ATTACHMENT E

Hangtown Creek
Comprehensive Watershed Plan

March 2012

RBF CONSULTING

The Hangtown Creek Comprehensive Watershed Plan was prepared under the direction of:

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HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

TABLE OF CONTENTS

Exhibits

Figures

Tables

Photographs

List of Abbreviations

CEQA: California Environmental Quality Act
CN: Curve Number
CY: Cubic Yard
DCIA: Directly connected impervious area
DA: Dominchelli and Associates
DEM: Digital Elevation Model
DWR: California Department of Water Resources
EDCDM: El Dorado County Drainage Manual
FDA: Flood Damage Assessment
FEMA: Federal Emergency Management Agency
FIS: Flood Insurance Study
F-RAM: Flood Rapid Assessment Model
FRAP: Fire and Resource Assessment Program
GIS: Geographic Information System
GMG: Global mapper grid
HEC-FDA: Hydraulic Engineering Center Flood Damage Assessment
HEC-HMS: Hydraulic Engineering Center Hydrologic Modeling System
HEC-RAS: Hydraulic Engineering Center River Analysis System
LID: Low Impact Development
LiDAR: Light Detection and Ranging
LS: Lump Sum
MAP: Mean Annual Precipitation
NDCIA: Non-directly connected impervious area
NEH: National Engineering Handbook
NMFS: National Marine Fisheries Services
NOAA: National Oceanic and Atmospheric Administration
NRCS: National Resource Conservation Service
SCS: Soil Conservation Service
SY: Square Yard
USACE: United States Army Corps of Engineers
USDA: United States Department of Agriculture
USFWS: United States Fish and Wildlife Service

EXECUTIVE SUMMARY

This Hangtown Creek Comprehensive Watershed Plan (Plan) provides an updated topographic, hydrologic and hydraulic basis for flood risk analysis. Potential projects were considered and economic analysis of selected options was performed.

The first step of developing the Plan was to obtain new topographic mapping. One-foot contour mapping was developed along the stream corridors and somewhat less accurate mapping was created for the rest of the watershed. The new mapping was used to as a basis for new watershed hydrology and hydraulic analysis.

A detailed hydrologic model of the watershed was created to compute flow rates in Hangtown Creek. Measured rainfall and stream data was used to validate the hydrologic model results. The peak discharges for 10-, 25-, 50-, and 100-year recurrence intervals storm events for the new model are higher than those used in the 1983 Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) but lower than those indicated in recent studies performed by others for projects in the watershed. The increase in discharge from the FEMA flow rates is primarily due to the more detailed methodology employed for the Plan compared to the general approach used by FEMA. Estimates of the impacts of development since 1983 on peak discharges are also provided.

An updated hydraulic model of Hangtown Creek was prepared using validated information from the FEMA model, new topographic data, field measurements and record drawings. Water surface profiles were prepared and compared to the FEMA profiles. The higher water surface profiles indicated by the Plan over those indicated by FEMA primarily result from the higher discharges indicated by the new hydrologic model.

Eighteen locations were evaluated to determine the feasibility of potential flood damage reduction projects. Only four of these were found to show any promise of being potentially feasible. Flood damage reduction analysis and concept-level planning cost estimates for these four projects show that benefit-cost ratios for two of the four projects are greater than 1. Table ES-1 summarizes the estimated costs and expected economic benefit determined for the four projects.

Table ES-1: Estimated Cost and Expected Benefit Summary

Project	Estimated Capital Costs	Comprehensive Project Costs	Comprehensive Damage Reduction	Benefit-Cost Ratio
C	$862,000	$962,000	$173,000	0.2
I	$766,000	$866,000	$1,068,000	1.2
L	$855,000	$955,000	$402,000	0.4
R	$832,000	$932,000	$1,017,000	1.1
C, I, L, R	$3,315,000	$3,715,00	$2,167,500[1]	0.6

[1] Combined comprehensive damage reduction is less than the sum of the individual projects.

1.0 Introduction

1.0 INTRODUCTION

1.1 Purpose

This Hangtown Creek Comprehensive Watershed Plan (Plan) provides a new hydrologic model of Hangtown Creek Watershed, an updated hydraulic model of the Creek, and an evaluation of potential projects that could reduce future flood damages. The updated hydraulic model uses structure information from the 1983 FEMA Flood Insurance Study validated through field investigation, new topography from Light Detection and Ranging (LiDAR) data supplemented with photogrammetry, record drawings, field observations, and limited field survey observations collected as part developing this Plan.

This report documents the updated hydrology and hydraulic models and describes potential flood damage reduction projects. Water surface profiles and preliminary floodplain mapping are provided for Hangtown Creek from a downstream limit near the wastewater treatment plant about 500 feet west of Mallard Lane upstream to the confluence with Hangtown Creek Tributary near the intersection of Smith Flat Road and Broadway. Cost estimates and flood damage reduction analysis are included for the projects that were deemed potentially feasible.

The Plan provides recommendations that could be used as a roadmap for the City to pursue construction of projects to reduce future flood damages, and can serve as a basis for project feasibility studies and future floodplain mapping efforts. This project does not include finalizing floodplain maps for submission to FEMA. However, this report includes modeling and base mapping that could be used to support floodplain map revisions in the future.

1.2 Funding and Scope

This project was funded by a Sierra Nevada Conservancy Safe Drinking Water, Water Quality and Supply, Flood Control River and Coastal Protection Act of 2008 (Proposition 84) Grant Program. The tasks included in this project were:

1. **Initial Data Collection.** This task included reviewing available FEMA data and record drawings, extracting information of value to this study and performing field reconnaissance.
2. **Survey Data Collection and Processing.** This task included collection of aerial LiDAR and photographic data, supplemental field survey and property boundary information for potential project locations.
3. **Existing Conditions Hydrology and Hydraulic Modeling.** This task included development of a hydrologic model of the watershed that provides 10-, 25-, 50- and 100-year discharge hydrographs throughout the watershed and an updated hydraulic model with its associated water surface profile results.

4. **Evaluation of Potential Flood Damage Reduction Projects.** This task involves evaluating the potential of identified sites to reduce peak flood discharges.
5. **Integration of Restoration and Water Quality Benefits into Potential Project.** This task includes performing database research and pedestrian field survey of the potential project sites to identify environmental constraints, impacts and opportunities.
6. **Progress Meetings and Public Presentations.** This task involves meeting with the City and presentation at a public meeting.
7. **Project Cost and Benefit Analysis.** Development of planning level cost estimates for flood reduction projects that are potentially feasible and estimates of expected flood damage reduction expected with those projects.
8. **Funding Pursuit Assistance.** Provided information to support future grant applications for project implementation and assistance coordinating with FEMA for FEMA to share in the costs of updating FEMA maps.

1.3 Setting

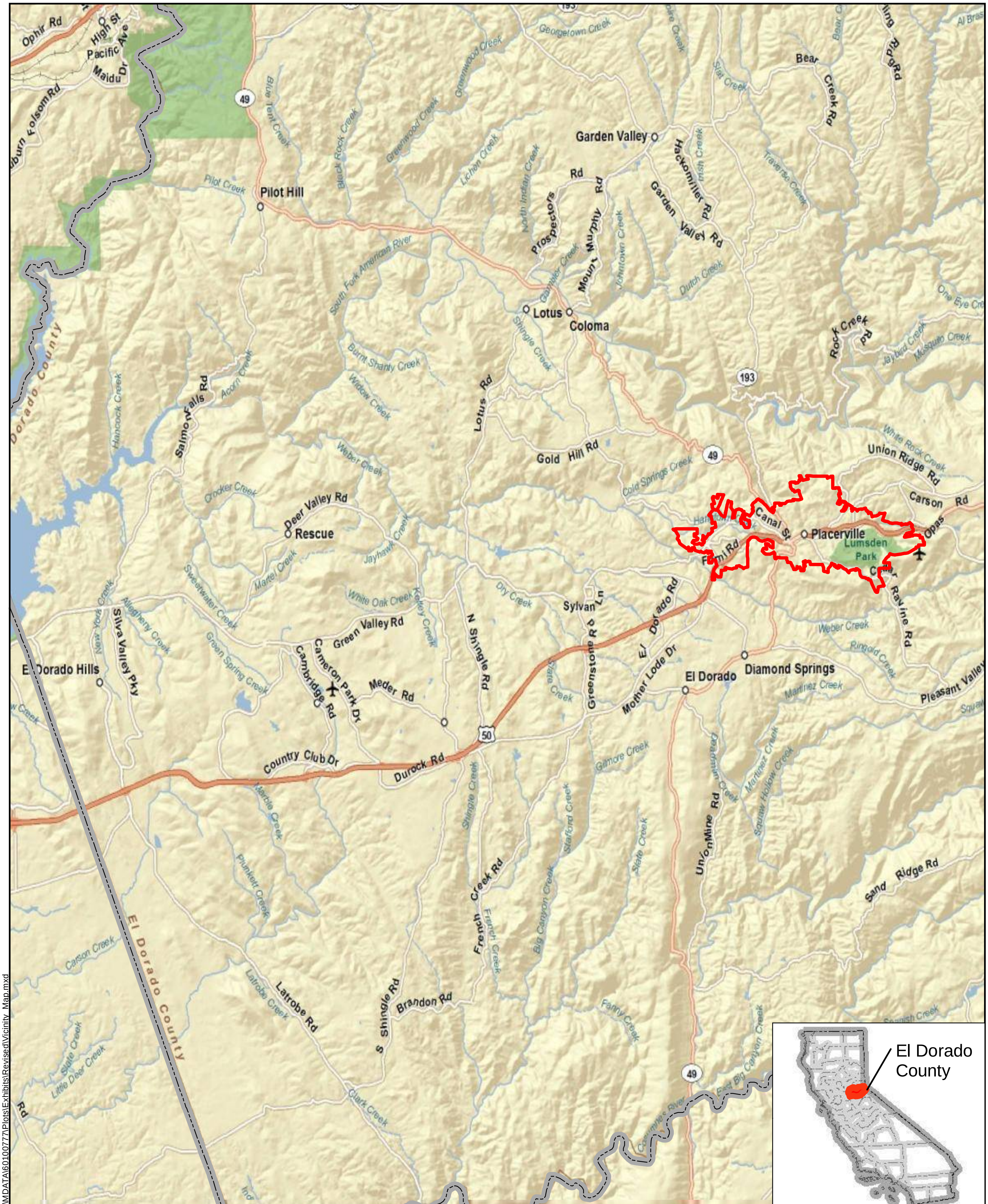
Hangtown Creek runs east to west through the City of Placerville. Numerous tributaries drain into Hangtown Creek from the north and south. Major tributaries include Randolph Canyon, Cedar Ravine, and Hangtown Creek Tributary. The Hangtown Creek watershed covers 8.6 square-miles of the Sierra Nevada foothills from an elevation of approximately 1,500 feet up to 2,600 feet. Hangtown Creek drains into Weber Creek, a tributary of the South Fork of the American River, approximately one mile downstream from the western corporate limit of the City near the wastewater treatment plant. A vicinity map of the watershed is included as Exhibit 1. An overall watershed map showing the major tributaries and City limits is provided as Exhibit 2.

The City of Placerville lies almost entirely within the Hangtown Creek watershed. Downtown Placerville straddles Hangtown Creek with significant portions of the creek located under parking lots and buildings. Numerous buildings have been constructed directly over the Creek. Examples of structures over the creek are provided as Photographs 1, 2 and 3.

Photograph 1: Building near Locust Avenue

Photograph 2: Building Adjacent to Broadway near Smith Flat Road

Photograph 3: Building Adjacent to Broadway near Smith Flat Road



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Vicinity Map

Exhibit 1

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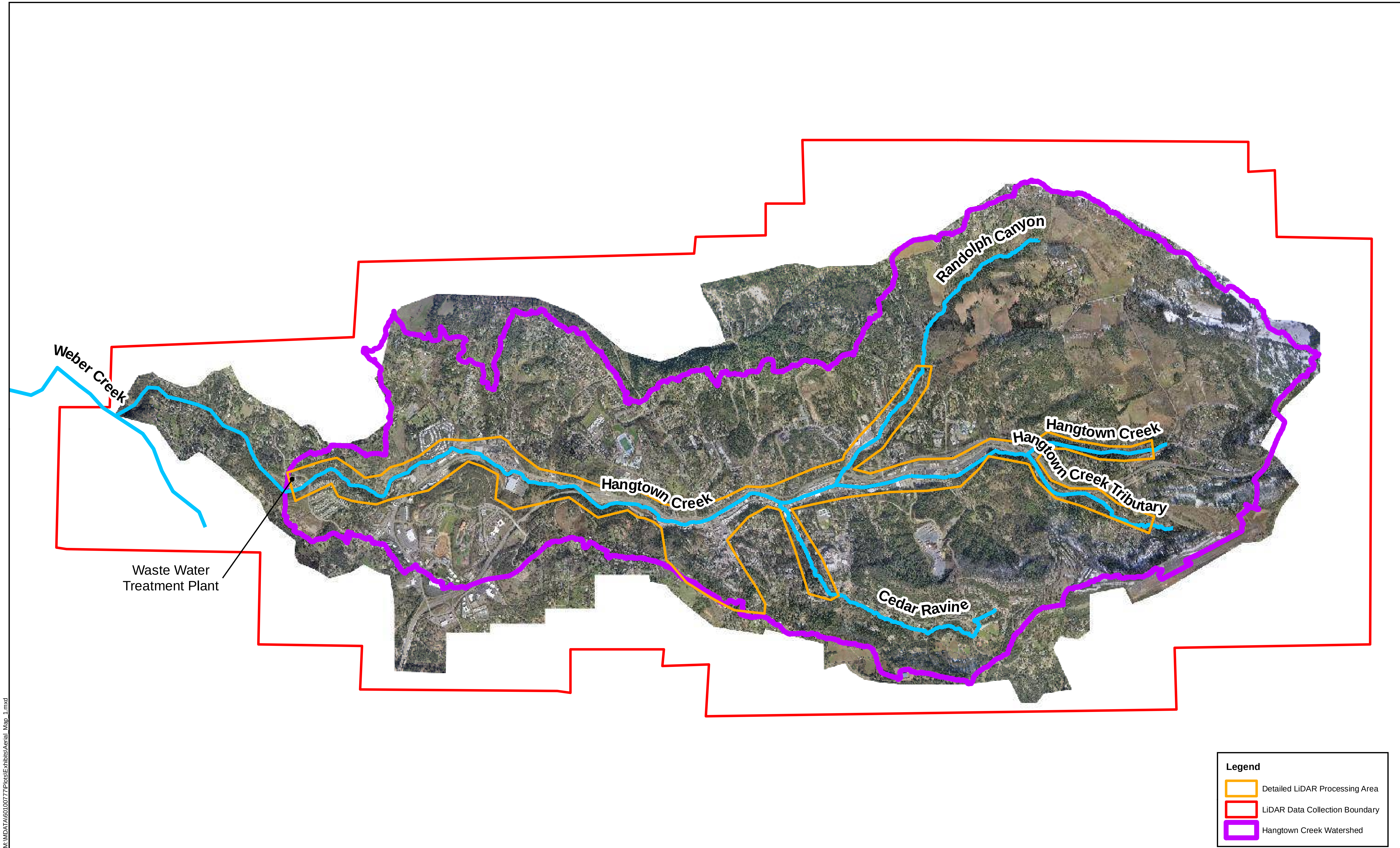

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PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

LiDAR Capture Area

Exhibit 2

Due to the proximity of some structures to the channel, there is a significant risk of severe flood damages in the event of a major storm event. Photograph 4, 5, and 6 show flooding that occurred during a major storm event on December 31, 2005. During a 24-hour period from December 30 to December 31, 2005, 5.4 inches of precipitation occurred at the Placerville Wastewater Treatment Plant. This corresponds to a return period between 25 and 50 years. However, the peak 2-hour to 6-hour rainfall quantities that were responsible for the peak discharges had a return period of approximately 10 years.

