cloudburst with no areal reduction was placed in each of these subbasins. The drainage areas for subbasins 1301, 1306, and 1305 are 0.84, 0.10, and 0.36 square miles, respectively. The storm shape matches the subbasin shape. The purpose of the storm over subbasin 1301 was to see if a cloudburst event that sends water down the channel that runs on the north side of the four warehouses would outflank or overtop the floodwall protecting it. Although this hydrograph was not used in the Feasibility Study, it can be used in P.E.D. for this purpose. The purpose of the cloudburst over subbasin 1306 was to see if flapgates or pumping stations are needed in the interior area behind the floodwall protecting the four warehouses. This was analyzed by Hydraulic Design Section and the results will be in the Hydraulics Attachment. The purpose of the cloudburst over subbasin 1305 was to determine if a cloudburst event in the Bella Vista area would cause interior drainage problems behind the floodwall. The storm duration is 3-hours. Frequencies modeled include the 10%, 2%, 1%, 0.5%, and 0.2% chance events. Point rainfall was placed in each subbasin with no areal reduction. The cloudburst hyetograph for subbasins 1301 and 1305 was created in 15 minute increments and the HEC-1 model was run in this time step as well. Due to the small size of subbasin 1306 (four warehouses), the HEC-1 model was run in a 5-minute time step. The 3-hour cloudburst event for each subbasin was balanced to the 15-, 30-, 60-, 120-, and 180-minute precipitation depths found in NOAA Atlas 14. Soil loss rates were based on Green and Ampt. Unit hydrographs were created from the Truckee Meadows Valley S-graph. The following table provides results of this analysis.

Table 7-2. Results for Cloudburst Over Subbasins 1301, 1306, and 1305

Exceedence Frequency	Subbasin 1301 d.a. (0.84 sq.mi.) Peak (cfs)	Subbasin 1306 d.a (0.10 sq.mi.) Peak (cfs)	Subbasin 1305 d.a. (0.36 sq.mi.) Peak (cfs)
10%	140	70	110
2%	310	110	220
1%	420	130	280
0.5%	540	160	350
0.2%	730	200	460

**7.4 General Rain for Interior Drainage behind Huffaker Dam.** The purpose of this analysis was to model potential interior drainage problems behind the two floodwalls during an event which filled the Huffaker Dam pool. The HEC-1 model described in Part IV, Section 3.0 was used for this simulation. The storm used is the Huffaker Hill Storm centering that was described in Section 3.7 and shown on Chart 3-3. The only modification to the HEC-1 model was to break down subbasin #130 into smaller pieces to correctly model interior drainage areas. The original subbasin #130 was broken down into subbasins #1301 through 1305, which are shown on Figure 7-2. The contributing drainage area to each of the subbasins #1301, #1306, and #1305 is the drainage area in each of those subbasins. Because this is the Huffaker Hills general rain centering, the contributing drainage area to subbasins #1304 includes the upstream drainage areas for Whites and Thomas creeks as well; the combined drainage area for #1304 is 29.23 square miles. The general rain flood hydrograph for #1304 does not include the upstream flow diverted from Whites Creek to Steamboat Creek or the upstream flow diverted from Thomas

Creek north to Boynton Slough. The concurrent interior runoff when Huffaker Dam incurs a 1% chance pool elevation is shown in the table below.

Table 7-3. Results for General Rain Interior Drainage

Exceedence Frequency	Sub #1301: Channel on West Side Warehouses d.a. (0.84 sq.mi.) Peak (cfs)	Sub #1306 Warehouses d.a (0.10 sq.mi.) Peak (cfs)	Sub #1304 White & Thomas Creeks d.a. (29.23 sq.mi.) Peak (cfs)	Sub #1305 Bella Vista d.a. (0.36 sq.mi.) Peak (cfs)
1%	90	17	1,100	45