Photograph 4: Flooding Adjacent to Placerville Drive Upstream from Home Depot

Photograph 5: Flooding near Lower Main Street

Photograph 6: Flooding along Smith Flat Road

1.4 Data Sources

1.4.1 FEMA Data

The current effective FEMA Flood Insurance Study (FIS) and maps were published in 1983. At numerous locations the FEMA maps do not reflect current conditions along the creek. FEMA provided copies of printouts of the HEC-2 files that are the basis for the effective mapping. The main stem of Hangtown Creek was modeled by FEMA using eight separate HEC-2 models. Copies of printouts for the Hangtown Creek Tributary, Cedar Ravine and Randolph Canyon models were also provided by FEMA. Separate models were used for the creek on either side of long culverts and the long culverts were not included in the models. However, the FEMA models did include detailed representation of 28 bridge and culvert structures.

The FEMA model data for Hangtown Creek was manually digitized and imported into HEC-RAS. The converted model was geo-referenced using a stream center line from the new topographic data and by adjusting the stream distances to account for the missing long culverts.

1.4.2 Aerial Topography

New topographic data was developed to provide an accurate and up-to-date basis for this study. LiDAR and photographic data were acquired by HJW Geospatial, a division of Photo Science Inc., in February 2011. To cost effectively support the study, more detailed data collection was performed along the creek corridor (and a small area to support a water line improvement project) and less detailed data was collected for the rest of the watershed and City limits. The limits of LiDAR data acquisition are shown on Exhibit 3. Specific mapping products included:

1. LiDAR data of the entire 10.1 square-mile area with collection in the 1.1 square-mile creek corridor at 2-3 points per square meter to support 1-foot contour mapping, as measured against checkpoints in flat, open areas. The accuracy of the mapping for the surrounding 9.0 square-miles is not be as high because it is intended for watershed delineation and other planning purposes, but not 1-foot contour mapping. Hangtown Creek, Randolph Creek and Cedar Ravine Creek, and the area of the Pardi Lane/Big Cut Road/Sacramento Street Water Main Replacement Project are included in the 1.1 square-mile area.
2. Aerial photography of the 10.1 square-mile area that covers the City limits and Hangtown Creek watershed tributary to the City, at 6-inch resolution color orthoimagery.
3. Breaklines and terrain modeling of the 1.1 square-mile area to support the 1-foot contour mapping. Planimetric mapping at 1″= 100′ scale, showing building footprints and other features other than utilities is included on the mapping of the 1.1 square-mile area.
4. Color orthoimagery at 0.5-foot resolution in TIF/TFW format and a SID mosaic.
5. LiDAR point data for the entire 10.1 square-mile area. [2-foot engineering contours (as opposed to cartographic contours) generated using an automated process for the part of the 10.1 square-mile area outside of the 1.1 square-mile area were also developed and provided as part of this project.]

1.4.3 Field Reconnaissance and Topographic Survey

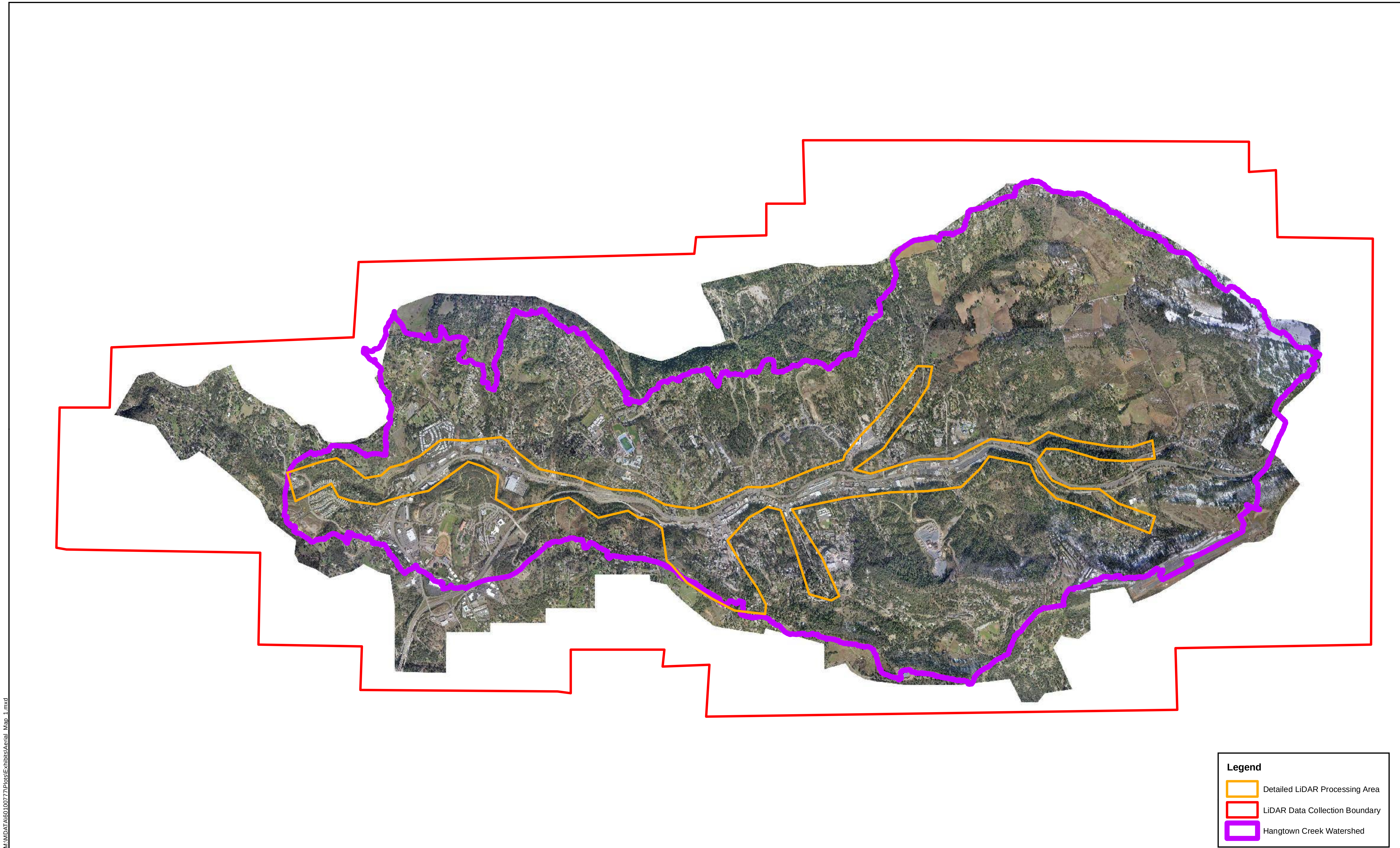
Field visits were conducted by RBF in February and June 2011 to verify the stream crossings such as bridges and culverts. A total of 47 stream crossings were visited and documented. Of the 47 stream crossings that were visited, 22 are in the effective FEMA HEC-RAS model. The additional stream crossings that were visited include 8 stream crossings upstream and downstream of the extents of the effective FEMA HEC-RAS model, 2 new bridges near Home Depot along Placerville Drive across from Cold Springs Road, and 15 stream crossings that were not included in the effective FEMA model. The stream crossings not included in the FEMA HEC-RAS model were lumped together with nearby bridges or buildings in the FEMA FIS model or, for some long culverts, hydraulic routing was performed separately from the HEC-2 models.

Supplemental field surveys were completed to determine the dimensions of three structures and to supplement LiDAR topographic data in hydraulically significant areas where vegetation obscured LiDAR penetration. Locations for supplemental field survey were determined subsequent to field visits. Structures at Center Street, Locust Lane, and Mallard Lane were surveyed to verify low chord elevations, upstream invert elevations, and overtopping elevations for inclusion in the HEC-RAS model. Eleven cross sections were surveyed, including seven cross sections near Wiltse Road, two cross sections near Locust Lane, one cross section near Center Street, and one cross section near Mallard Lane.

1.4.4 Record Drawings

Record drawings were received from the City of Placerville for the two Home Depot bridges that were constructed in 2004 and for channel modifications upstream from Home Depot. These record drawings were used to define the geometry of the two Home Depot bridges and to verify the channel cross sections upstream from Home Depot.

Caltrans provided record drawings for four structures on Hangtown Creek: Placerville Drive, Spring Street, Canal Street, and Bedford Avenue. The record drawings were used to define the geometry of these structures in the HEC-RAS model.



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Source: HJW, RBF
Date: 07/26/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

LiDAR Data Acquisition Limits

Exhibit 3

2.0 Hydrology

2.0 HYDROLOGY

A hydrologic study of the watershed was performed to develop appropriate runoff hydrographs for floodplain mapping and flood reduction project evaluations. The hydrology developed for the Plan was computed using a rainfall to runoff transformation computer program based on detailed sub-basin delineations and associated parameters that represent existing land use. The approach employed is consistent with the methodology presented in the El Dorado County Drainage Manual (EDCDM) and appears to calibrate reasonably well to observed conditions. The hydrologic results of this study can be considered to be a technical improvement over the discharge rates in the FEMA FIS.

This study computed peak discharges that are significantly higher than those listed in the FEMA FIS. One reason for the changes in discharges is because this new study is based on a detailed hydrology study while the FIS flows were based on a generalized approach. However, increased impervious area due to urbanization in the Hangtown Creek watershed since the FEMA FIS was completed in 1983 has altered the hydrology of the watershed and increased in discharge rates. To assess the significance of development on peak discharge rates, a model using EDCDM methodology was prepared based on undeveloped conditions. Instead of trying to precisely model 1983 watershed conditions, estimates of what portion of development induced flow increases to date that had already occurred by 1983 were made by interpolation. It was assumed that 70 percent of watershed development impacts on peak discharge rates occurred prior to 1983. This section of the report describes the analysis performed to calculate the discharges.

The U.S. Army Corps of Engineers (USACE) Hydrology Modeling System (HEC-HMS) software, Version 3.5 was used to compute the runoff hydrographs for this Plan based on precipitation events with 10-, 25-, 50-, and 100-year average annual recurrence intervals. The underlying process in setting up the HEC-HMS model and calculating key hydrologic parameters is explained in this section.

2.1 Stormwater Runoff Computation Methodology

The analysis used to determine discharge rates for this study follow the hydrograph method detailed in the EDCDM. This methodology was determined to be appropriate to use in the Hangtown Creek Watershed because it produced reasonable results and was able to reproduce observed conditions based on available data. Precipitation depths published in 2011 in NOAA Atlas 14 are consistent with rainfall depths determined using the EDCDM methods.

2.1.1 <u>FEMA Hydrology</u>

The procedure used by FEMA to determine discharge rates was described in the FIS as:

The discharge for the streams studied in detail were determined based on an adaption of the Sacramento District of the U.S. Army Corps of Engineers procedures and involved a regional approach using an S-curve hydrograph developed for the area, computation of unit hydrographs for each sub-basin, formulation of a U.S. Army Corps of Engineers Standard Project Flood as the basis for rainfall distribution, computation of a Standard project Flood for each sub-basin, and application of ratios of the 10-, 50-, 100-, and 500-year events to the Standard project event. These ratios were derived by the U.S. Army Corps of Engineers as a result of regional studies.

As indicated, a regional approach was applied. This suggests that the FEMA flows were not based on watershed specific characteristics such as soil types and impervious area. Documentation for the FEMA hydrology was not available.

2.1.2 El Dorado County Drainage Manual Procedures

The EDCDM hydrograph method follows the Soil Conservation Service (SCS) curve number and unit hydrograph method described in the SCS *National Engineering Handbook* (1971), commonly referred to as NEH-4. An updated version of NEH-4 was produced in the 1990s, but the procedures have not changed. The basic approach is to determine the appropriate rainfall depths and distribution, compute the portion of the rainfall that becomes runoff (this is referred to as effective rainfall), transform the effective rainfall to runoff hydrographs based on the SCS unit hydrograph and lag time for the sub-watershed areas, estimate baseflow contribution, and combine the appropriately routed sub-watershed hydrographs to develop cumulative hydrographs at combination points through the entire watershed. Key details of the process are included in this report.

2.2 Project Hydrology

Determination of 10-, 25-, 50- and 100-year runoff hydrographs for the purposes of this project involved the following key steps which are described in detail in the following sections:

1. Flow path identification and watershed delineation
2. Establishing precipitation depths
3. Determining SCS curve numbers and impervious area estimates
4. Estimating sub-watershed times of concentration
5. Application of baseflow
6. Stream routing
7. Model verification

2.2.1 Flow Path Identification and Watershed Delineation

The new topographic data that was obtained as part of this project was used to delineate watersheds, identify flow paths and determine the elevations and slopes used in the hydrology study. To efficiently use the LiDAR data, a terrain surface was developed by importing the LiDAR files into Global Mapper software. The resultant terrain in Global Mapper Grid (GMG) format was exported with a 6.25-foot grid spacing for watershed analyses. This is functionally equivalent to a Digital Elevation Model (DEM) of the topographic data. The GMG format was used to create 2-foot contours in Global Mapper for areas where 1-foot contours were not available and these were exported as shapefiles.

The DEM was processed in HEC-GeoHMS software (from USACE) to generate sub-basins, stream lines, and the longest flow paths in each sub-basin. The underlying steps involved can be found in the documentation provided with HEC-GeoHMS. Though the automated process is a good first step in the delineation process, the data was checked using aerial imagery and contour data. The computer generated boundaries were revised:

1. To better align with ridge lines observed from the contours,
2. To be consistent with observed storm drainage features and an irrigation canal, and
3. To provide concentration points at key drainage features such as detention basins.