HUFFAKER DETENTION BASIN  
PATHS FOR MSEL @ 4435 FT (NAVD88)  
BASED ON 10m DEM DATA  
11 JULY 2007

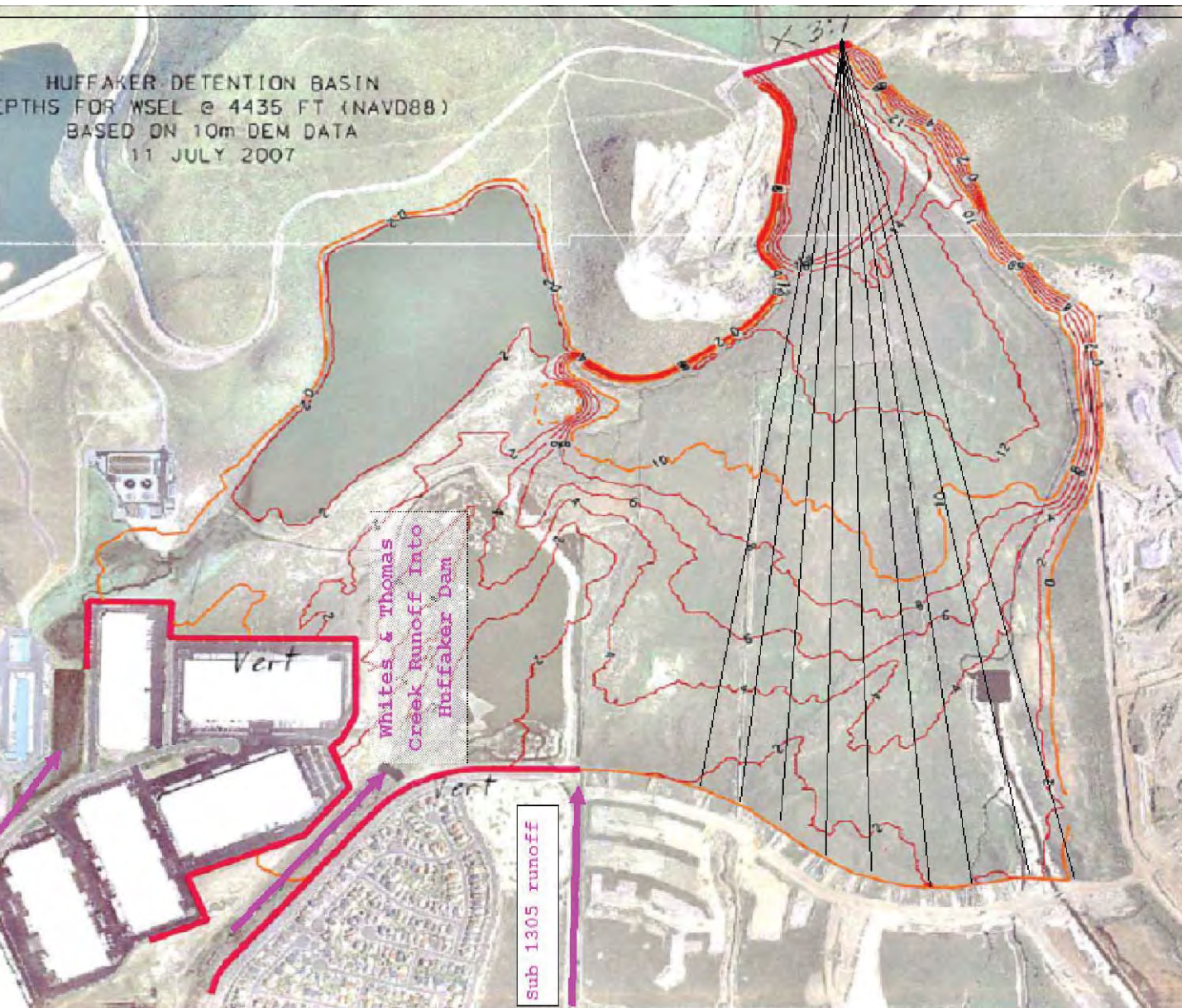


Chart 7-1

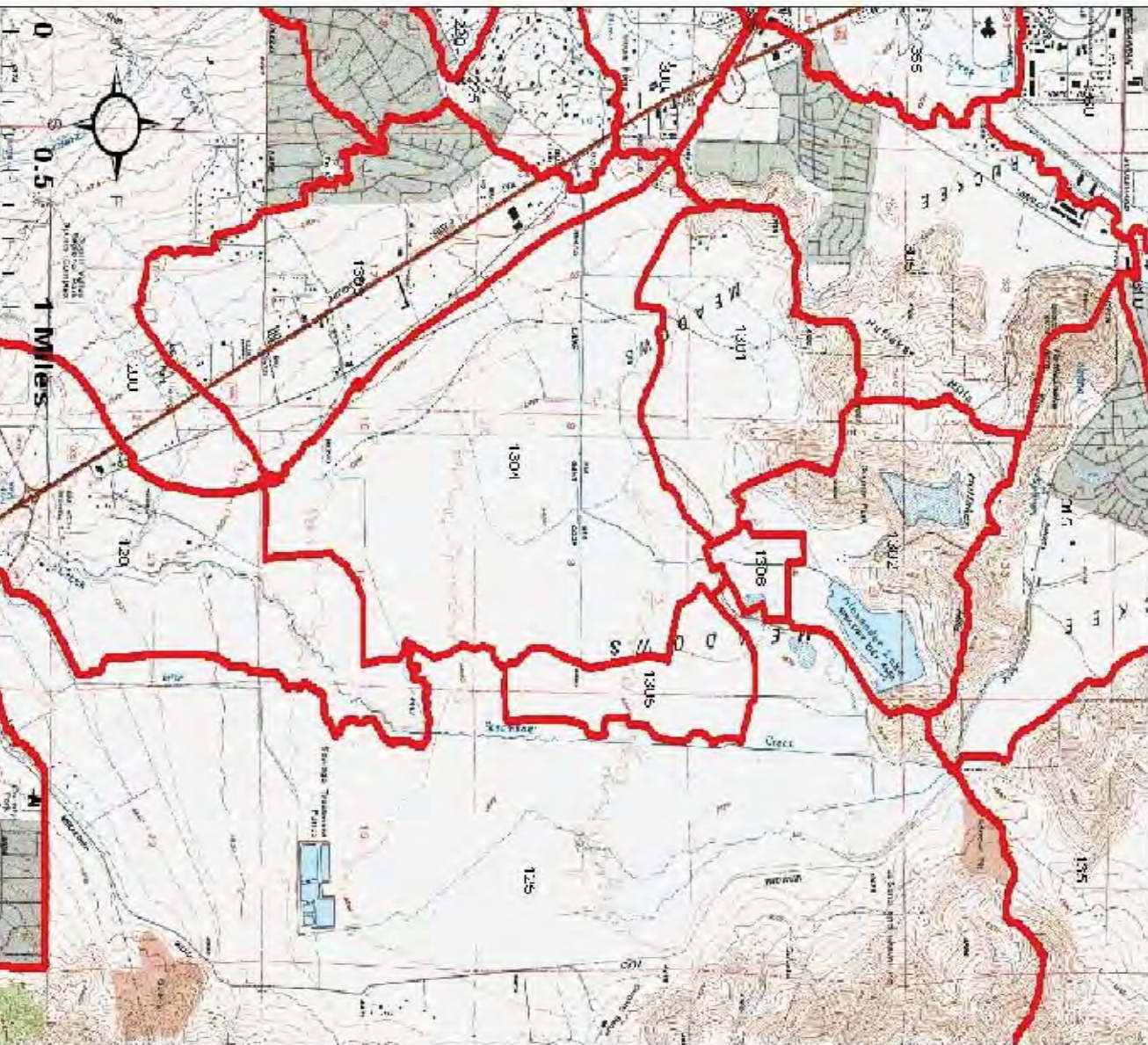
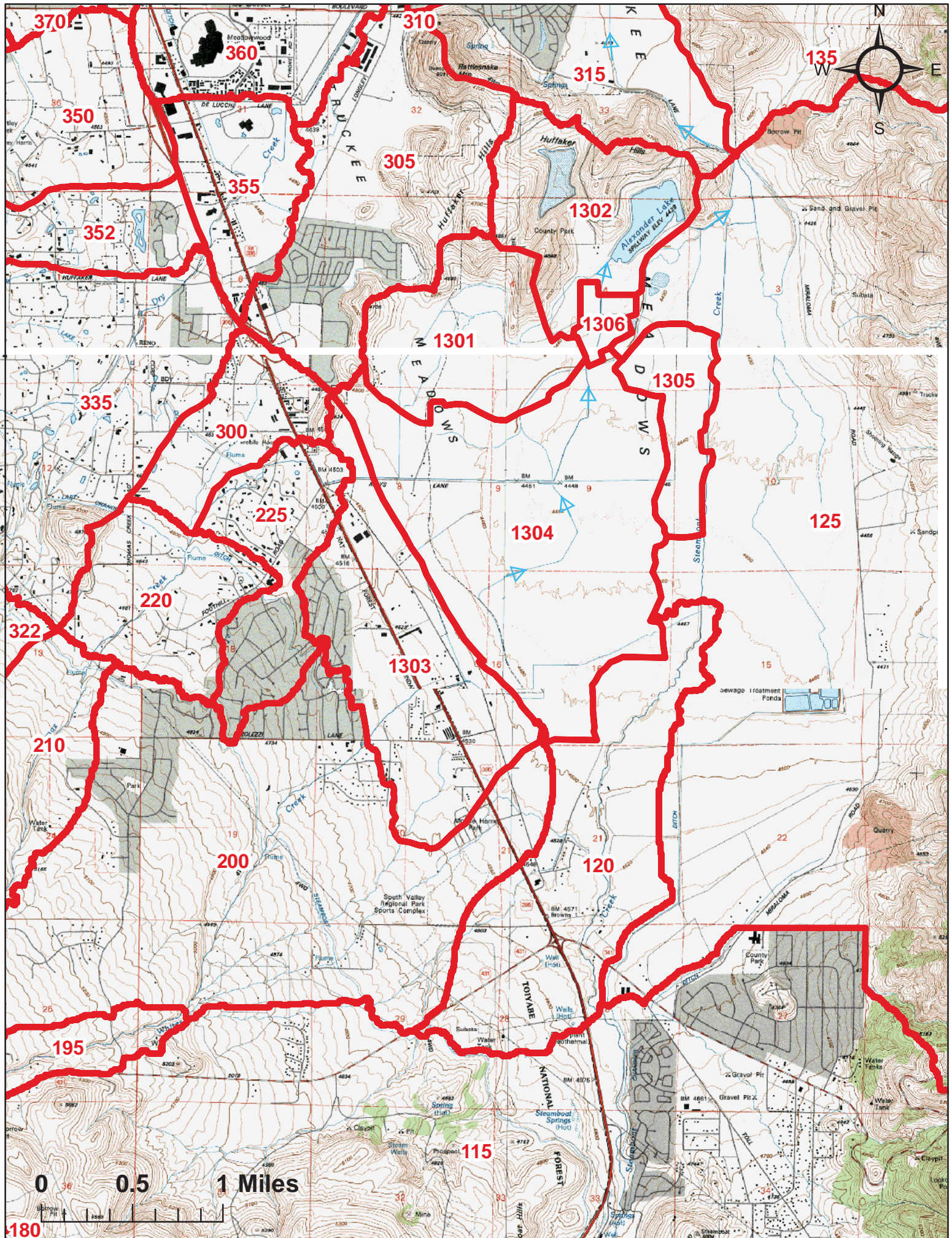


Chart 7-2



## 8.0 South Side of Downtown Reno.

**8.1 Introduction.** The Corps alternatives include floodwalls on the south side of the Truckee River in downtown Reno. The footprints for the floodwalls in the Downtown Reno Reach are shown in Chart 8-1 and 8-2. Chart 8-1 shows the 1.85 square mile drainage behind a short section of floodwall that connects to and runs a short distance upstream of Booth Street. Chart 8-2 shows the 1.5 acre empty lot that is flooded behind a short section of floodwall just upstream of the East 2<sup>nd</sup> Street Bridge. These structures may cause interior drainage problems. A 3-hour cloudburst and 72-hour general rain event were modeled in HEC-1 for the Booth Street Floodwall. Since cloudburst events have been found to dominate flooding in very small areas, only a cloudburst event was analyzed for the floodwall next to the East 2<sup>nd</sup> Street Bridge. Frequencies modeled include the 10%, 2%, 1%, 0.5%, and 0.2% chance events. The following text describes this analysis.

**8.2 Floodwall Upstream of Booth Street.** The 1.85 square mile area was modeled as one subbasin. A 3-hour cloudburst hyetograph was balanced to the 5-, 10-, 15-, 30-, 60-, 120-, and 180-minute durations found in NOAA Atlas 14. The triangular-shaped rainfall hyetograph was built in 5-minute increments and the model was run on a 5-minute timestep. The Truckee Meadows Valley S-graph was used to create a 5-minute unit hydrograph for the rainfall to runoff transform. Soil loss rates are based on Green & Ampt. Areal reduction factors appropriate for a 2-square mile area were applied to point precipitation for this subbasin.

Table 8-1. Results for Cloudburst, Upstream of Booth Street

Exceedence Frequency	Peak (cfs)	Max. 6-hour volume (ac-ft)
10%	300	30
2%	580	45
1%	830	58
0.5%	1140	75
0.2%	1660	105

A 72-hour general rain event was also created for this location. A pattern based on the 1963 storm (Ref. 5) was balanced to the 3-, 6-, 12-, 24-, 48-hour depths found in NOAA Atlas 14. The 72-hour depth was estimated as a ratio (48-hour depth times 1.04). Normally, in this interior drainage analysis, the 3-hour precipitation from NOAA Atlas 14 is multiplied by a ratio of 0.73, when it is used for a general rain event (as explained in Section 3.5). By mistake, the 3-hour depth was not reduced for this simulation. This will result in higher runoff and a more conservative answer. It is recommended that if this study proceeds to the P.E.D. phase, that this oversight be fixed and the model be re-run. Areal reduction factors appropriate for a 2-square mile area were applied to point precipitation for this subbasin. The HEC-1 model was run on a 15-minute time step. The Truckee Meadows Valley S-graph was used to create a 15-minute unit hydrograph for the rainfall to runoff transform. Soil loss rates are based on Green & Ampt. The following table provides results for the general rain simulation.

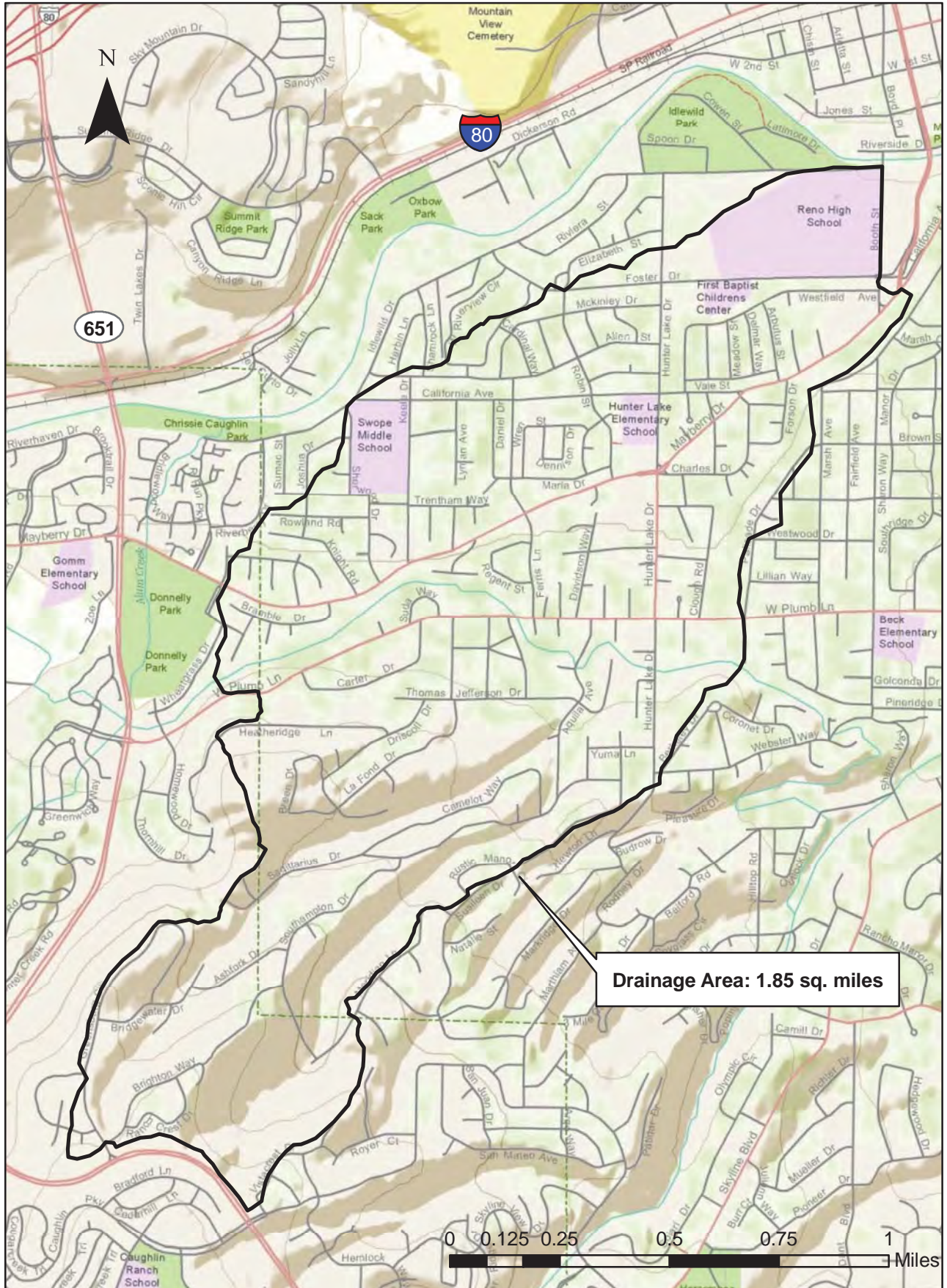
Table 8-2. Results for General Rain, Upstream of Booth Street

Exceedence Frequency	Peak (cfs)	Max. 24-hour volume (ac-ft)
10%	123	68
2%	172	92
1%	198	100
0.5%	280	120
0.2%	450	160

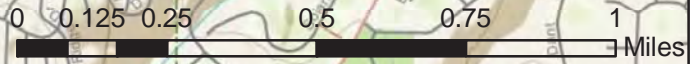
**8.3 East 2<sup>nd</sup> Street Interior Drainage Area.** A floodwall is proposed on the south side of the Truckee River just upstream of the East 2<sup>nd</sup> Street Bridge. This location is shown in Chart 8-2. The area protected is only 1.5 acres and consists of an open lot with no structures. Due to the extremely small size of this area, only a cloudburst event was analyzed. The 3-hour storm has a triangular shape and was balanced to the 5-, 10-, 15-, 30-, 60-, 120- and 180-minute durations found in NOAA Atlas 14. Due to the small size of the area, no areal reduction factors were applied to the point rainfall, and the HEC-1 model was run in 2-minute computation intervals. A 2-minute unit hydrograph was derived from the Truckee Meadows Valley S-graph. The soil loss rates were based on Green & Ampt and a value of 30 percent impervious surface area was applied to the subbasin. The results of the modeling are shown below.

Table 8-3. Results for Cloudburst, East 2<sup>nd</sup> St. Drainage Area

Exceedence Frequency	Peak (cfs)	Max. 6-hour volume (ac-ft)
10%	1	<1
2%	2	<1
1%	2	<1
0.5%	3	<1
0.2%	5	<1



Drainage Area: 1.85 sq. miles



**"Upstream of Booth St.  
Interior Drainage Area"**

**Chart 8-1**



A

B

## 9.0 North Side of Downtown Reno

**9.1 Introduction.** Floodwalls are proposed along portions of the north side of the Truckee River in downtown Reno. The floodwalls may block runoff from entering the Truckee River. The footprints of the floodwalls in the Downtown Reno Reach are shown on Chart 9-1. The downtown area is highly urbanized. The City of Reno recently completed the “Retrac Project”, which involved dropping the existing Union Pacific railroad track that runs through downtown into a 30 foot deep, open trench. The railroad track runs east-west, parallel with 3<sup>rd</sup> Street. Bridges were used to connect roads on each side of the trench. A company named Stantec Consulting, Inc. built an HEC-1 model to model the 1% chance event runoff from the watershed above the railroad track. The model did not cover the area between the railroad track and the Truckee River. The Corps obtained the with-project HEC-1 model from Stantec and modified it for its own use. The Corps goal was to determine how much runoff would reach the land side of the downtown floodwalls for the purposes of interior flooding analyses.

**9.2 HEC-1 Model Development.** A detailed description of the original Stantec HEC-1 model adopted by the Corps exists in a report written in 2003 (Ref. 36). A subbasin map for the Stantec HEC-1 model is shown in Chart 9-2. The Stantec model was created by delineations of upper areas using 7.5 quad maps, analysis of previous studies including one for Evans Creek, delineations using 2-foot topographic data provided by the City of Reno for downtown areas, and on-site field investigations. The 2<sup>nd</sup> Street subbasin is a separate HEC-1 model and was not used by the Corps since it drains to an area west of the proposed floodwall. The rest of the watershed in the HEC-1 model covers 13.25 square miles. The Retrac Project placed a floodwall on the north side of the new railroad track to keep water out of the 30-foot deep trench. Runoff accumulates at this floodwall and flows east along 3<sup>rd</sup> Street and either crosses over the railroad trench at one of the bridges or continues east until it discharges into the Truckee River at Wells Avenue. Using the Manning Equation, the Corps determined that 3<sup>rd</sup> Street is able to convey 110 cfs at West Street, 30 cfs at Sierra Street, and 12 cfs at Virginia Street before flow is conveyed across the bridge at each location. The Retrac Project specifically designed the new bridges to convey runoff across them, up to the 1% chance pre-project condition. The goal of Retrac Project drainage plan was to keep flooding on the north side of the railroad trench equal or less than the without-project condition (no railroad trench and floodwall).

A subbasin flowchart is displayed on Chart 9-3 to show the connectivity of the modified HEC-1 model used by the Corps for this analysis. Several small dams exist in the watershed including Upper Peavine Dam, Lower Peavine Dam, West Wash Dam and East Wash Dam. All four dams were designed to detain the upstream 1% chance flows. Outflow from the dams and surface runoff from the subbasins named Elmcrest, Lower Peavine, Upper Peavine, Sprout, West Wash and East Wash all concentrate at the Vine Street Sump located on the north side of Highway 80. The sump connects to an underground pipe (84 inch RCP) which has a capacity of approximately 800 cfs. The pipe runs under the 80 freeway, and follows Vine Street until it reaches an inverted siphon at the intersection of Vine Street and the railroad trench. The inverted siphon allows head pressure to push the flow in the pipe under the railroad trench, where it continues south on Vine, then takes a left turn and heads east on 2<sup>nd</sup> Street, then takes a right turn and heads south on Arlington until it discharges into the Truckee River in a large, concrete box culvert under the Arlington Bridge.

Upstream, on the north side of Highway 80, at the Vine Street Sump, two 10' x 4' RCB's (west side of Vine Street) convey excess runoff (that runoff which exceeds capacity of the sump) south under the freeway where it continues on the surface down Vine Street towards the railroad. An inlet exists at the intersection of Vine Street, and the railroad to allow surface water to enter the 84" RCP when it is not flowing at capacity.

The Stantec model used a 24-hour design storm pattern balanced to NOAA Atlas 2. The model uses the SCS unit hydrograph to transform rainfall to runoff and SCS Curve Numbers for loss rates. The routing method is accomplished using Muskingum-Cunge.

The following describes the Corps modifications to the Stantec HEC-1 model. The Corps developed new design storms, including a 24-hour duration general rain and a 3-hour cloudburst based on NOAA Atlas 14 (the Stantec model used NOAA Atlas 2). Stantec simplified the modeling of runoff between Highway 80 and the railroad track by routing and combining all the runoff at a concentration point called CPE near Wells Ave on the eastern edge of the railroad project. Separate calculations for detailed street flow in this zone were accomplished by Stantec using another program. The Corps modifications to the Stantec HEC-1 model begin at Highway 80. The main goal of the Corps was to determine how much water could cross over the railroad trench and combine with the runoff from the urban area between the railroad and the Truckee River. This is the runoff that would potentially pond behind the Corps' new Downtown floodwalls. The Corps modified the HEC-1 model as follows: 1) modified the code to track the amount of water entering the 84" RCP at the Vine Street Sump at Highway 80 and the excess that would cross under the freeway through the 10' x 4' box culverts, 2) routed the outflow from the pair of 10' x 4' box culverts southward on the surface of Vine Street to its intersection with the railroad trench, 3) routed and tracked the amount of flow in the 84" Vine Street RCP, 4) determined if surface flow on Vine Street could enter the 84" RCP at the intersection of Vine and the railroad via an inlet, otherwise route the excess eastward along 3<sup>rd</sup> Street, 5) track total surface runoff flowing eastward on 3<sup>rd</sup> Street to determine how much, if any, could cross the bridges, 6) increased the size of the model by adding the drainage area between the railroad and the Truckee River (subbasins DT10 through DT70).