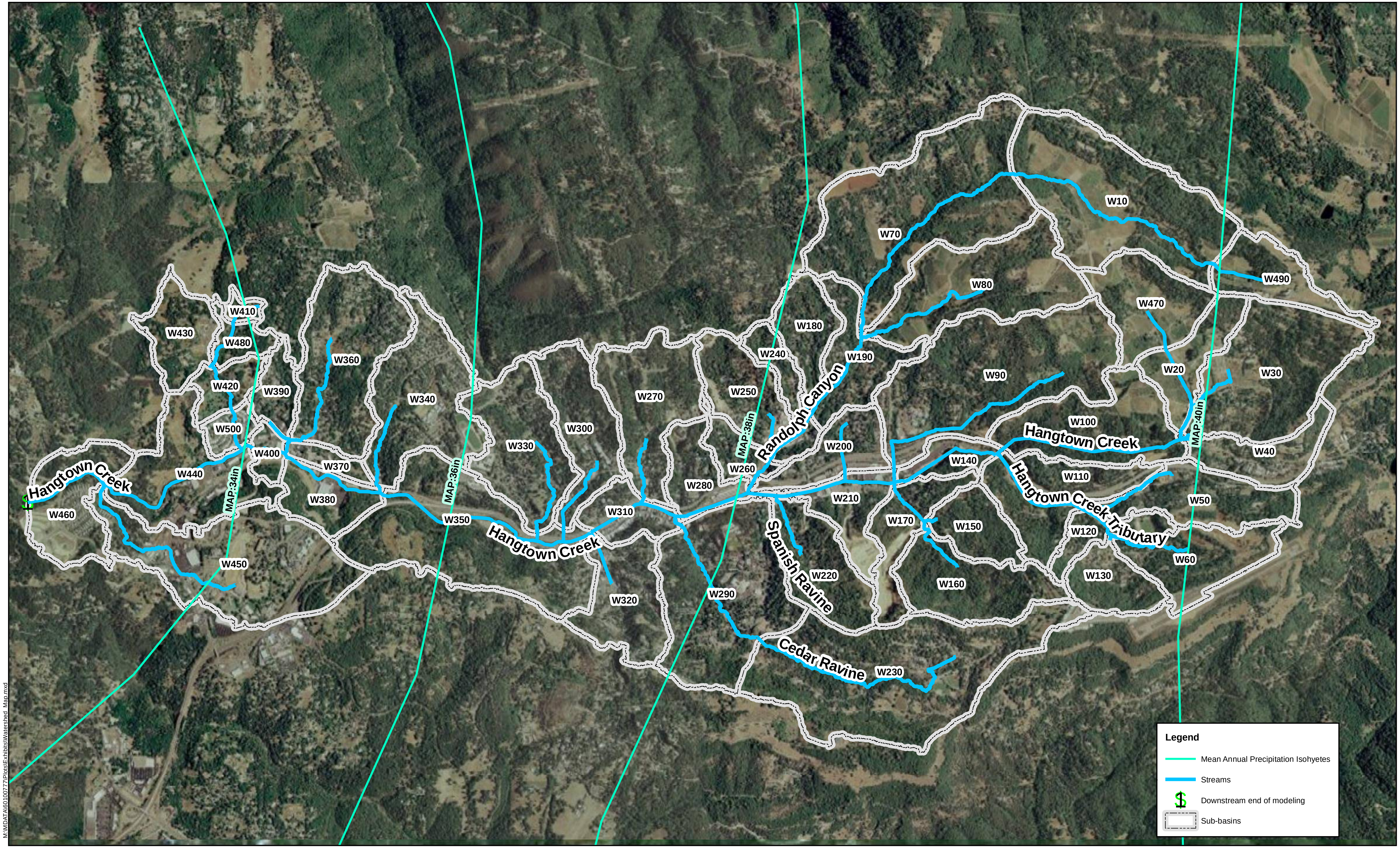
The watershed boundaries and stream lines are shown on Exhibit 4.

2.2.2 Establishing Precipitation Depths

The EDCDM provides precipitation depth information that defines the design storms used for project analysis. For this study, a storm duration of 24 hours was selected because it is appropriate for evaluating any detention basins that would be considered for flood control in the Hangtown Creek watershed. It is expected that any detention basin that would be used for flood control would be designed using the resultant hydrographs to just delay peak runoff by a few hours, at the most, and to drain relatively rapidly. Also, a duration of 24 hours is the standard duration for which the SCS precipitation distributions and loss rate methodology are applied.

The steps of the process used to assign the design storm depths are:

1. Locate each sub-basin on the mean annual rainfall map included in the EDCDM,
2. Correlate the mean annual precipitation to the appropriate 24-hour rainfall depths from the Tables in Appendix 2.2 of the EDCDM, and
3. Apply the SCS Type 1A design storm temporal distribution because the watershed is higher than an elevation of 1640 feet.



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Feet
0 1,000 2,000 4,000

Source: ESRI, RBF, EDCDM
Date: 08/17/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Streams, Sub-basins and Mean Annual Precipitation in Hangtown Creek Watershed

Exhibit 4

Appendix 2.2 of the EDCDM includes a map of *Mean Annual Rainfall for El Dorado County, California*. Contours of equal mean annual rainfall, referred to as isohyets or isohyetal lines, that are shown on the EDCDM map in the vicinity of Placerville were digitized into an ESRI shapefile format and a continuous raster surface by linearly interpolating between the different isohyets. These isohyets are shown on Exhibit 4. The mean annual rainfall value for each sub-basin was determined from the mean annual rainfall surface by using ArcGIS to find the value at the centroid of each sub-basin. The corresponding 24-hour rainfall depths for 10-, 25-, 50- and 100-year storm events were then interpolated from the EDCDM rainfall tables that are also in Appendix 2.2 of the EDCDM. The resultant rainfall values at each sub-basin are listed in Table 1.

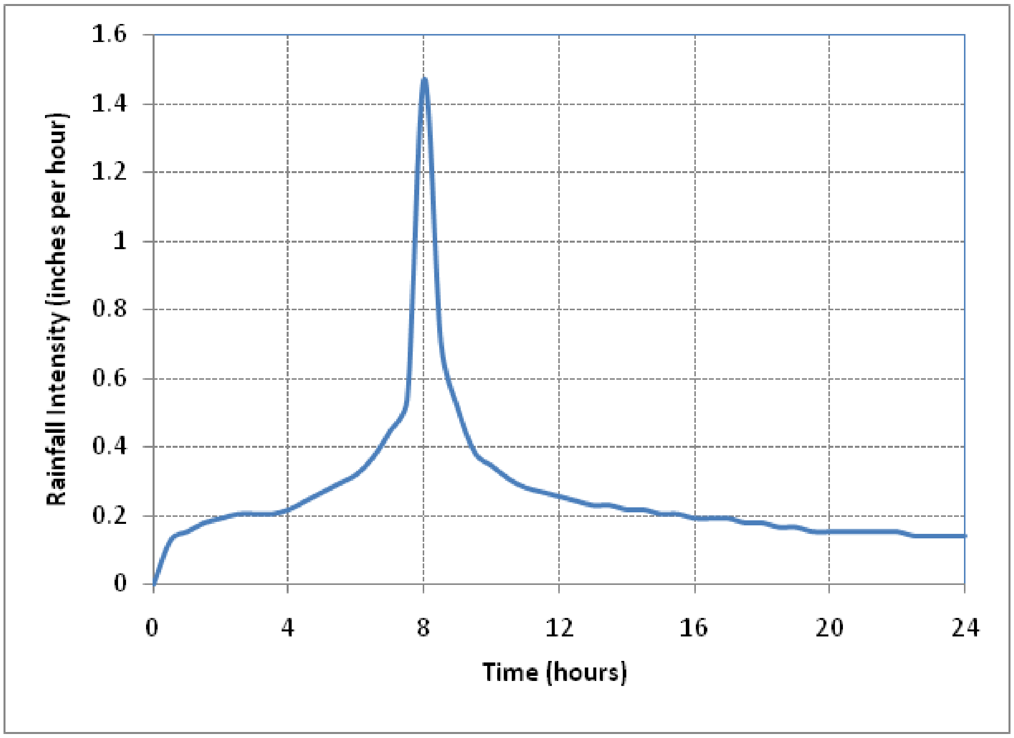
Table 1: Mean Annual Precipitation Depth and 24-hour Design Storm Rainfall Depths.

Basin Name	Mean Annual Precipitation (in)	10-year depth (in)	25-year depth (in)	50-year depth (in)	100-year depth (in)
W10	39.49	5.06	5.94	6.57	7.17
W20	39.93	5.10	5.99	6.62	7.23
W30	40.16	5.13	6.02	6.65	7.27
W40	40.17	5.13	6.02	6.65	7.27
W50	40.00	5.11	6.00	6.63	7.25
W60	40.02	5.11	6.00	6.63	7.25
W70	38.58	4.97	5.83	6.45	7.04
W80	38.91	5.00	5.87	6.49	7.09
W90	39.07	5.02	5.89	6.51	7.11
W100	39.56	5.07	5.95	6.57	7.18
W110	39.41	5.05	5.93	6.55	7.16
W120	39.65	5.08	5.96	6.59	7.20
W130	39.70	5.08	5.96	6.59	7.20
W140	38.94	5.01	5.87	6.49	7.10
W150	39.10	5.02	5.89	6.51	7.12
W160	39.09	5.02	5.89	6.51	7.12
W170	38.79	4.99	5.86	6.47	7.07
W180	38.15	4.93	5.78	6.39	6.98
W190	38.40	4.95	5.81	6.42	7.02
W200	38.42	4.95	5.81	6.43	7.02
W210	38.55	4.97	5.83	6.44	7.04
W220	38.44	4.95	5.81	6.43	7.02
W230	38.80	4.99	5.86	6.48	7.08
W240	38.11	4.92	5.78	6.39	6.98
W250	37.87	4.90	5.75	6.36	6.94

Basin Name	Mean Annual Precipitation (in)	10-year depth (in)	25-year depth (in)	50-year depth (in)	100-year depth (in)
W260	37.93	4.90	5.75	6.36	6.95
W270	37.25	4.84	5.68	6.28	6.86
W280	37.71	4.88	5.73	6.33	6.92
W290	38.05	4.92	5.77	6.38	6.97
W300	36.80	4.79	5.62	6.22	6.79
W310	37.27	4.84	5.68	6.28	6.86
W320	37.43	4.85	5.70	6.30	6.88
W330	36.42	4.75	5.58	6.17	6.74
W340	35.57	4.67	5.48	6.06	6.62
W350	36.16	4.73	5.55	6.13	6.70
W360	34.85	4.60	5.39	5.96	6.51
W370	34.80	4.59	5.39	5.96	6.51
W380	34.82	4.59	5.39	5.96	6.51
W390	34.23	4.53	5.32	5.88	6.43
W400	34.24	4.53	5.32	5.88	6.43
W410	34.02	4.51	5.29	5.86	6.40
W420	33.91	4.50	5.28	5.84	6.38
W430	33.78	4.49	5.27	5.82	6.36
W440	33.84	4.49	5.27	5.83	6.37
W450	34.28	4.54	5.33	5.89	6.43
W460	33.44	4.46	5.23	5.78	6.32
W470	39.69	5.08	5.96	6.59	7.20
W480	33.97	4.51	5.29	5.85	6.39
W490	40.16	5.13	6.02	6.65	7.27
W500	33.95	4.51	5.29	5.85	6.39

To compute runoff hydrographs, the 24-hour rainfall depths must be distributed in intervals over the 24-hour period. , Because the watershed is higher than 1640 feet above mean sea level, the SCS Type IA design storm temporal distribution was used, as prescribed in Section 2.3.3 of the EDCDM. To properly compute peak flow conditions considering the basin lag times, a 6-minute rainfall data time interval was used. Figure 1 illustrates the rainfall patterns applied to one of the sub-basins. In practice, a unit distribution is scaled for each sub-basin based on the 24-hour depth determined to be appropriate for it based on its centroid location relative to the isohyetal lines.

Figure 1: Example of SCS Type IA Rainfall Pattern Applied to Sub-Basins



The National Weather Service published the NOAA Atlas 14 Precipitation Frequency Estimates during the preparation of this Plan. The NOAA Atlas 14 precipitation depths in the watershed vary by only about 3 to 5 percent from the depths published in the EDCDM. Therefore, the values in the EDCDM were determined to be appropriate for use in this study.

2.2.3 <u>Determining SCS Curve Numbers and Impervious Area Estimates</u>

The amount of rainfall that becomes runoff is dependent on ground cover, the capacity of the underlying soil to absorb water, the amount impervious area, and whether or not runoff from the impervious area is directed onto adjacent pervious ground or along concentrated flow paths connected to the creek. This study uses Soil Conservation Service (SCS) methodology as described in the EDCDM and the United States Department of Agriculture (USDA) *Urban Hydrology for Small Watersheds Technical Release 55*, dated June 1986 (TR-55), to compute the amount of rainfall that becomes runoff. The amount of rainfall that becomes runoff is also referred to as effective rainfall. The parameters used to determine effective rainfall are a function of hydrologic soil group and land use.

A GIS overlay of the watershed sub-basins, the hydrologic soil groups, vegetative cover, and land use polygons was created to perform this study. This allowed parameters to be computed for each sub-element and then to be computed for each sub-basin using an area weighted average.

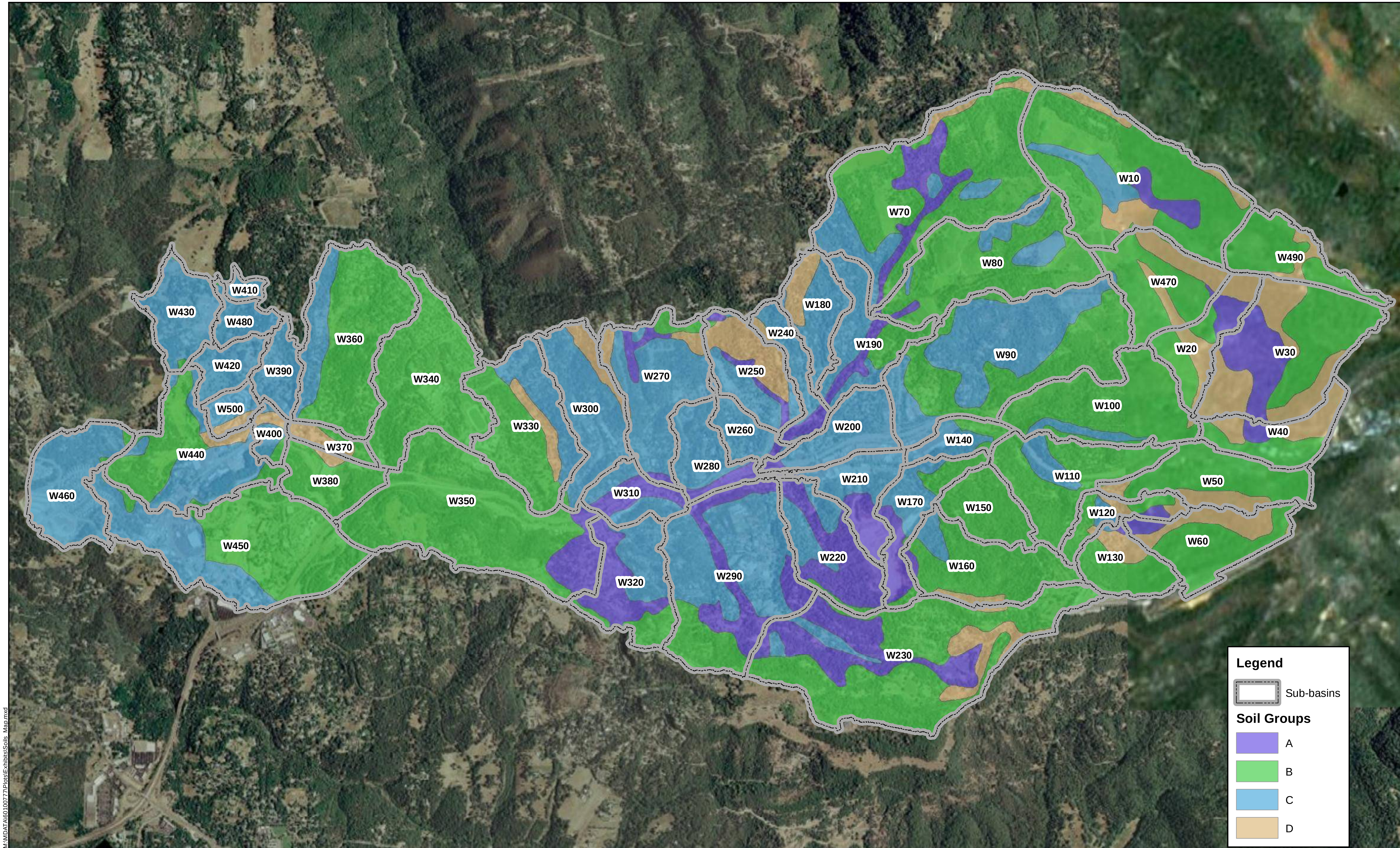
A SCS Curve Number (CN) was assigned for pervious areas based on the hydrologic soil group and pervious area ground cover. Curve numbers were then modified based on the non-directly connected impervious area to obtain a composite curve number for each element. Then a composite curve number was computed for each watershed sub-basin based on a weighted average of the elements in each sub-basin. This sub-basin composite curve number was then used to compute the runoff from the portion of each sub-basin that was not identified as being directly connected impervious area.