**9.3 Development of HEC-1 Model Between Railroad and Truckee River.** As mentioned earlier, the Stantec HEC-1 Model did not cover the area between the railroad track and the Truckee River. The Corps added this small area, which is only 0.34 square miles. This area was divided into 9 separate subareas (subbasins DT10 through DT70), based on coordination with Hydraulic Design Section which performed the mapping of the interior flooding. These subbasins are shown on Chart 9-4. Two-foot topographic data were analyzed to determine the flow path of water at each street intersection. At some intersections, visual analysis alone could not determine the flow path, and InRoads software was used to determine the most likely flow path. When InRoads indicated an even likelihood of flow moving in either direction, judgment was used to determine the percent split flow down each street. These percentages are indicated on the flowchart in Figure 9-3. In the future, should the study go into P.E.D. phase, the Corps may want to use 1-foot contour data to provide better definition of flow paths. Like the rest of the model, the SCS unit hydrograph method was used for precipitation to runoff transform. For the general rain, soil loss rates utilized were SCS Curve Numbers. Since Reference 35 indicated

that SCS Curve Numbers are not appropriate for modeling storms less than or greater than a 24-hour duration, initial and constant loss rates were utilized for the 3-hour cloudburst model. The equivalent constant loss rate for each CN was determined by subtracting the total precipitation minus the total excess in the 0.2% chance general rain HEC-1 model output, after the initial abstract is satisfied. This gives an average loss rate during the simulation. The resulting constant loss rates ranged from 0.01 to 0.08 inches per hour. The flow routing was accomplished using Muskingum-Cunge. The only storm drain modeled for this area is the 800 cfs capacity pipe that runs south on Vine Street, east on 2<sup>nd</sup> Street, and then dumps into the Truckee River under the Arlington Ave. Bridge. The other known storm drains had a smaller outlet capacity to the Truckee River (approximately 5 cfs) and were deemed too small to be worth modeling. This decision will conservatively increase the total volume of surface runoff calculated.

**9.4 General Rain Event.** The general rain model uses a 24-hour storm balanced to the 15-minute, 1-hour, 2-hour, 3-hour, 6-hour, 12-hour, and 24-hour precipitation frequency depths for the watershed above the railroad trench. The depths were input on a PH card in the HEC-1 model, which distributes them in a triangular pattern with the most intense rainfall in the center of the storm. NOAA Atlas 14 depth duration frequency maps were overlaid on the subbasin map of the HEC-1 model in GIS in order to determine the averaged depths for each subbasin. The model runs on a 5-minute time step. The original Stantec HEC-1 model used point rainfall in each subbasin, with no areal reduction factors. For consistency, the Corps did not use areal reduction in their HEC-1 model for general rain either. As the whole watershed is only 13.25 square miles, this was not considered a significant issue.

The centroid of the 0.34 square mile area between the railroad trench and the Truckee River was used to find the NOAA 14 Atlas precipitation depths for all 9 subbasins. In other words, the same hyetograph was used for all 9 subbasins since it is so small. Since summer cloudburst events dominate the short-duration depths in NOAA Atlas 14, an adjustment was made to make the depths realistic for a general rain event. This adjustment was determined by studying 1% chance depths for the annual versus the winter precipitation frequency curves found in Reference 14 for the Reno Airport precipitation gage. The table below provides the ratios that were multiplied times the NOAA 14 precipitation to achieve depths representative of a general rain event.

Table 9-1. Ratios Applied to NOAA Atlas 14 Depths to Achieve General Rain Depths

Duration	Ratio Applied to Each Depth Duration
15-minute	0.36
30-minute	0.45
1-hr	0.54
2-hr	0.59
3-hr	0.72
6-hr	0.87
12-hr	1.0
24-hr	1.0

Note: If the 1-hour NOAA 14 depth is 1 inch, the new depth for general rain would be  $0.54 \times 1 = 0.54$  inches.

The results of the general rain event modeling for the 10%, 2%, and 1% chance flood events are shown in the table below.

Table 9-2. Results for General Rain Modeling

% Flood	Peak Flow (cfs)							
	Subbasin DT10	Subbasin DT21 <sup>1</sup> CP CRD3	Subbasin DT30 <sup>2</sup> CP CRD4	Arlington Ave Box Culvert	Subbasin DT40 <sup>3</sup> CP CRD5	Subbasin DT51 <sup>4</sup> CP CRD6	Subbasin DT60 <sup>5</sup> CP CRD7	Subbasin DT70
10%	15	27	49	402	20	7	8	4
2%	28	48	120	732	28	10	14	7
1%	36	61	161	816	32	11	17	9

\*1 – CP CRD3 refers to Concentration Point CRD3 which is the cumulative runoff from the outlet of subbasin DT20 and DT21.

See Flowchart on Chart 9-3.

\*2 – CP CRD4 is a combination of runoff from subbasin DT30 and flow crossing over the West St and Sierra St. bridges.

\*3 – CP CRD5 is a combination of runoff from subbasin DT40 and flow crossing over the Virginia St. Bridge.

\*4 – CP CRD6 is a combination of runoff from subbasin DT50, DT51, and flow crossing over the Virginia St. Bridge.

\*5 – CP CRD7 is a combination of runoff from subbasin DT50, DT60, and flow crossing over the Virginia St. Bridge.

**9.5 Cloudburst Event.** The cloudburst HEC-1 model uses a 3-hour storm balanced to 15-minute, 30-minute, 1-hour, 2-hour, and 3-hour depths. The depths were input on a PH card in HEC-1 model, which distributes them in a triangular pattern with the most intense rainfall in the center of the storm. NOAA Atlas 14 depth duration frequency maps were overlaid on the subbasin map of the HEC-1 model in GIS in order to determine the averaged depths for each. The centroid of the 0.34 square mile area between the railroad trench and the Truckee River was used to determine the precipitation depths for all 9 subbasins. In other words, the same hyetograph was used for all 9 subbasins in this location. The model runs on a 5-minute time step. Since the intensity of cloudburst cell precipitation can change drastically in a small distance, a storm centering along with areal reduction factors was applied to the cloudburst HEC-1 model design storm. The bullseye of the cloudburst was positioned in the watershed just upstream of the railroad track. The storm centering is on the Vine subbasin. Areal reduction factors from Reference 14 were applied to the subbasins based on their relative position to the storm centering, with the largest areal reduction applied to those subbasins farthest from the bullseye. The cloudburst storm covered the entire 13 square miles of the HEC-1 model, slightly larger than the 10 square mile limit placed on cloudbursts in other watersheds in this interior drainage report.

The results of the cloudburst event modeling for the 10%, 2%, and 1% chance flood events are shown in the table below.

Table 9-3. Cloudburst Modeling Results

% Flo-Od	Peak Flow (cfs)							
	Subbasin DT10	Subbasin DT21 <sup>1</sup> CP CRD3	Subbasin DT30 <sup>2</sup> CP CRD4	Arlington Ave Box Culvert	Subbasin DT40 <sup>3</sup> CP CRD5	Subbasin DT51 <sup>4</sup> CP CRD6	Subbasin DT60 <sup>5</sup> CP CRD7	Subbasin DT70
10%	38	63	49	422	20	7	19	10
2%	62	103	111	734	27	12	31	17
1%	76	126	138	816	31	15	38	20

\*1 – CP CRD3 refers to Concentration Point CRD3 which is the cumulative runoff from the outlet of subbasin DT20 and DT21.

See Flowchart on Chart 9-3.

\*2 – CP CRD4 is a combination of runoff from subbasin DT30 and flow crossing over the West St and Sierra St. bridges.

\*3 – CP CRD5 is a combination of runoff from subbasin DT40 and flow crossing over the Virginia St. Bridge.

\*4 – CP CRD6 is a combination of runoff from subbasin DT50, DT51, and flow crossing over the Virginia St. Bridge.

\*5 – CP CRD7 is a combination of runoff from subbasin DT50, DT60, and flow crossing over the Virginia St. Bridge.



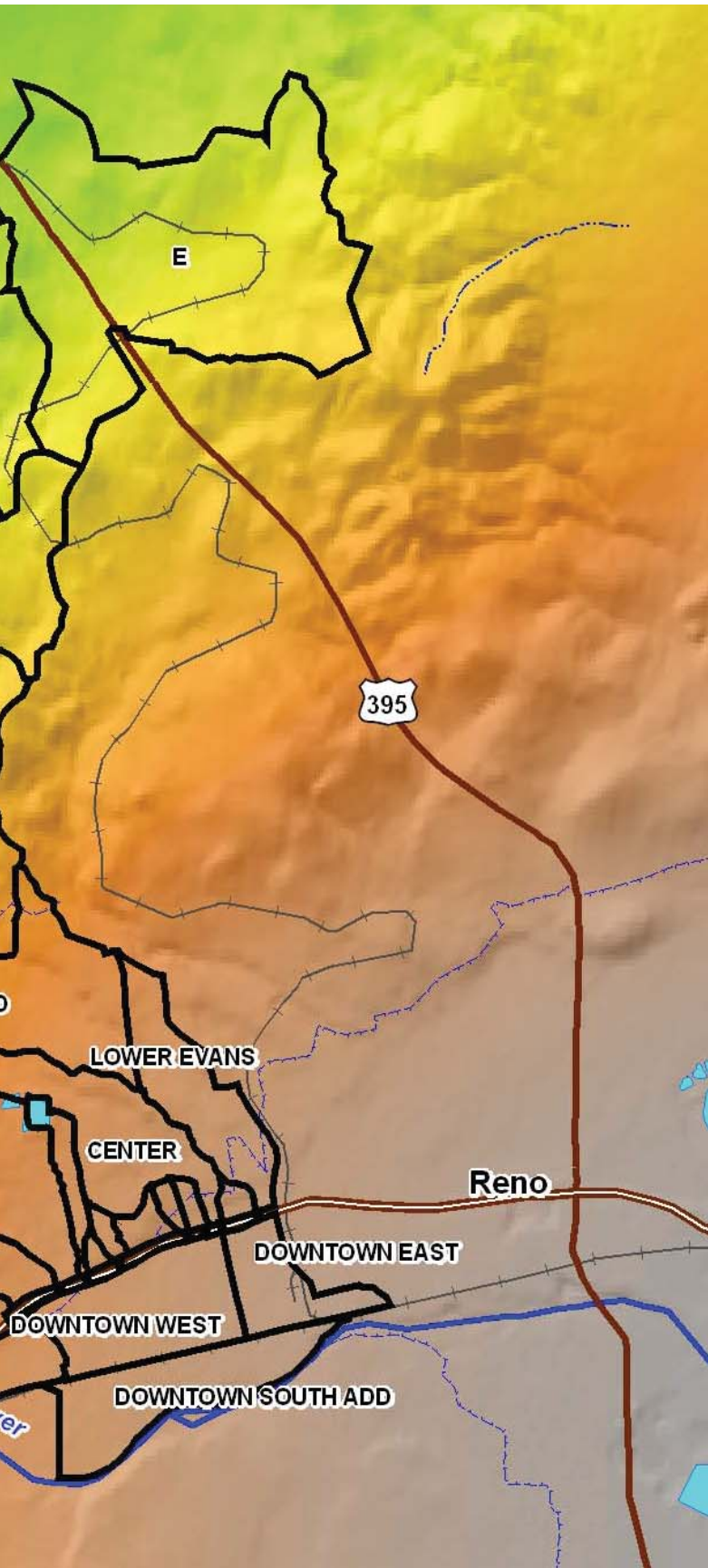


Chart 9-2



39.5166666° N

39.5222222° N

39.5277777° N

39.5333333° N

119.8333333° W

119.8250000° W

119.8166666° W

119.8083333° W



39.5166666° N

39.5222222° N

39.5277777° N

39.5333333° N

119.8333333° W

119.8250000° W

119.8166666° W

119.8083333° W

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## **10.0 Long Valley Creek and Lockwood**

**10.1 Introduction.** The Long Valley Creek watershed is approximately 100 square miles in area. It is the largest tributary on the Truckee River downstream of the Vista gage. A map of the watershed is shown in Chart 10-1 and a photograph of the community is shown in Chart 10-2. The watershed is arid and elevations range from 4,340 feet at its base to 7,420 feet at the top. There is no significant snowpack in the watershed and much of the year, including spring, the creek bed is dry. Near the mouth of the tributary sits the community of Lockwood, which has over 800 residents. The community sits within the 1% chance FEMA floodplain caused by Long Valley Creek. Long Valley Creek channel capacity is limited, partly due to bridge culverts on the creek that are partially filled in with sediment. A previous Corps hydrology study in the 1980's indicated the 1% chance peak flow for this watershed was 10,000 cfs. The 1980 hydrology was used to map the 1% chance FEMA floodplain. The community of Lockwood was flooded in February 1986, when the creek had an estimated peak discharge of 5,400 cfs.

Some of the proposed project alternatives in the Truckee Meadows Reach cause an increase in the peak flow and stage on the Truckee River at Lockwood for rare events like the 1% chance flood. The local sponsor plans to build a wall to protect the community from the increased stages on the Truckee River. Higher stages in the Truckee River can also increase the backwater effect on Long Valley Creek runoff. An analysis of Long Valley Creek flow frequency was needed to quantify the impacts of the with-project condition on the community of Lockwood. The concurrent flow in Long Valley Creek when the Truckee River is incurring an n-year event was determined in Part III of this Attachment. This hydrology was utilized by Hydraulic Design Section to analyze flood impacts on this community for the existing and with-project floods on the Truckee River mainstem (particularly the 1 in 100 and 1 in 117 chance events.).

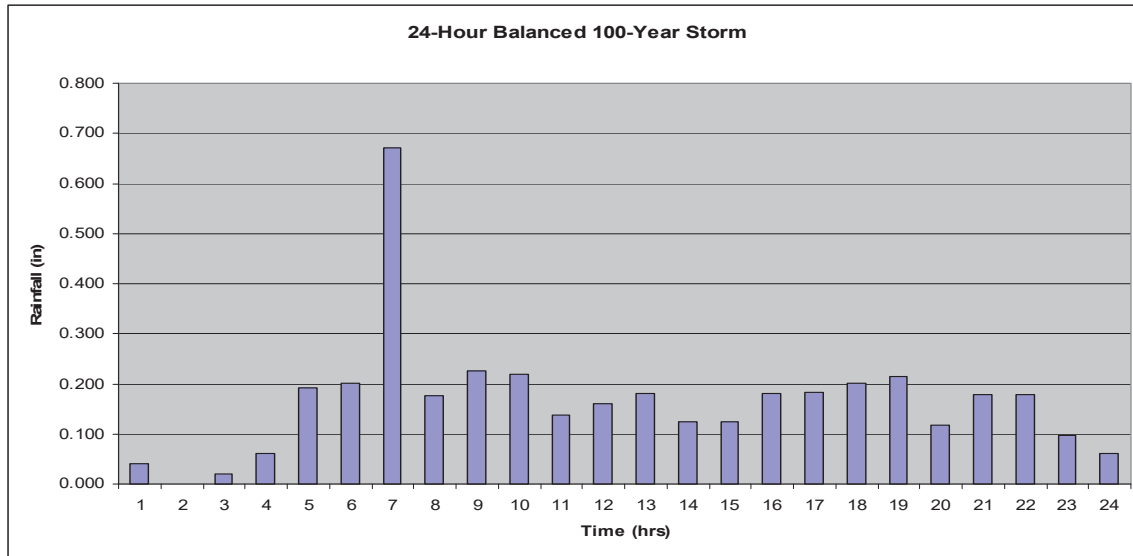
An additional analysis was needed to determine interior flooding problems when Long Valley Creek incurs an n-year event. With-project condition interior flooding problems can be worse than existing conditions due to either higher stages in the Truckee River or due to the wall that the local sponsors plan to build along the Truckee River to protect the Lockwood community. Several storm centerings were evaluated for both cloudburst and general rain events. This section describes the hydrologic analysis needed for this effort. A USGS peak instantaneous flow gage record exists on the Long Valley Creek.

**10.2 General Rain on Long Valley Creek.** An HEC-1 model was built for the entire watershed. The subbasin map for the model is shown in Chart 10-3. Approximately 2.7 square miles of subbasin 30 is a dry lakebed which will not contribute any runoff. The basin area card in HEC-1 (BA card) was reduced by this amount to account for this fact. 1-hour unit hydrographs were created from the Truckee Meadows Mountain General Rain S-curve. The model is run on a 1-hour time step. The soil loss rate method chosen was initial and constant. The constant loss rate for each frequency event was adjusted by calibration to the peak flow frequency curve adopted for this watershed.

A 24-hour duration general rain event was created for the model. 1-, 3-, 6-, 12-, and 24-hour precipitation frequency depths were gathered from NOAA Atlas 14 for each subbasin. The Jan-Feb 1963 storm pattern from Reference 5 was balanced to the depths mentioned above. The

storm was watershed-shaped as opposed to a circular or oval shape. A storm centering was created with the bullseye in subbasin 25. Chart 10-4 shows the areal reduction factors applied to each subbasin to achieve the storm centering. An example storm hyetograph for one of the subbasins is shown in the figure below.

Figure 10-1. Example General Rain Hyetograph for Long Valley Creek.



**10.3 Long Valley Creek Peak Flow Frequency Curve.** A staff gage called Long Valley Creek near Happy Valley (USGS # 10350100) lies in the lower portion of the watershed. The gage is in Lagomarsino Canyon, 3.3 miles upstream of the confluence with the Truckee River. The instrumentation at the site can only measure the instantaneous peak stage during a storm. There are no daily or hourly flow records. The peak of record occurred in February 1986 and was 5,400 cfs. Table 10-1 provides the period of record data for the gage.

**Table 10-1.**

List of Observed Annual Peak Flows at USGS #10350100  
 Long Valley Creek near Happy Valley Gage  
 Drainage area 82.6 sq.mi.

Year	Date	Peak (cfs)	Year	Date	Peak (cfs)
1956	23-Dec-55	2,500	1986	19-Feb-86	5,400
1967	16-Mar-67	1,430	1995	10-Mar-95	2,750
1968	20-Feb-68	1.0	1996	4-Feb-96	370
1969	25-Jan-69	2,560	1997	2-Jan-97	1,600
1970	24-Jan-70	400	1998	24-Mar-98	570
1971	26-Jun-71	20	1999	9-Feb-99	2
1972	16-Mar-72	1.0	2000	30-Jun-00	50
1973		0.1	2002	18-Mar-02	0.1
1974	17-Jan-74	105	2003	February	0.2
1975	March	180	2004	15-Aug-04	51
1976	1-Aug-76	17	2005	28-Mar-05	49
1977		1.5	2006	31-Dec-05	2,600
1978	March	85	2007	2-Jun-07	2

The gage record has missing water years (broken record) and only 26 years of data. The Steamboat Creek watershed, which is adjacent to Long Valley Creek, has higher elevations and a significant snowpack. The two watersheds have poor correlation. Correlation with the two-day rainfall recorded at the precipitation gage at Virginia City (which is just over the ridgeline) was used to estimate data for missing water years. Using a polynomial equation, enough correlation was found to estimate peak flows on Long Valley Creek. The correlation plot is shown in Figure 10-2. The correlation allowed for the estimation of an additional 16 water years of data. It is believed that the 1986 flood is the highest flow of record in 52 years based on discussion with Storey County. The 26 years of systematic record peak flows were plotted based on their ranking in the 42 years of combined systematic and estimated data. In addition, the plotting positions were adjusted for one historic flood (1986) that was the greatest in 52 years of record. Due to a sharp drop in flows at the lower tail of the frequency curve, statistics were derived for the upper end of the curve using a best fit regression line through the upper 10 recorded peak flow values. The final adopted frequency curve is shown in Figure 10-3.