2.2.3.1 Hydrologic Soil Groups

Tabular and spatial soils data showing the SCS hydrologic soil groups were obtained from the Natural Resource Conservation Service (NRCS). Table 2 describes the hydrologic soil groups and lists the curve number used for the pervious areas as part of the composite curve number calculation. The spatial extents of the various hydrologic soil groups are shown on Exhibit 5.

Table 2: NRCS SCS Hydrologic Soil Groups

Hydrologic Soil Group	Description
A	Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravel.
B	Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well-drained to well-drained soils with moderately fine to moderately coarse textures.
C	Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water, or soils with moderately fine to fine texture.
D	Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.



RBF CONSULTING

0 1,000 2,000 4,000 Feet

Source: RBF, NRCS
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Hydrologic Soil Groups

Exhibit 5

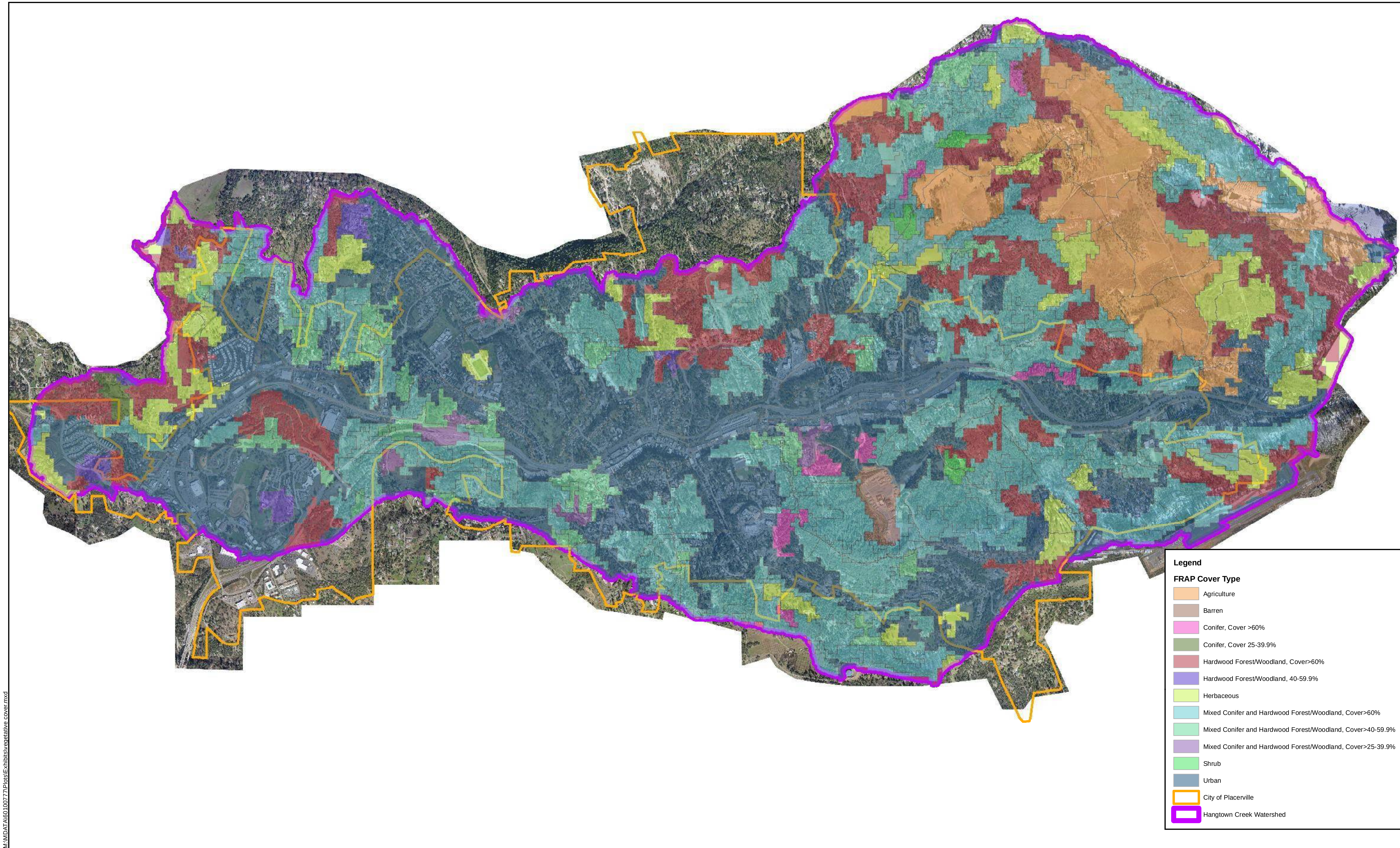
2.2.3.2 Pervious Area Curve Numbers

Curve numbers for pervious areas are a function of hydrologic soil group and ground cover. Spatial ground cover data was obtained from the Fire and Resource Assessment Program (FRAP) of the California Department of Forestry and Fire Protection. The cover type for each was determined from the ―Cover‖ attribute in the tabular data and verified with aerial imagery. Hydrologic conditions of fair or poor were estimated using the Wildlife Habitat Relationships Tree Density attribute. Exhibit 6 illustrates the spatial extent of the various FRAP ground cover classifications. The FRAP cover types were matched to TR-55 cover types and hydrologic conditions to be able to assign curve numbers to areas. Tables 2-2a, 2-2b and 2-2c of TR-55 list appropriate values for combinations of ground cover and hydrologic soils groups that occur in the Hangtown Creek watershed. The areas designated by FRAP as ―Agriculture‖ were inspected by aerial imagery to determine if row crops or orchard was the prevailing cover type. Table 3 shows the correlation between the FRAP cover types and the TR-55 cover types used to generate curve numbers.

Table 3: FRAP and TR-55 Cover Types Correlation

FRAP Cover Type	TR-55 Cover Type and Hydrologic Soil Condition
Agriculture	Row Crop, Poor
Agriculture	Orchard, Fair
Barren	Open Space, Fair
Conifer, Cover>60%	Woods, Fair
Conifer, Cover 25 to 39.9%	Woods, Poor
Hardwood Forest/Woodland, Cover >60%	Woods, Fair
Hardwood Forest/Woodland, Cover 40-59.9%	Woods, Poor
Herbaceous	Open Space, Fair
Mixed Conifer and Hardwood Forest/Woodland, Cover >60%	Woods, Fair
Mixed Conifer and Hardwood Forest/Woodland, Cover 40 to 59.9%	Woods, Poor
Mixed Conifer and Hardwood Forest/Woodland, Cover 25 to39.9%	Woods, Poor
Shrub	Brush
Urban	Open Space, Fair

Exhibit 7 shows the TR-55 vegetation classifications used for pervious area curve number assignments and Table 4 lists the curve number values applied in this study.



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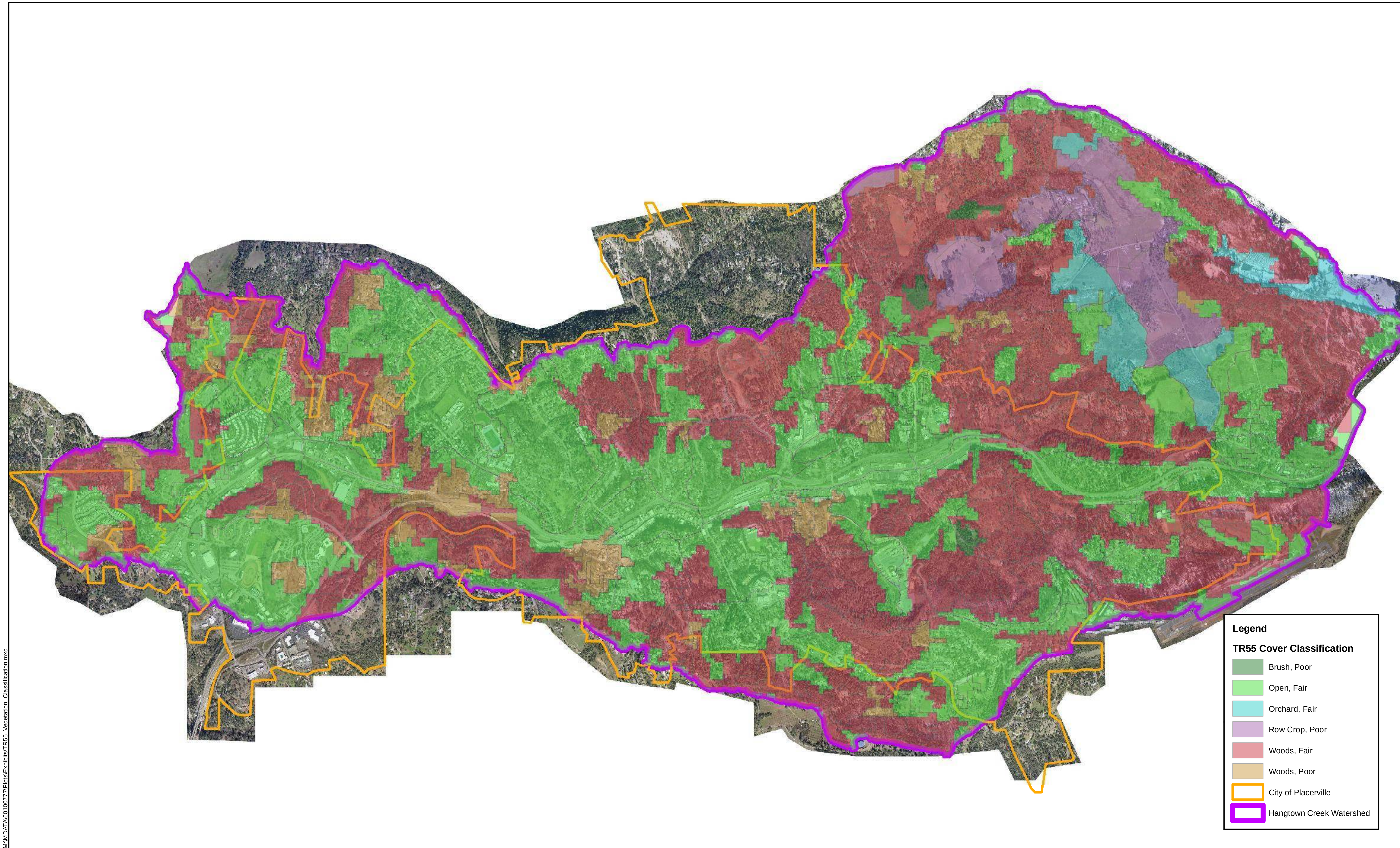
RBF CONSULTING

Source: HJW, RBF, FRAP CDFFP
Date: 07/26/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

FRAP Ground Cover Classification

Exhibit 6



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RBF CONSULTING

0 0.25 0.5 1 Miles

Source: HJW, RBF, FRAP CDFFP
Date: 07/26/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

TR-55 Vegetation Classifications

Exhibit 7

Table 4: Pervious Area Curve Numbers

Cover Type	Hydrologic Condition	A	B	C	D
Row Crop	Poor	72	81	88	91
Orchard	Fair	43	65	76	82
Woods	Poor	45	66	77	83
Woods	Fair	36	60	73	79
Brush	Poor	48	67	77	83
Open Space	Fair	49	69	79	84