Figure 10-2. Correlation of Long Valley Creek Peak Flows with Virginia City Precipitation

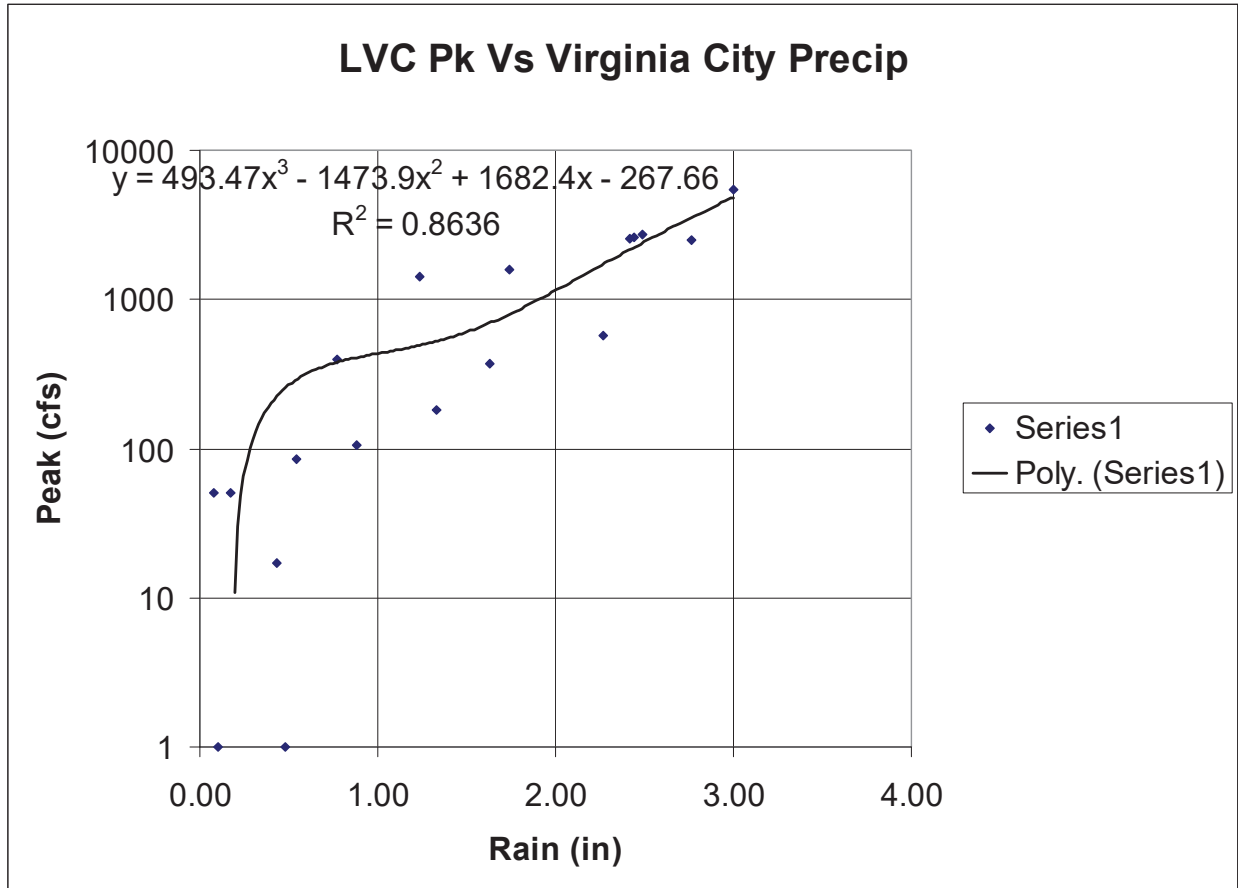
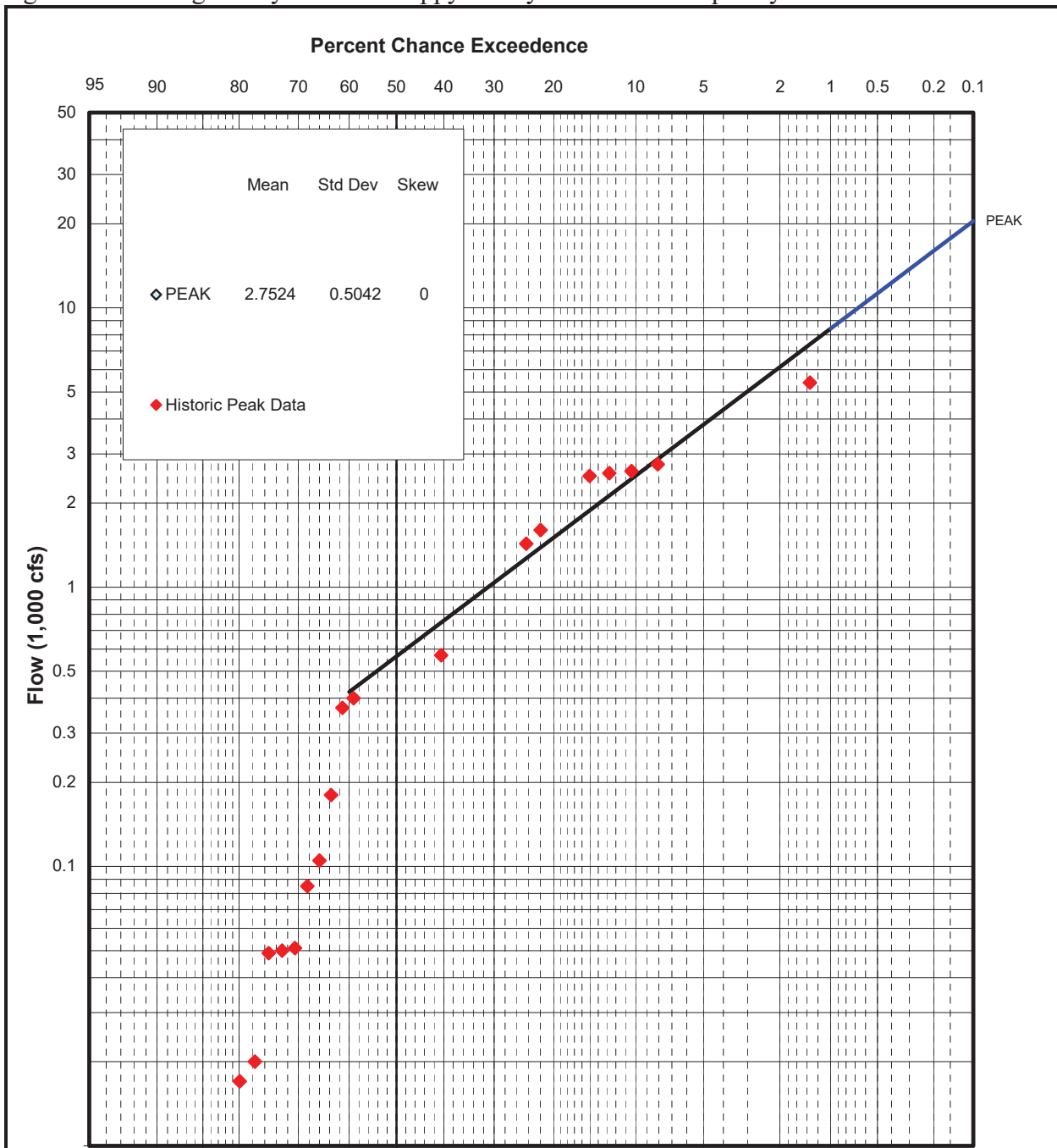


Figure 10-3. Long Valley Creek at Happy Valley Peak Flow Frequency Curve



**NOTES:**

1. Systematic record=26 years
2. Estimated values (not shown) estimated by correlation w/ Virginia City Precip.
3. Median plotting positions used to plot flows.
4. One historic flood with period of 52 yrs
4. Drainage area 82.6 sq.mi.
5. USGS Station 10350100

**LONG VALLEY CREEK AT HAPPY VALLEY  
PEAK FLOW FREQUENCY**

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U.S. ARMY CORPS OF ENGINEERS  
SACRAMENTO DISTRICT

Table 10-2. Long Valley Creek at Happy Valley Peak Flow Frequency Curve

d.a. (82.6 sq.mi.)	Exceedence Frequency						
	20%	10%	5%	2%	1%	0.50%	0.20%
Peak Flow at Subbasin #C21, Gage Site (cfs)	1,500	2,500	3,800	6,100	8,400	11,200	16,000

The loss rates for the general rain HEC-1 model were adjusted so that the peak flow at subbasin C21 came close to the adopted peak flow frequency curve in Figure 10-2. The HEC-1 general rain model results are shown in the table below.

Table 10-3. General Rain HEC-1 Model Results.

d.a. (82.6 sq.mi.)	Exceedence Frequency						
	20%	10%	5%	2%	1%	0.50%	0.20%
Peak Flow at Subbasin #C21, Gage Site (cfs)							
d.a. (82.6 sq.mi.)	1,400	2,400	3,700	5,900	8,500	11,000	15,900
Peak Flow at Outlet of Watershed (cfs)							
d.a. (98.3 sq.mi.)	1,500	2,500	3,800	6,200	9,000	12,000	17,700
Loss Rates (in/hr)	0.17	0.16	0.17	0.13	0.10	0.08	0.04

**10.4 Ten Square Mile Cloudburst Centered Upstream of Long Valley Creek Outlet.** A 10 square mile cloudburst storm series was centered on Long Valley Creek upstream of the canyon mouth. The purpose of the centering was to create high flows on Long Valley Creek channel that might leave the channel and flood the community of Lockwood. The storm overlaid on Long Valley Creek is shown on Chart 10-5. A single subbasin was used to model the upper 9.43 square miles of the watershed, while an additional four subbasins were used to model the areas subject to interior flooding area behind a proposed floodwall. The four smaller subbasins are shown on Chart 10-6. A 10-minute unit hydrograph was derived from the Truckee Mountain Cloudburst S-graph. Soil loss rates with an initial loss of 0.10 inches and a constant loss rate of 0.05 inches were used in the model. A 3-hour duration, triangular-shaped cloudburst storm was balanced to the 5-, 10-, 15-, 30-, 60-, 120-, and 180-minute NOAA Atlas 14 rainfall. Frequencies modeled include the 10%, 4%, 2%, 1%, 0.5%, and 0.2% chance events. Areal reduction factors for 9.4 square miles were applied to the rainfall for the upper 9.4 square mile subbasin, while areal reduction factors for 10 square miles were applied to the four subbasins next to the river. The HEC-1 model was run on a 10-minute time step. The results of the model runs are shown below. Since the peak flow of 1,700 cfs for the 1% chance event stays completely in the Long Valley Creek channel, this storm centering was not used for interior drainage analysis by Hydraulic Design Section.

Table 10-4. Results for 10-Square Mile Cloudburst Centered Upstream of Long Valley Creek Outlet

	Exceedence Frequency						
	20%	10%	4%	2%	1%	0.50%	0.20%
Long Valley Creek Peak Flow in cfs d.a. (9.43 sq.mi.)	540	760	1,100	1,400	1,700	2,100	2,900
Concurrent Peak Flow, West Interior Drainage Area (cfs) d.a. (0.09 sq.mi.)	2	2	5	9	14	20	30
Concurrent Peak Flow, East Interior Drainage Area (cfs) d.a. (0.26 sq.mi.)	12	15	20	30	45	67	100

Note: This storm centering was not used for interior drainage analysis for Lockwood since the Long Valley Creek 1% peak flow stays completely within the channel.

**10.5 Cloudburst Centered Over Lockwood Interior Drainage Areas.** A 3-hour duration cloudburst storm centered directly over the small subbasins shown in Chart 10-6 was modeled in HEC-1. As cloudburst events are a summer phenomenon, no significant flow is expected in the Truckee River during this type of event. The purpose of this centering is to see if what measures might be needed to mitigate for flooding behind the proposed wall along the riverfront of Lockwood. The design storm was balanced to the 5-, 10-, 15-, 30-, 60-, 120-, and 180-minute duration point rainfall from NOAA Atlas 14. No areal reduction factors were applied to the precipitation for these subbasins. Precipitation was also placed in a 4 square mile area upstream of the outlet of Long Valley Creek to produce concurrent runoff in the channel; however, the concurrent runoff on Long Valley Creek stayed completely within the channel banks. The results of this modeling are shown in the table below.

Table 10-5. Results for Cloudburst Centered Over Lockwood Interior Drainage Area.

	Exceedence Frequency						
	20%	10%	4%	2%	1%	0.50%	0.20%
Long Valley Creek Peak Flow in cfs d.a. (4.00 sq.mi.)	400	580	870	1,140	1,400	1,800	2,500
Concurrent Peak Flow, West Interior Drainage Area (cfs) d.a. (0.09 sq.mi.)	24	33	47	60	76	96	130
Concurrent Peak Flow, East Interior Drainage Area (cfs) d.a. (0.26 sq.mi.)	80	110	160	200	250	310	410