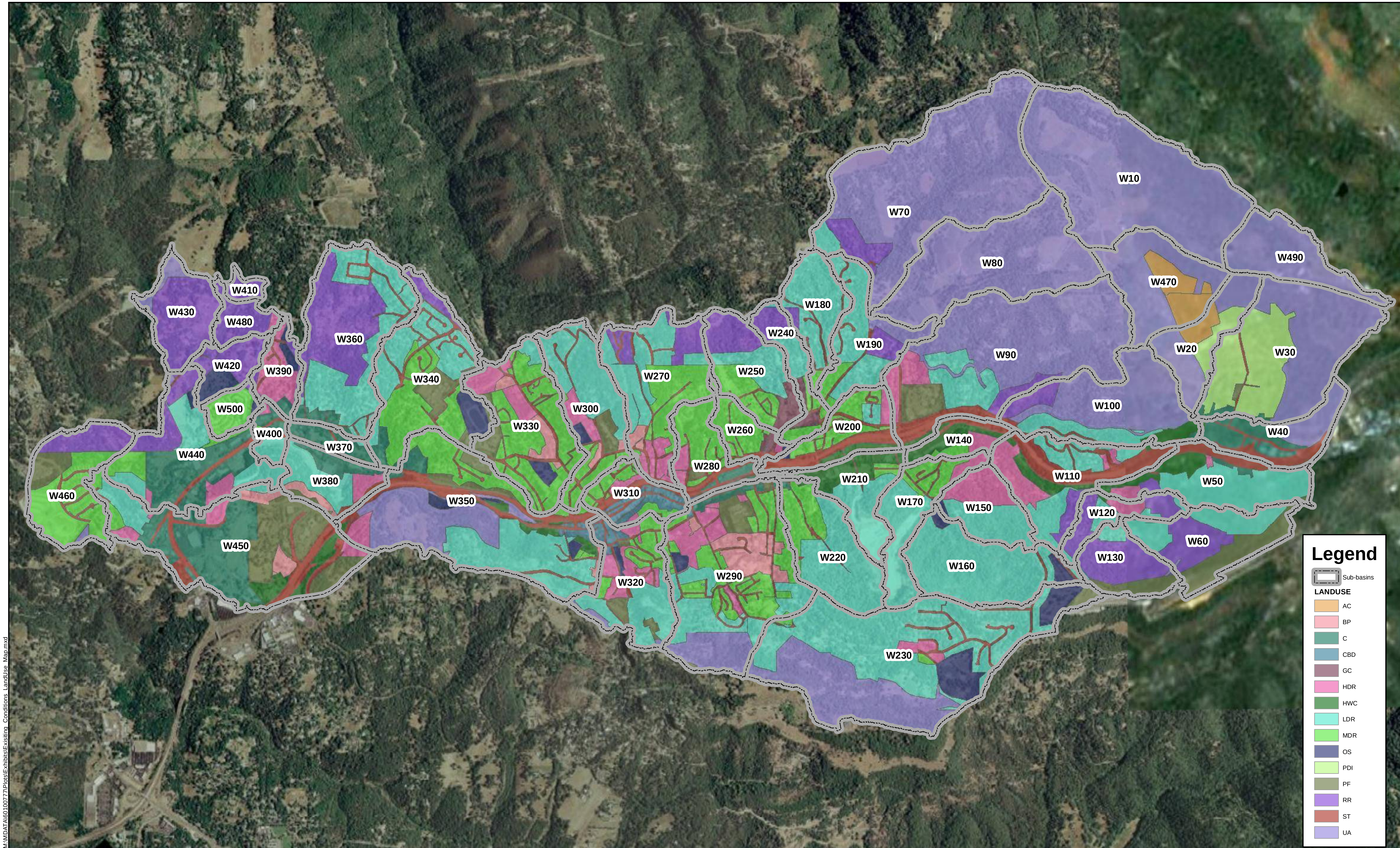
2.2.3.3 Impervious Area Estimates

Existing land use data provided by the City and the aerial imagery obtained as part of this project were used to assign values for imperviousness by land use. The land use of unincorporated areas was categorized as open space based on examination of the aerial imagery. The extent of the various land uses are shown on Exhibit 8.

Average values for impervious area as a percentage of total area and average values for directly connected impervious area (DCIA), also as a percentage of total area, were estimated and listed in Table 5.

Table 5: Impervious Area Rates by Land Use

Land use	Description	Impervious Area (%)	DCIA (%)
AC	Agriculture	2	0
BP	Business Park	70	63
C	Commercial	50	40
CBD	Central Business District	90	90
GC	General Commercial	85	76.5
HDR	High Density Residential	50	30
HWC	Highway Commercial	85	76.5
LDR	Low Density Residential	20	0
MDR	Medium Density Residential	35	17.5
OS	Open Space	2	0
PDI	Planned Development Industrial	10	0
PF	Public Facilities	30	15
RR	Rural Residential	15	0
ST	Streets	95	85.5
UA	Unincorporated Areas	10	0



RBF CONSULTING

Feet
0 1,000 2,000 4,000

Source: RBF, City of Placerville
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Existing Conditions Land Use

Exhibit 8

Composite curve numbers were computed for combined pervious and non-directly connected impervious area (NDCIA) based on the area weighted average of the pervious area curve numbers and a curve number of 100 applied to the NDCIAs. Impervious area was computed for each GIS overlay element as the element area times the impervious area percentage listed in Table 5. The DCIA was computed for each GIS overlay element as the element area times the DCIA percentage listed in Table 5. The NDCIA was computed for each GIS overlay element as the impervious area minus the DCIA. The sum of the products of curve number and area values for each sub-basin were divided by the sum of the pervious areas and NDCIAs for each sub-basin to determine the curve numbers listed in
Table 6.

Table 6: Sub-basin Areas and Curve Numbers

Basin	Total Area (Sq. Miles)	DCIA Percent	NDCIA Percent	Pervious Area CN	Composite CN
W10	0.420	0.0	10.0	67.6	70.9
W20	0.107	0.0	8.8	63.6	66.7
W30	0.308	1.9	10.0	68.6	74.0
W40	0.106	37.0	10.0	70.0	78.9
W50	0.158	26.1	16.3	70.0	80.6
W60	0.154	5.2	16.8	66.3	72.4
W70	0.474	0.2	10.6	66.3	71.5
W80	0.281	0.0	10.0	67.9	71.1
W90	0.435	8.8	12.1	68.4	74.9
W100	0.222	7.5	12.0	66.1	75.8
W110	0.184	39.5	15.2	68.9	80.2
W120	0.065	6.0	17.0	65.0	72.1
W130	0.091	3.8	15.5	64.7	72.6
W140	0.054	61.1	12.2	70.8	83.1
W150	0.092	12.1	19.6	64.9	73.3
W160	0.181	2.8	18.4	69.1	77.7
W170	0.113	10.9	18.8	65.3	75.4
W180	0.097	9.0	18.7	75.9	84.4
W190	0.131	8.8	16.8	64.8	74.6
W200	0.104	50.7	13.5	69.7	78.8
W210	0.102	32.3	15.2	65.5	75.7
W220	0.154	8.6	18.8	59.5	73.6
W230	0.523	4.4	14.7	68.0	75.9
W240	0.071	19.4	15.9	69.3	78.7
W250	0.132	17.1	16.1	67.8	77.1
W260	0.057	33.0	15.2	71.5	81.9
W270	0.210	16.8	17.2	70.7	78.9
W280	0.096	38.3	14.6	66.0	78.5
W290	0.369	24.6	14.7	64.4	73.7

Basin	Total Area (Sq. Miles)	DCIA Percent	NDCIA Percent	Pervious Area CN	Composite CN
W300	0.162	15.3	17.5	75.9	83.1
W310	0.077	60.6	10.1	67.7	75.6
W320	0.163	23.2	16.2	60.0	69.6
W330	0.215	27.8	15.6	73.1	80.6
W340	0.251	15.8	15.8	67.1	75.8
W350	0.420	22.4	13.6	62.1	74.3
W360	0.278	5.7	16.9	68.1	76.6
W370	0.034	46.5	10.3	71.9	83.8
W380	0.094	26.6	15.4	63.3	74.9
W390	0.058	26.3	15.5	76.3	82.1
W400	0.032	28.8	14.8	77.4	83.2
W410	0.015	0.0	15.0	73.0	77.0
W420	0.057	1.9	11.1	76.6	82.6
W430	0.104	0.4	14.2	72.0	76.6
W440	0.229	22.4	14.4	71.4	77.3
W450	0.374	39.5	12.5	69.1	78.7
W460	0.156	15.6	16.0	75.3	80.2
W470	0.162	0.0	7.8	69.6	71.4
W480	0.043	2.5	15.4	72.2	77.7
W490	0.113	0.0	10.0	72.8	75.5
W500	0.036	15.9	16.5	79.5	83.6

2.2.3.4 Application of Curve Numbers

Curve numbers are used to calculate effective rainfall (R_e, in inches), which is that portion of incident rainfall (R_i, in inches) that becomes runoff, according to Equation 1:

Equation 1: Effective Rainfall

$R_e = \frac{(R_i - 0.2S^2)}{R_i + 0.8S}$, where $S = \frac{1000}{CN} - 10$

The USACE computer program HEC-HMS calculates the effective rainfall for each calculation time interval based on the cumulative rainfall and uses the effective rainfall calculations for runoff routing analysis.

2.2.4 *Estimating Sub-Basin Times of Concentration*

Sub-basin times of concentration are dependent on flow lengths, slopes, roughness and flow depths. The components of sub-basin time of concentration are:

1. Sheet flow
2. Shallow concentrated flow
3. Channel flow travel time

The process used to compute time of concentration follows SCS methodology described in TR-55 and the EDCDM for the longest flow path in each sub-basin identified using topographical data. Locations where flow transitions from sheet flow to shallow concentrated flow and from shallow concentrated flow to channel flow were estimated using imagery, topography, TR-55 guidance and engineering judgment.

2.2.4.1 Sheet Flow

Travel time of sheet flow can be estimated with the following simplified solution to the kinematic-wave equation, Equation 2:

Equation 2: Sheet Flow Travel Time

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

In which:
T_t = sheet flow travel time
n = overland-flow roughness coefficient (a value of 0.6, which is an average of dense and light underbrush values was used)
P_2 = 2-year 24-hour rainfall depth (estimated to be equal to 4.2 inches)

Slopes for sheet flow were calculated from the upstream and downstream elevations, identified from the DEM, and length of the sheet flow.

2.2.4.2 Shallow Flow

Shallow concentrated flow paths were categorized as paved or unpaved by examining the aerial imagery and travel times were calculated using Equation 3 or Equation 4, as appropriate:

Equation 3: Unpaved Shallow Concentrated Flow

$$V = 16.1345\sqrt{S_0}$$

Equation 4: Paved Shallow Concentrated Flow

$$V = 20.3283\sqrt{S_0}$$

In which:
V = shallow-concentrated flow velocity in feet per second
S_0 = slope in feet/foot

2.2.4.3 Channel Flow

For each of the sub-basins, the LiDAR terrain was examined in Global Mapper to identify whether or not there was a well-defined channel. If no well-defined channel was found, total travel time was computed as the sum of sheet flow and shallow-concentrated flow travel times, only. Where well-defined channels were observed, dimensions of approximated trapezoidal sections were used for the flow routing calculations. Channel dimensions observed using Global Mapper at a typical location along the flow path was assumed to define the channel for the entire flow length. Table 7 lists sub-basins having channel flow and the corresponding channel shape and dimensions.

Table 7: Channel Dimensions for Channel Flow Travel Time

Sub-basin ID	Bottom Width (feet)	Depth (feet)	Side Slope (H:1V)
W50	3	4	3
W60	4	8	2
W70	6	5	2
W90	8	4	2
W100	13	6	3
W110	5	10	3
W140	20	11	1.5
W160	60	10	3
W190	5	5	2
W210	7	13	2
W250	4	5	4
W290	5	5	1
W310	10	10	1
W340	5	6	2
W350	15	15	1
W390	5	6	2
W400	8	8	2
W410	5	5	2
W420	20	9	2
W430	5	3	2
W440	5	8	2
W450	3	4	2
W460	10	9	2

Travel times for channel flow were calculated using Manning's Equation, Equation 5, with an n-value of 0.08.

Equation 5: Manning's Equation

$$V = \frac{1.49}{n} r^{2/3} s^{1/2}$$

In which:
V = channel flow velocity
r = hydraulic radius (area/wetted perimeter)
s = channel slope (feet/foot)

2.2.4.4 Lag Times

The time of concentration for each sub-basin was computed as the sum of sheet flow, shallow-concentrated flow and channel flow travel times which were based on flow lengths and calculated velocities. Lag time, defined as the time from the centroid of the precipitation excess (effective rainfall) to the time of the peak of the unit hydrograph (USACE EM 1110-2-1417, 1994), is calculated as 60 percent of the time of concentration according to an empirical relationship developed by SCS. Lag time is used in the transformation process to compute sub-basin runoff from effective rainfall. Lengths and slopes for sheet flow, shallow concentrated flow and channel flow, and the resultant lag times for each sub-basin are listed in Table 8.

Table 8: Lag Times

Sub-basin ID	Sheet Flow		Shallow Flow			Channel Flow		Lag Time (hrs)
	Length (ft)	Slope (ft/ft)	Paved or Unpaved (P or U)	Length (ft)	Slope (ft/ft)	Length (ft)	Slope (ft/ft)	
W10	256	0.2180	U	5851	0.0475	0	0.0000	0.489
W20	238	0.1334	U	2711	0.0383	0	0.0000	0.386
W30	213	0.2905	U	4750	0.0658	0	0.0000	0.354
W40	262	0.3545	P	5711	0.0582	0	0.0000	0.372
W50	109	0.5136	P	2424	0.0592	2603	0.0392	0.229
W60	276	0.0756	U	3940	0.0941	390	0.0384	0.483
W70	276	0.0492	U	2135	0.1084	5947	0.0403	0.608
W80	358	0.1464	U	6281	0.0677	0	0.0000	0.574
W90	274	0.0891	U	2285	0.0712	5638	0.0511	0.530
W100	146	0.1137	U	1992	0.1053	3195	0.0384	0.300
W110	307	0.1802	P	3206	0.0522	2336	0.0273	0.422
W120	230	0.2235	P	2157	0.1063	0	0.0000	0.246
W130	195	0.3518	U	2067	0.1518	0	0.0000	0.195
W140	278	0.1072	P	343	0.4766	2569	0.0243	0.345

Sub-basin ID	Sheet Flow		Shallow Flow			Channel Flow		Lag Time (hrs)
	Length (ft)	Slope (ft/ft)	Paved or Unpaved (P or U)	Length (ft)	Slope (ft/ft)	Length (ft)	Slope (ft/ft)	
W150	152	0.4047	U	2502	0.1275	0	0.0000	0.181
W160	339	0.0342	U	3250	0.1481	1661	0.0449	0.661
W170	178	0.1300	P	4447	0.0931	0	0.0000	0.314
W180	277	0.2202	U	3957	0.0707	0	0.0000	0.378
W190	164	0.0989	U	2590	0.0859	2023	0.0228	0.356
W200	144	0.2087	U	1649	0.0542	0	0.0000	0.209
W210	186	0.0242	P	2509	0.1025	661	0.0336	0.469
W220	250	0.1606	U	4534	0.0854	0	0.0000	0.395
W230	387	0.0129	U	10432	0.0564	0	0.0000	1.366
W240	274	0.2812	U	4017	0.0444	0	0.0000	0.399
W250	112	0.2563	P	2691	0.0466	1333	0.0225	0.246
W260	99	0.1528	P	2039	0.0826	0	0.0000	0.173
W270	245	0.0725	P	3829	0.0359	0	0.0000	0.483
W280	227	0.1295	P	3287	0.0523	0	0.0000	0.355
W290	210	0.1474	P	1179	0.1347	3973	0.0230	0.361
W300	0	0.0000	U	7058	0.0478	0	0.0000	0.333
W310	135	0.0178	U	836	0.0124	1764	0.0095	0.476
W320	282	0.1906	U	4726	0.0786	0	0.0000	0.416
W330	247	0.0081	U	6014	0.0277	0	0.0000	1.140
W340	337	0.0267	P	2543	0.0672	2281	0.0392	0.738
W350	268	0.0426	P	3769	0.0906	5409	0.0155	0.623
W360	366	0.0345	P	6388	0.0477	0	0.0000	0.828
W370	158	0.1050	P	1878	0.0597	0	0.0000	0.256
W380	222	0.0918	P	1998	0.1020	0	0.0000	0.318
W390	0	0.0000	U	1492	0.0505	1475	0.0236	0.108
W400	131	0.2286	U	921	0.2538	342	0.0023	0.164
W410	0	0.0000	U	339	0.0543	570	0.0070	0.046
W420	238	0.0269	U	554	0.1042	1135	0.0437	0.493
W430	192	0.0418	U	1216	0.0617	1884	0.0211	0.454
W440	199	0.0694	U	658	0.0666	3287	0.0374	0.359
W450	209	0.1387	P	1131	0.0327	7616	0.0332	0.494
W460	0	0.0000	U	0	0.0000	1837	0.0170	0.043
W470	110	0.1034	P	2752	0.0441	0	0.0000	0.253
W480	281	0.0611	U	1396	0.0395	0	0.0000	0.452
W490	230	0.0370	U	4281	0.0647	0	0.0000	0.568
W500	200	0.0579	U	898	0.0240	0	0.0000	0.355

2.2.5 *Application of Baseflow*

A recession baseflow method was adopted for this study to account for runoff from groundwater bodies in the form of seepage and springs. An initial discharge for each storm event was assigned to individual sub-basins as specified in Table 2.6.1 on page 2-25 of EDCDM. HEC-HMS defines the recession constant parameter as a ratio of current recession flow to the recession flow one day earlier, whereas the recession constant provided in Table 2.6.1 of the EDCDM was for HEC-1, which defined the parameter as a ratio of current recession flow to the recession flow one hour later (*RTIOR*=1.05). Therefore the HEC-HMS recession constant was obtained using Equation 6.