Long Valley Creek Shown on Right Side of Photo

Chart 10-1

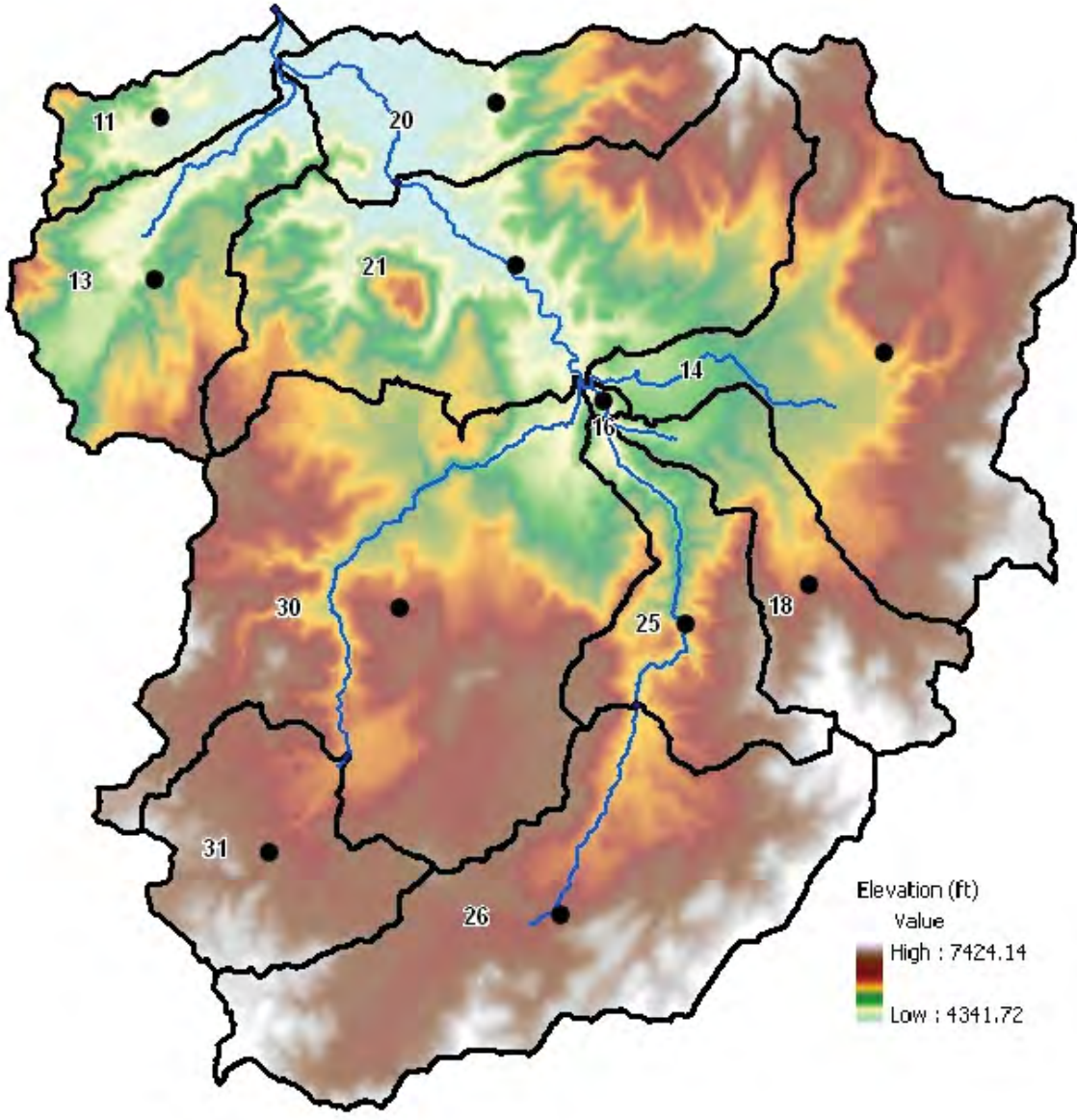


Truckee R

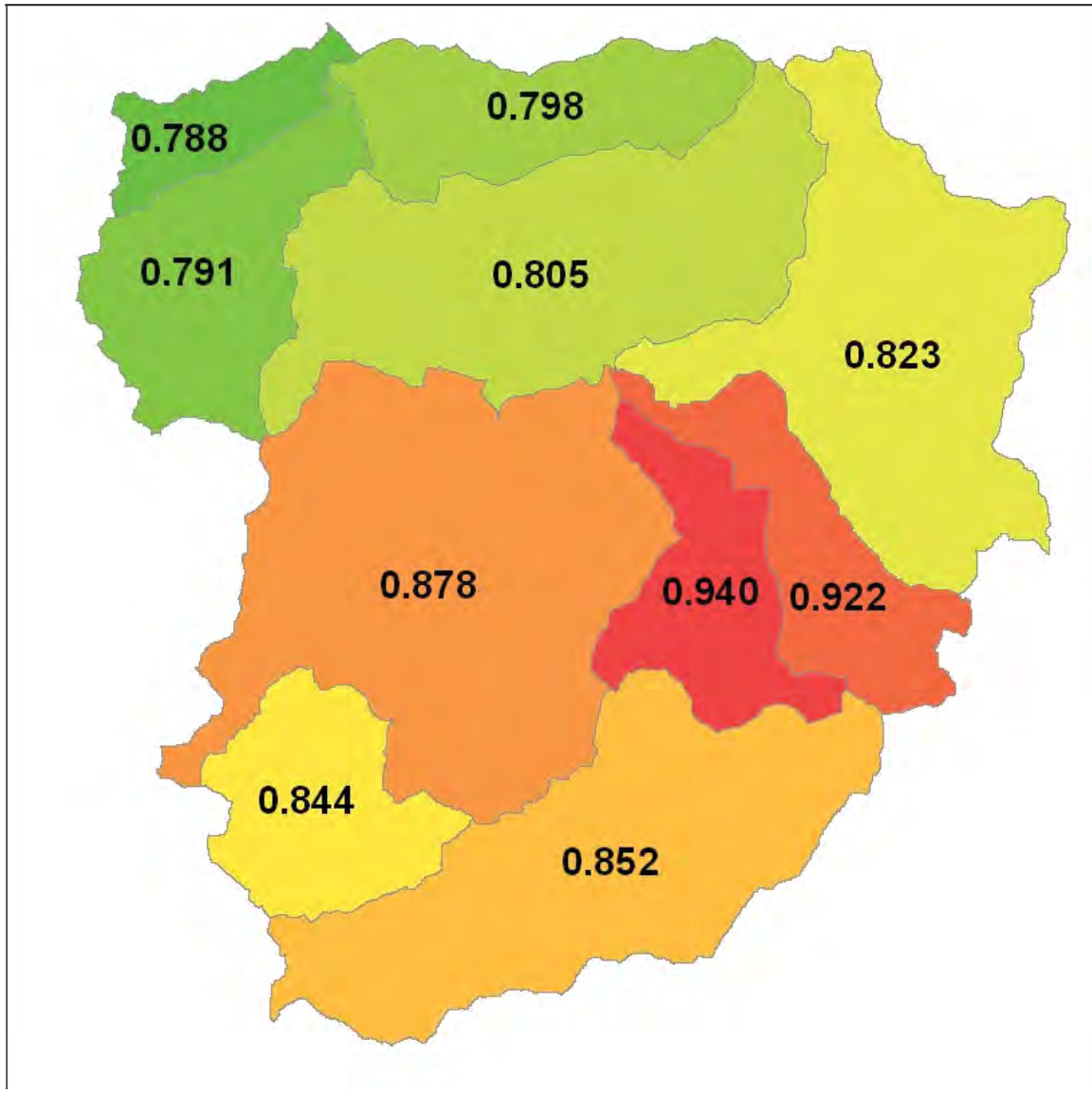


Comm  
Loc

0 0.05 0.1 0.2

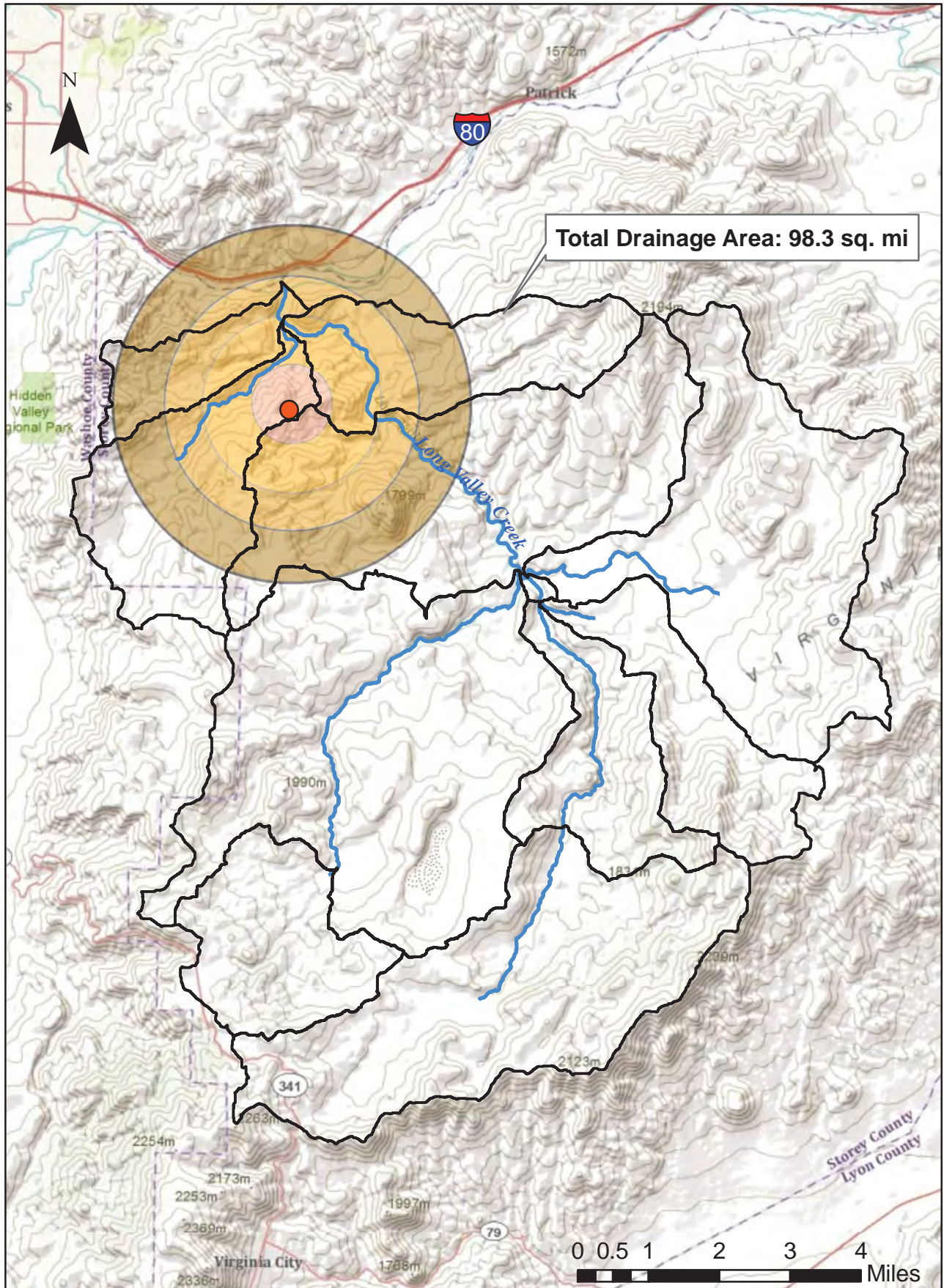


Long Valley Creek HEC-1 Model Subbasin Map



**Depth Area Reduction Factors Used to Create Storm Centering Over Long Valley Creek Watershed for General Rain Storm**

Chart 10-4



*"Long Valley Creek Storm Centering"*

**Chart 10-5**

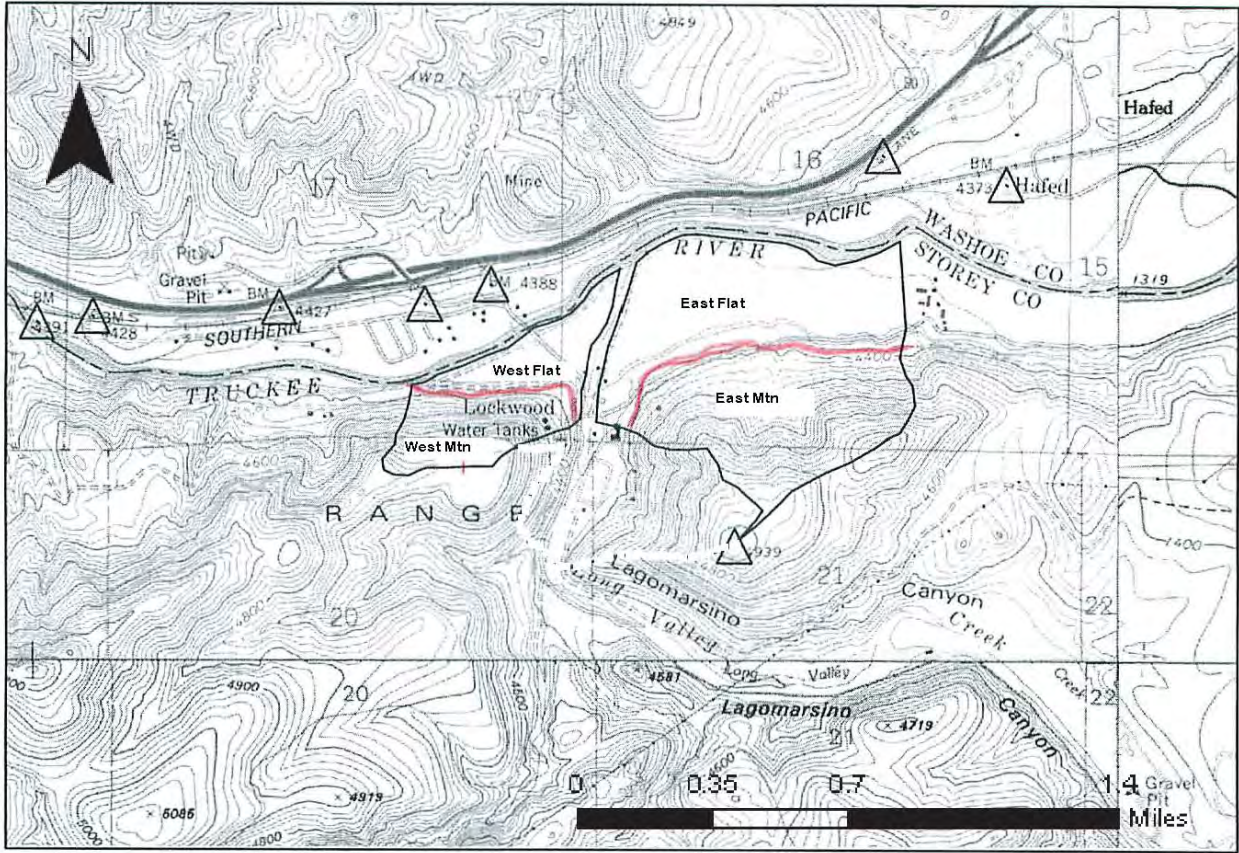


Chart 10-6

**PART V of Attachment A**

**RISK AND UNCERTAINTY INCLUDING  
CLIMATE CHANGE**