Equation 6: HEC-HMS Baseflow Recession Constant

$$Recession\ Constant = \frac{1}{(RTIOR)^{24}} \quad \text{where,} \quad RTIOR = \frac{Current\ recession\ baseflow}{Recession\ flow\ 1\ hour\ later}$$

The parameter specifying when to reset the base flow in HEC-HMS model was assigned per Table 2.6.1 (*QRCSN*=0.1). Table 9 lists the baseflow values applied in this study.

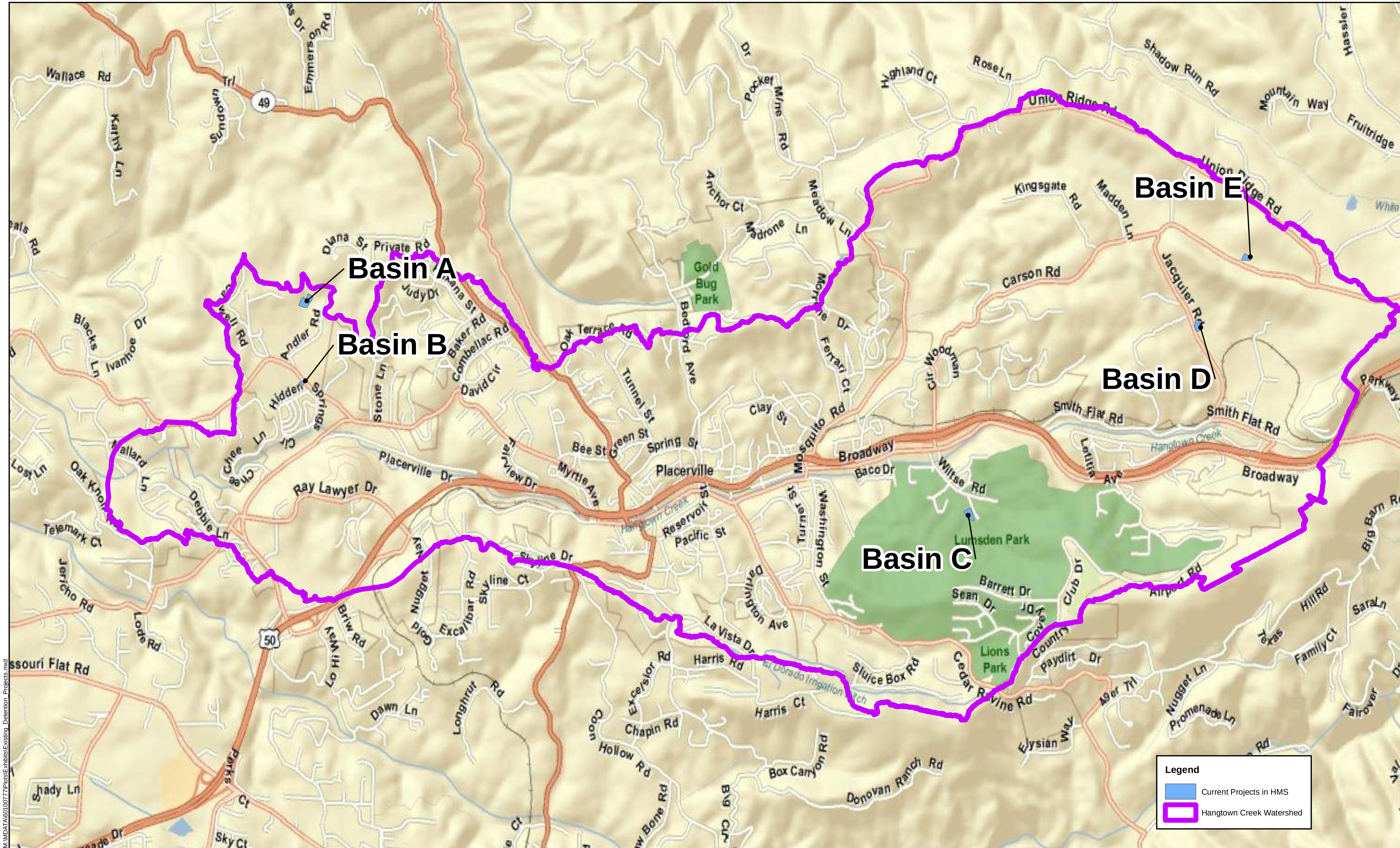
Table 9: Baseflow in Cubic Feet per Second (cfs) per Square-Mile

Return Period (years)	Initial Flow (cfs/square-mile)
10	5
25	6
50	8
100	10

2.2.6 *Stream and Storage Routing*

The Muskingum-Cunge method was chosen for stream flow routing. A trapezoidal channel with 5-foot bottom width, 2:1 side slopes, and a Manning's roughness coefficient of 0.08 was assumed for all of the main channels in the watershed. The impact of a major culvert across Highway 50 at the downstream of Cedar Ravine's confluence with Hangtown Creek was tested using Modified Puls routing and a storage-discharge relationship developed from HEC-RAS results but no significant impact was found.

Storage routing was used to simulate the function of some features that detain runoff. Exhibit 9 shows the locations where storage routing was used in the HEC-HMS to simulate existing detention basins. Detention-discharge curves for Lumsden Lake Detention Basin were taken from *Drainage Report: Lumsden Ranch Development Project* by Domenichelli and Associates (DA). Other detention-discharge curves were developed by examining the topography and estimating the size and configuration of the outlet structures.



RBF CONSULTING

0 0.25 0.5 1 Miles

Source: HJW, RBF
Date: 07/26/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Existing Detention Basins Modeled in HEC-HMS

Exhibit 9

2.2.7 *Model Verification*

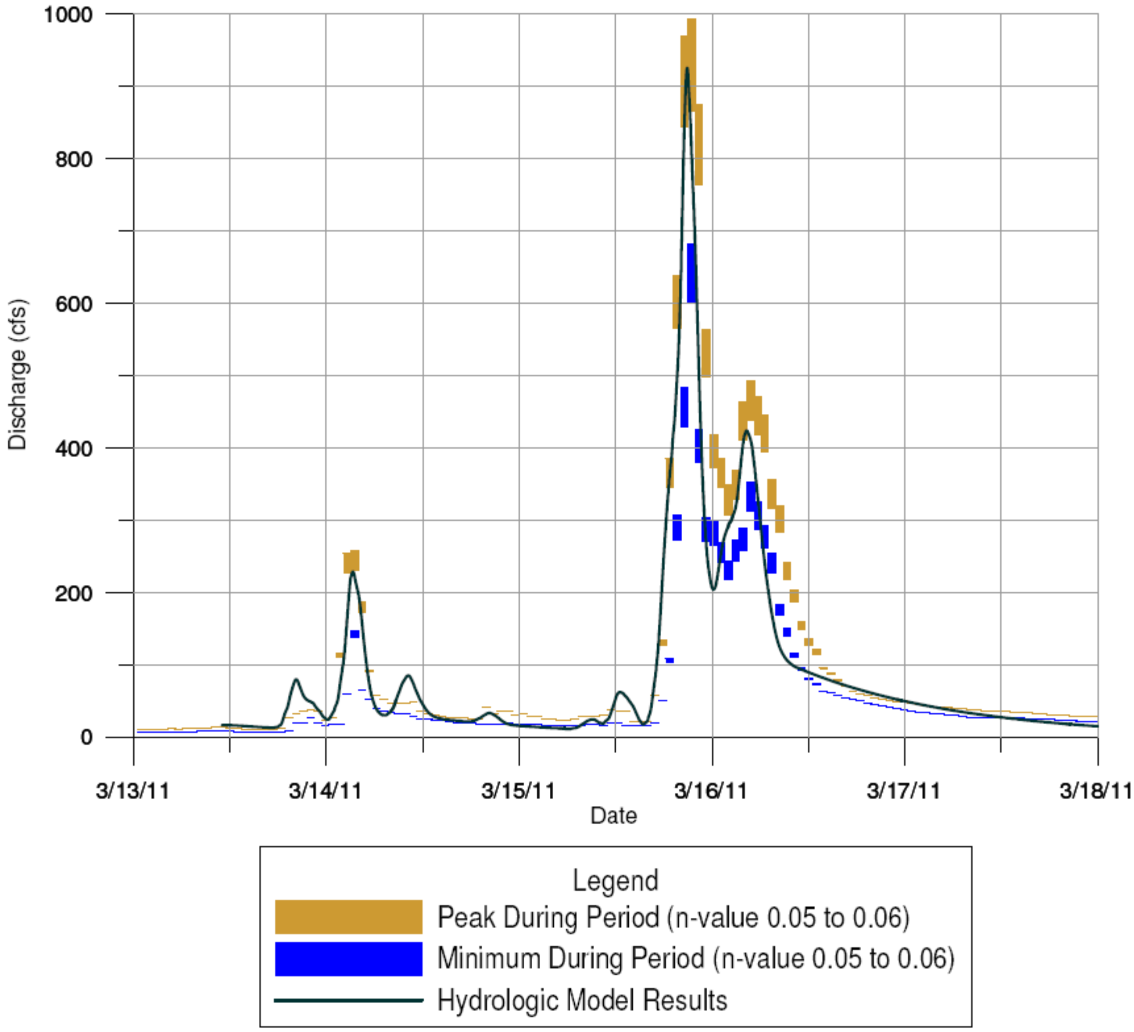
Stream gage and rainfall data for a location near the downstream end of the watershed by Mallard Lane was provided by the City and used to verify that the hydrologic model computes appropriate results. This data included precipitation depths and minimum and maximum stream depths measured in intervals of approximately 1-hour time, and a rating curve relating stream depths to discharges, but only for discharges less than 40 cfs. Two rating curves were computed using HEC-RAS to estimate the stage-discharge relationship above 40 cfs. One of the high flow rating curves was computed using a Manning's n-value (a measure of stream roughness) of 0.05 and one curve was computed using a value 0.06 within the vicinity of the stream gage. The two rating curves were used to provide a range of observed maximum and minimum stream flows during each time interval based on observed stream depth.

A 3-day calibration event was selected as March 13 and 16, 2011. The total rainfall depth at the waste water treatment plant over the 3 day period was 2.8 inches, with the peak 24-hours receiving 1.8 inches. According to NOAA Atlas 14, the return period for this rainfall event was less than 1 year. NOAA Atlas 14 indicates a 1-year, 24-hour rainfall depth of 2.6 inches, and a 1-year, 3-day depth of 3.9 inches. The EDCDM indicates a depth of 2.9 inches for a 24-hour event with a 2.33 year return period.

The rainfall gage is located in an area with a MAP of 32 inches but the precipitation across the entire watershed ranges from a MAP of 34 to 40 inches. Therefore, the observed precipitation data was scaled based on the percentage increase in rainfall depth of a 2-year 24 hour storm event as calculated for each sub-basin. The initial base flow was assigned as 2 cfs per square mile per EDCDM. Each sub-basin was assigned with the scaled precipitation depending on its proximity to the corresponding MAP isohyet. The verification process involved using the hydrologic model to compute discharges from the measured precipitation depths and comparing these discharges to discharges based on stream depths and the rating curves to demonstrate that the hydrologic model produces appropriate discharges.

The simulated discharges at HEC-HMS Node J30, the node closest to the stream gage location, were compared with the observed maximum and minimum discharges developed using the two rating curves. The comparison of the simulated discharges and the range of observed maximum and minimum discharges are illustrated in Figure 2.

Figure 2: Comparison between Modeled and Observed Flows



Peak flows fall within the range of maximum flows for this calibration event, thereby validating the hydrologic model. Some hydrograph timing differences can be seen in the comparison, but there are reasonable explanations that can account for these differences. Various factors that may have contributed to these differences include:

- Observed flows were provided at approximate one-hour intervals while HEC-HMS results were tabulated at a 5-minute interval,
- Modeled discharges from scaled precipitation depths may not have realistically represented the precipitation pattern and time distribution through the watershed,
- Uncertainties in the extrapolation of the stream gage rating curve due to topography and the Manning's n-value of the channel, and
- Gage data fluctuations due to wave conditions.

For the purposes of this study, the agreement between observed and simulated discharges was considered reasonable and no further changes were made to the hydrologic parameters in the HEC-HMS model.

Table 10 compares the discharges at the upstream and downstream ends of Hangtown Creek and at the confluences of Cedar Ravine and Randolph Canyon tributaries computed for this study to flows listed in the FEMA FIS and a drainage study of the Lumsden Ranch development project performed by Domenichelli and Associates (DA) in 2007. Discharges from the current study are higher than the ones estimated by FEMA but lower than the ones computed by DA.

In conclusion, discharges from this comprehensive watershed plan are within the range of discharges found from previous studies and the discharges estimated from the current model match reasonably well with the observed flow data.

Table 10: Flow Comparison between Current Study, FEMA and Lumsden Ranch Study

Description	HMS Node	Tributary Area (sq. mi)			10 year			25 year			50 year			100 year		
		RBF	FEMA	DA	RBF	FEMA	DA	RBF	FEMA	DA	RBF	FEMA	DA	RBF	FEMA	DA
Near Upstream end of Hangtown creek	J72	1.56	1.6	-	496	270	-	645	-	-	755	410	-	864	490	-
Broadway and Mosquito Road	J56	2.79	-	-	855	-	996	1115	-	-	1306		-	1495	-	1850
From Randolph Canyon	J104	4.57	4.7	-	1205	760	-	1599	-	-	1890	1140	-	2180	1380	-
From Cedar Ravine	J33	5.55	5.6	-	1385	1000	1750	1839	-	-	2174	1540	-	2508	1870	3219
Main Street and Sacramento Street	J18	7.80	8	-	1947	1580	-	2568	-	-	3027	2410	-	3483	2920	-
Downstream end of Hangtown creek	J30	8.56	-	-	2102	-	-	2769	-	-	3261	-	-	3749	-	-

2.2.8 *Estimated Impacts of Development*

To estimate the portion of the increase in flow rate since 1983 caused by development, a pre-development model was created in HEC-HMS, assuming no impervious area in the watershed. A detailed analysis of pre-development or 1983 conditions is beyond the scope of this report. However, general assumptions can be made for the entire watershed to estimate the impact development has had on peak flow rates in Hangtown Creek.

The existing conditions HEC-HMS model developed using the EDCDM was used as a base to develop the pre-development model. All impervious area was removed, pervious area curve numbers were recalculated to exclude DCIA, and detention routing from constructed detention basins was removed.

The population of Placerville has grown from approximately 7,000 to 10,000 during the period from 1983 to 2011. If it is assumed that 70 percent of development in the watershed that has occurred to date had occurred by 1983, and that the development that occurred by 1983 caused 70 percent of the increase in peak flow rate from pre-development conditions to existing conditions, then the estimated 1983 flows can be interpolated as flows from the pre-development model plus 70 percent of the increase in flows from the pre-development model to the existing conditions (2011) model. Estimated 1983 peak flow rates interpolated from the pre-development and existing conditions model results are shown in Table 11. The flow rates used in the FIS are also included for comparison.

From this brief analysis of 1983 development conditions, it can be concluded that differences in peak flow rates from the flows that are in the effective FIS and peak flow rates listed in Tables 10 and 11 are caused not only by the impacts of development, but also by differences in the modeling assumptions and methodology used to generate the peak flow rates.

Table 11: Estimated 100-Year Flow Increases from 1983 to 2011

Location	HMS Node	FEMA (1983)	RBF (Estimated 1983)	RBF (2011)	Estimated Increase Due to Development from 1983 to 2011	Percentage Increase Due to Development
	(ID)	(cfs)	(cfs)	(cfs)	(cfs)	%
Near Upstream end of Hangtown creek	J72	490	837	864	27	3
Broadway and Mosquito Road	J56	-	1430	1495	65	5
From Randolph Canyon	J104	1380	2111	2180	69	3
From Cedar Ravine	J33	1870	2406	2508	102	4
Main Street and Sacramento Street	J18	2920	3300	3483	183	6
Downstream end of Hangtown creek	J30	-	3545	3749	204	6

3.0 Hydraulic Model

3.0 HYDRAULIC MODEL

3.1 Effective FEMA Model

The effective FEMA model, created in 1981, was obtained from FEMA in HEC-2 printout format. The HEC-2 printout was digitized and converted to HEC-RAS to acquire cross section, structure, and n-value information. Several significant features were not included in the 1981 model, including bridges, buildings, and culverts on Hangtown Creek. Also, construction activities have occurred since the development of the 1981 model including the rerouting of Hangtown Creek at Home Depot on Placerville Drive. The downstream end of the effective model was near Pierroz Road. The upstream portion of the model extended upstream of Highway 50 adjacent to Smith Flat Road.

HEC-2 printouts of other models were also acquired from FEMA, including reaches for Randolph Canyon, Cedar Ravine, and Hangtown Creek Tributary (extending upstream from Smith Flat Road, adjacent to Broadway). These models were not converted to HEC-RAS as the scope of this Plan based on available budget was limited to the main reach of Hangtown Creek. Future studies may extend updated hydraulic modeling on the tributaries.

3.2 Existing Conditions Model

A new HEC-RAS model was created using topography developed from the 2011 LiDAR. Cross section locations were defined to appropriately capture hydraulically significant features in the topography, including bridges and culverts. Lettered cross sections from the FIS were also included for comparison. The HEC-RAS model was extended from just downstream of Pierroz Road to just downstream of Mallard Lane, adjacent to the wastewater treatment plant, to allow for use of the available gage data as part of the model calibration.

3.2.1 Stream Crossings

Where applicable, structural geometry of stream crossings was taken from the existing FEMA model, verified in the field, and tied to 2011 LiDAR elevations. For structures not included in the FEMA model, approximate field dimensions were taken and 2011 LiDAR elevations were used directly. This approach is consistent with the goal of the Plan to determine approximate flood risks and the potential impacts of projects. For future FEMA floodplain mapping, additional structure survey information may be required.

Field reconnaissance of Hangtown Creek was conducted in February 2011. All structures between Mallard Lane and Smith Flat Road were documented, photographed, and catalogued. Where applicable, basic field measurements such as bridge opening or culvert width, depth to invert from bridge deck, bridge deck thickness, pier width, and hand rail height were taken. Photographs of each structure are included on the DVD with this report. Table 12 lists the structures that were identified and if they are included in the FIS HEC-RAS model and the 2011 HEC-RAS model.

Table 12: Stream Crossings Identified on Hangtown Creek.

Location	2011 HEC-RAS Model Structure Name	Effective FIS Station	Included 2011 Model
Mallard Lane	HAN_100	Not included	Yes
Abandoned Bridge	HAN_150	Not included	No--hydraulically insignificant
Vicini Bridge	HAN_200	Not included	Yes
Pierroz Road	HAN_250	12742	Yes
Placerville Drive	HAN_260	13422.5	Yes
560A Placerville Drive Culvert	HAN_280	14318	Yes
Home Depot Lower Bridge	HAN_300	Not included	Yes
Home Depot Upper Bridge	HAN_310	Not included	Yes
Maytag Bridge	HAN_330	16026	Yes
Route 50 Culvert	HAN_350	Not included	Yes
Placerville Drive	HAN_370	Not included	Yes
Abandoned Railroad Bridge	HAN_380	19514	Yes
Canal Street	HAN_400	20157	Yes
Spring Street	HAN_420	20843	Yes
Carrows Culvert	HAN_430	Not included	Yes
Center Street	HAN_450	21584.5	Yes
Center Street Buildings and Parking Garage	HAN_460	Not included	No (combined with HAN-480)
Red Bridge	HAN_470	Not included	No (combined with HAN-480)
Building	HAN_480	Not included	Yes
Building	HAN_490	Not included	No
Bedford Avenue	HAN_500	22838	Yes
Town Hall Bridge	HAN_520	23377	Yes
Clay Street	HAN_540	23660	Yes
Dance Studio Patio	HAN_560	23907.5	Yes
617 Main Street (Orange Building)	HAN_570	24097	Yes
Locust Avenue	HAN_580	Not included	Yes
Medical Plaza Culvert	HAN_600	25390	Yes
Auto Shop Bridge	HAN_610	Not included	Yes
Mosquito Road	HAN_630	25677.5	Yes

Location	2011 HEC-RAS Model Structure Name	Effective FIS Station	Included 2011 Model
Rehab Center Culvert	HAN_640	Not included	Yes
McDonalds Lower Bridge	HAN_650	Not included	Yes
McDonalds Upper Bridge	HAN_660	26134.5	Yes
Rite Aid Culvert	HAN_680	Not included	Yes
Savemart Culvert	HAN_700	Not included	Yes
Pacific Telephone Company Drive	HAN_710	27420	Yes
Blairs Lane	HAN_730	27604	Yes
Mountain Democrat Building	HAN_750	Not included	No--included as obstruction in cross section
Wiltse Road	HAN_760	28642	Yes
Motel 9/Dollar Tree Culvert	HAN_800	30565	Yes
Auto Shop Culvert	HAN_820	30877.5	Yes
Sports Bar Culvert	HAN_830	Not included	Yes
Creekview Mini-Mart Culvert	HAN_840	31062.5	Yes
1606 Broadway Culvert	HAN_850	Not included	No-above upstream limit of modeling
Lower Hangtown Motel Culvert	HAN_870	Not included	No-above upstream limit of modeling
Hangtown Motel Culvert	HAN_880	Not included	No-above upstream limit of modeling
Upper Hangtown Motel Culvert	HAN_890	Not included	No-above upstream limit of modeling
Lower Taqueria Culvert	HAN_900	Not included	No-above upstream limit of modeling

3.2.2 *Manning's n-values*

Manning's n-values used in 1981 FEMA model were applied to the HEC-RAS model developed for the Plan. Manning's n-values vary from 0.015 to 0.06 in the channel and are 0.1 for the floodplain. A list of n-values is included in the Appendix.

3.2.3 *Flood Profiles and Floodplain Mapping*

Steady-state HEC-RAS model calculations were made for the 10-, 25-, 50-, and 100-year storm events. Flow rates in HEC-RAS were applied at appropriate locations where discharge rates are expected to change based on peak flow rates from HEC-HMS results.

Table 13 lists the flows that were used in the HEC-RAS model. The flood profiles for the various recurrence interval events are included at the end of this report.

The HEC-RAS flood profiles were used to delineate approximate floodplain extents using automated methods based on the 2011 topography and the 100-year storm event profile. These approximate floodplain delineations are provided on the preliminary maps included at the end of this report. It should be noted that detailed floodplain mapping consistent with FEMA mapping requirements is beyond the scope of this study.

Table 13: HEC-RAS Steady-State Flows

Description	HMS Node	Tributary Area (sq mi)	HEC-RAS Cross Section	Flows (cfs)			
				10-year	25-year	50-year	100-year
Upstream extent of RAS	J72	1.56	24688.91	496	645	755	864
	J75	2.43	22107.41	740	966	1132	1297
	J39	2.64	20973.32	814	1060	1239	1418
	J56	2.79	19287.39	856	1116	1306	1495
At Confluence with Randolph Canyon	J104	4.57	18659.54	1205	1599	1890	2180
At Confluence with Cedar Ravine	J33	5.55	16930.52	1385	1838.6	2174	2508
	J109	5.76	16047.31	1451	1924.7	2274	2622
	J44	6.00	14674.64	1517	2010	2374	2736
	J118	6.17	14038.60	1563	2070	2444	2816
	J121	6.38	13413.72	1615	2134	2516	2896
	J50	7.05	9147.76	1787	2359	2781	3200
	J53	7.51	6628.95	1884	2485	2929	3370
	J18	7.80	5616.92	1947	2569	3027	3483
	J136	8.41	1774.25	2082	2743	3230	3715
	J30	8.56	65.15	2103	2769	3261	3749

4.0 Potential Flood Control Projects

4.0 POTENTIAL FLOOD CONTROL PROJECTS

Significant flood risks exist in the watershed, especially in the Hangtown Creek stream corridor. Updated flood profiles and floodplain delineation improves the understanding of this risk. One of the purposes of the Plan is to identify flood risks and determine if there are potentially feasible detention basins that could be built to reduce the flood risk to structures along Hangtown Creek.

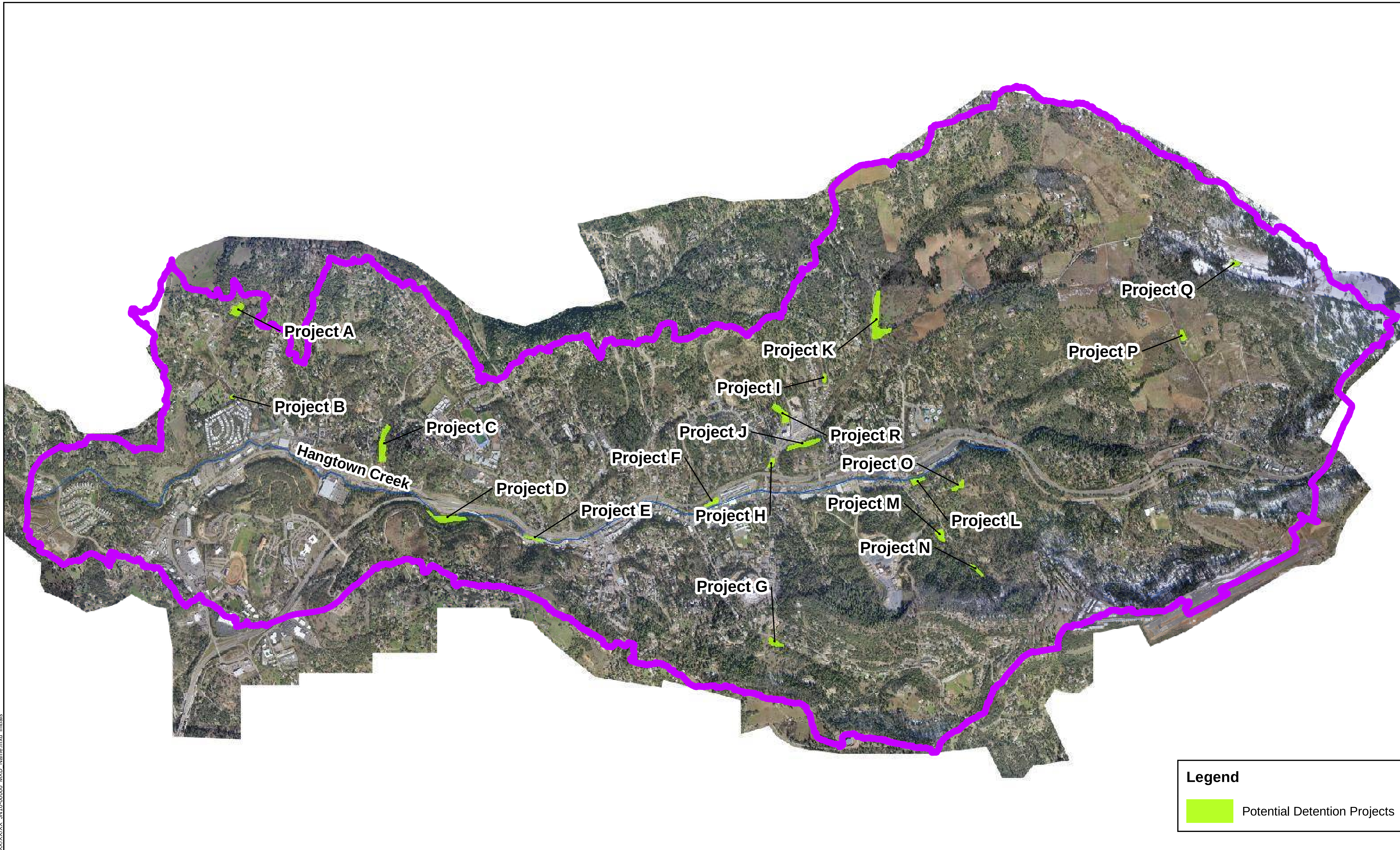
To reduce flood depths and frequency of flooding, potential flood control projects were identified in the Hangtown Creek Watershed. Due to topographic constraints and dense development above and adjacent to the stream, there are few sites for projects that could detain enough runoff volume to significantly reduce peak flows at locations where damage would be expected.

Eighteen sites throughout the Hangtown Creek watershed were investigated for peak flow reduction potential. The sites consisted of on-channel detention, off-channel detention, and upper watershed detention projects. Maximum potential projects volumes were estimated based on topographic constraints at each identified site. Some potential basin volumes were limited by a maximum embankment height of 25 feet to remain out of the jurisdiction of the Division of Safety of Dams.

Limiting factors for the sites were identified and include upstream tributary area, existing buildings or other development within the potential storage area, inadequate volume for achieving peak flow reduction, and timing of local peak flows relative to the peak flow in the main channel. The projects were classified as not feasible or potentially feasible based on the limiting factors.

Ten of the eighteen projects were determined to be infeasible based on initial investigation. Potential stage-storage-discharge relationships were developed for the remaining projects, namely Projects C, I, L, M, N, P, Q and R. These relationships were used to model the potential projects in HEC-HMS to quantify peak flow reduction potential. Additionally, Project L, an off-channel detention basin along Hangtown Creek upstream from Wiltse Road, was modeled in HEC-RAS as it is within the Hangtown Creek HEC-RAS model extents. A short segment of stream upstream and downstream of the project was modeled using the unsteady flow capabilities of HEC-RAS to better assess the potential for flow diversions to reduce the peak flow rate in Hangtown Creek.

The results of the HEC-HMS and HEC-RAS models with the potential projects were analyzed to determine which projects might have enough flow reduction potential to be feasible from flow reduction perspective. Only Projects C, I, L and R were determined to be potentially feasible. The potential project sites are shown in Exhibit 10 and listed in Table 14 along with 100-year peak flow reduction estimates for projects modeled in HEC-HMS (and HEC-RAS in the case of Project L) and limiting factors that impact the feasibility of each project.



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PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Hangtown Creek Potential Detention Projects

Exhibit 10

Table 14: Potential Project Sites for Peak Flow Reduction

Project Letter	Project Location	Project Description	Feasibility	Potential 100-year Peak Flow Reduction (cfs)	Limiting factor(s)
A	Colin Road	Increase size of existing storage area	Not Feasible due to small tributary area	--	Very small tributary area
B	Cold Springs Road near Cemetery	Increase size of existing storage area	Not Feasible	--	Existing buildings prevent additional storage volume
C	Placerville Drive Behind Polaris Store	Create storage area in watershed above commercial development	Potentially Feasible, although limited area of benefit	100	Impact is limited to parcels downstream of project, near Home Depot on Placerville Drive
D	Culvert at Highway 50	On-channel storage on Hangtown Creek	Not Feasible	--	Existing culvert has limited capacity and detains flows; no additional volume available
E	Abandoned Railroad bridge adjacent to Placerville Drive	Reduce bridge opening to restrict flows	Not Feasible	--	Not enough volume to reduce peak flow rate substantially.
F	Downstream of Locust Lane	Off-channel detention basin along Hangtown Creek	Not Feasible due to planned development	--	Site planned for future parking area
G	Adjacent to Cedar Ravine Road near Quail Drive	Create storage area with reduced culvert and embankment	Not Feasible	--	Local peak flow occurs after peak flow in system. No system flow reduction potential.
H	Parking area west of Mosquito Road and north of Highway 50	Off-channel detention basin along Randolph Canyon Tributary	Not Feasible	--	Site planned for future development
I	Between Morrene Drive and Hawks Landing Court	Create new storage area with embankment and outlet control	Potentially Feasible	50	Detention storage limited by existing development.
J	Mosquito Road near Clay Street	On-channel detention area	Not Feasible	--	Not enough volume for significant flow reduction

Project Letter	Project Location	Project Description	Feasibility	Potential 100-year Peak Flow Reduction (cfs)	Limiting factor(s)
K	NA	On-channel detention area	Not Feasible	--	Existing buildings prevent storage volume
L	Upstream from Wiltse Road	Off-channel detention basin along Hangtown Creek	Potentially Feasible	100	Detention storage volume limited by upstream water surface elevation and downstream channel elevation
M	Existing Lumsden Ranch Detention Basin	Increase size of existing storage area	Not Feasible	40	Very tall embankment required for limited storage volume
N	Upstream from Lumsden Ranch Detention Basin	Create new storage area in watershed	Not Feasible due to small tributary area	25	Very tall embankment required for limited storage volume
O	Near Lane Court	Create new storage area with embankment and outlet control	Not Feasible	--	Very small tributary area
P	Jacquier Road near Partridge Place Road	Increase size of existing storage area	Not Feasible	20	Limited detention volume and local peak flow occurring about the same time as system peak flow
Q	Carson Road near Union Ridge Road at existing pond	Increase size of existing storage area	Not Feasible	20	Limited detention volume and local peak flow occurring about the same time as system peak flow
R	Behind El Dorado Irrigation District Building	Create new storage area in watershed above development	Potentially Feasible	50	Flow reduction potential limited by tributary area and embankment height

4.1 Projects Identified as Potentially Feasible

4.1.1 Project C – Placerville Drive behind Placerville Polaris

Watershed 490 of the HEC-HMS model drains about 72 acres into Hangtown Creek just downstream of the Highway 50 culvert. The area behind the existing businesses along Placerville Drive near Middletown Road presents an opportunity for storage of runoff. Part of the project site is shown in Photograph 7.

Photograph 7: Site of Potential Project C near Placerville Drive and Highway 50

The potential 100-year peak flow reduction in Hangtown Creek is 100 cfs just downstream of Highway 50, but would impact only the parcels below Highway 50. Although benefit from this project would be limited to a relatively small number of parcels, it could provide some flood reduction in an area currently at risk of flooding and may be feasible as mitigation for future development impacts.

4.1.2 Project I – Between Morrene Drive and Hawks Landing Court

Between Hawks Landing Court and Morrene Drive, near Mosquito Road, an open area exists that has the potential to capture upper watershed runoff and delay it from entering the Randolph Canyon tributary by building embankments around the area. The project site is shown in Photograph 8. The project size is limited by houses surrounding the project site. The potential 100-year peak flow reduction at the confluence of Hangtown Creek and the Randolph Canyon tributary is 50 cfs.

Photograph 8: Potential Project Site between Morrene Drive and Hawks Landing Court

4.1.3 Project L – Upstream from Wiltse Road

Upstream from Wiltse Road is a potential off-channel detention basin behind an existing auto repair shop near Orchard Lane where Hangtown Creek daylights from a culvert. The location is shown in Photograph 9.

Photograph 9: Location of Potential Off-Channel Detention Basin near Wiltse Road

The project consists of excavating about 5-acre feet of floodplain storage and creating an off-channel detention basin with an embankment and outlet structure. The project would be a passive off-channel system that would allow flow to enter the detention basin over a weir or other flow control device when the flow reaches a certain depth. The water would pond in the detention basin and be released through a small outlet with a flap gate so that the water would be detained until the floodwave passed, reducing the peak flow downstream from the project. The detention basin has the capacity to reduce peak flows by about 100 cfs downstream of the project. This project produces the largest 100-year peak flow reduction for the most parcels. The potential benefit of the project in reducing flood damages is presented in Section 5.0.

4.1.4 <u>Project R – Behind El Dorado Irrigation District Building</u>

The area just north of the El Dorado Irrigation District building on Mosquito Road has detention storage capacity for the 84 acres that drain to it. The potential project area is shown in Photograph 10.

Photograph 10: Potential project site near Mosquito Road behind El Dorado Irrigation District Building

An existing drainage inlet collects flows coming from the draw. An embankment spanning the width of the draw could be constructed to detain flows and produce peak flow reduction benefit. The site has a potential for a 50 cfs reduction of the 100-year peak flow at the confluence of Randolph Canyon Tributary and Hangtown Creek. The potential of this project to reduce flood damages is presented in Section 5.0.

4.2 Locations Determined to not have Potential to Support a Flood Damage Reduction Project

4.2.1 Project M – Existing Lumsden Ranch Detention Basin

The existing Lumsden Ranch Detention Basin could be enlarged with a higher embankment to increase the available storage volume. The project site is shown in Photograph 11.

Photograph 11: Existing Lumsden Ranch Detention Basin that could be Enlarged for Additional Detention Storage Capacity

The project has a potential of reducing the 100-year peak flow by about 30 to 50 cfs by raising the existing embankment by about 10 feet. Due to topographical constraints, this project is unlikely to be feasible and would require extensive modification of the existing detention basin for a marginal benefit. It is not recommended that this potential project be studied further.

The project was also analyzed to determine if a low-level outlet could be configured to lower the pond elevation by two or three feet prior to the wet season.

For the 10-year and 100-year events, the revised outlet configuration could provide about 5 cfs of local peak flow reduction upstream from Hangtown Creek. However, due to the timing of the hydrograph peaks, this option was determined to have a potential for increasing discharge in Hangtown Creek. Due to limited benefit and potential negative impacts, this project is not feasible.

4.2.2 *Project N – Upstream from Lumsden Ranch Detention Basin*

Upstream from the Lumsden Ranch Detention Basin, an embankment could be constructed across the draw to create a storage area to capture and detain runoff from about 50 acres of tributary area. The area is steep and wooded and has the potential for an approximate peak flow reduction of 25 cfs. The existing vegetation may present environmental constraints. This project is not recommended for further investigation due to limited potential benefit related to its limited tributary area.

4.3 Non-structural Alternatives

Other options for flood damage reduction are also available. Building elevation programs, buy-outs, relocation of structures, and other related programs may provide some damage reduction. These programs may not be viable in all cases in the City especially through the economically important downtown corridor, but may present viable opportunities on a case-by-case basis.

Low Impact Development (LID) measures that promote infiltration attempt to make the developed portions of a watershed mimic undeveloped conditions. LID measures such as pervious pavement, disconnected roof drains, and drainage swales provide opportunities for runoff to infiltrate prior to discharging into a creek or an underground storm drain system and reduce runoff volume. LID cannot reduce existing peak flows except where it is applied to areas undergoing redevelopment. LID should be incorporated into future development and redevelopment areas to reduce the impact of development on peak flow rates. However, the feasibility of some LID measures is limited due to the steepness of the terrain.

4.4 Projects Recommended for Flood Damage Reduction Analysis

Based on the analysis presented in this section, the opportunities for locating and constructing flood control projects to reduce the peak flow rates in Hangtown Creek are limited by topographical constraints and the proximity of existing development. Eighteen sites were analyzed and only four project sites (C, I, L, and R) provide peak flow reduction benefits in excess of 50 cfs for the 100-year event for. A 50 cfs peak flow reduction corresponds to a decrease in water surface elevation of only about 2 to 4 inches and a 100 cfs peak flow reduction produces a decrease in water surface elevation of about 3 to 6 inches through the areas of concern. The peak flow reduction for each of the projects at selected locations is presented in Table 15.

Table 15: Peak flow reduction in cubic feet per second from potential projects

Project	Hangtown Creek at Center Street				Hangtown Creek At Wastewater Treatment Plant			
	10-year	25-year	50-year	100-year	10-year	25-year	50-year	100-year
C	0	0	0	0	53	71	84	100
I	26	34	41	50	26	34	41	50
L	5	25	60	100	5	25	60	100
R	26	34	41	50	26	34	41	50
Net Combined	42	81	129	182	121	181	241	310

The net combined benefit at the waste water treatment plant is greater than the sum of the individual projects for the 50-year and 100-year events, but less than the sum of the individual projects for the 10-year and 25-year events due to effects of timing in the watershed.

Flood damage reduction analysis was performed for these four projects and the combination of the four projects and is presented in Section 5.2.

4.5 Biological Resource Summary

As part of the Hangtown Creek Comprehensive Watershed Plan, ECORP Consulting, Inc. (ECORP) conducted a biological resources assessment for the three sites of the potential projects determined to most likely be feasible. The purpose of the biological resources assessment is to assess the potential for occurrence of potential jurisdictional wetlands, special-status plants and wildlife species, or their habitat, within the proposed project sites. The full report by ECORP is included on the disk included with this report. Conclusions are include here, also.

The biological resource summary is preliminary in nature. The conclusions and recommendations presented in the report are based upon review of existing baseline data and a site reconnaissance visit. This assessment did not include a detailed tree/arborist survey, a formal wetland delineation in accordance with USACE, or determinate-level surveys to support Section 7 Consultation with either the U.S. Fish and Wildlife Service (USFWS) or the National Oceanic and Atmospheric Administration (NOAA)/National Marine Fisheries Services (NMFS).

Based upon the vegetation communities, habitats, and current site conditions, there are several potentially occurring special-status species that could occupy or periodically visit the proposed project sites.

The woodland and grasslands within the proposed project sites represent potentially suitable habitat for several special-status plant species including: Jepson's onion, Nissenan manzanita, Pleasant Valley mariposa-lily, Red Hills soaproot, Brandegee's clarkia, Parry's horkelia, brownish beak-rush, oval-leaved viburnum, and El Dorado

County mule ears. Surveys should be conducted during the appropriate season for these species. If special-status plants are found, the lead agency may require mitigation including (but not limited to) avoidance or transplantation.

Several special-status amphibians and reptiles have the potential to occur on-site including California red-legged frog, foothill yellow-legged frog, coast horned lizard and northwestern pond turtle. While the habitat within the proposed project sites is considered only marginally suitable for the special-status frogs and turtle, they may use the drainage corridors for dispersal. Habitat assessments or protocol level surveys may be required by resources agencies to determine the presence or absence of these species.

Preconstruction nesting bird surveys may be required during the CEQA process. If special-status bird nests are detected during these pre-construction surveys, avoidance buffers may be required while the nests are occupied.

Several species of bats have the potential to roost or forage within the proposed project sites. To avoid impacts to bats, tree removal should occur in April (prior to the establishment of nursery sites) or between August 15th and October 31st (after the young have been reared and prior to the onset of hibernation) of each calendar year.

While the proposed project sites are relatively small areas, certain habitat restoration and/or enhancement opportunities may exist. For example, the low-quality habitat within Project L, currently made up of non-native weedy plants and gravel/asphalt, could be replanted with a variety of native plants or enhancement of the aquatic habitat by integrating off-channel detention.

5.0 Project Benefit and Cost Analysis

5.0 PROJECT BENEFIT AND COST ANALYSIS

Project economic benefits and costs are evaluated to provide justification for project implementation. Flood damage reduction analysis is used to estimate of project benefits. Project design, permitting, construction and maintenance costs need to be considered as part of overall project costs. Projects are economically justifiable when economic benefits exceed overall costs. In some cases, even when estimated costs exceed expected project benefits, other factors can cause projects to be feasible by providing funding through grants or development impact mitigation programs.

5.1 Flood Damage Analysis Methodology

To quantify the potential fiscal benefits of the flood reduction projects, flood damage reduction analysis was performed for existing conditions and conditions with potentially feasible projects. The flood damage reduction analysis uses HEC-RAS 10-, 25-, 50- and 100-year steady-state profiles developed for existing conditions and proposed conditions with Projects C, I, L, and R. Estimated damage losses are developed using structure finished floor elevations, structure and content value, and damage-loss curves.

Parcel data from El Dorado County was used to determine parcels that could potentially receive benefit from the projects. The parcel data included structure valuation from the El Dorado County tax assessor (2010 structure values). Finished floor elevations for structures were estimated using the 2011 LiDAR elevations and field investigation by Sweeney Land Surveying.

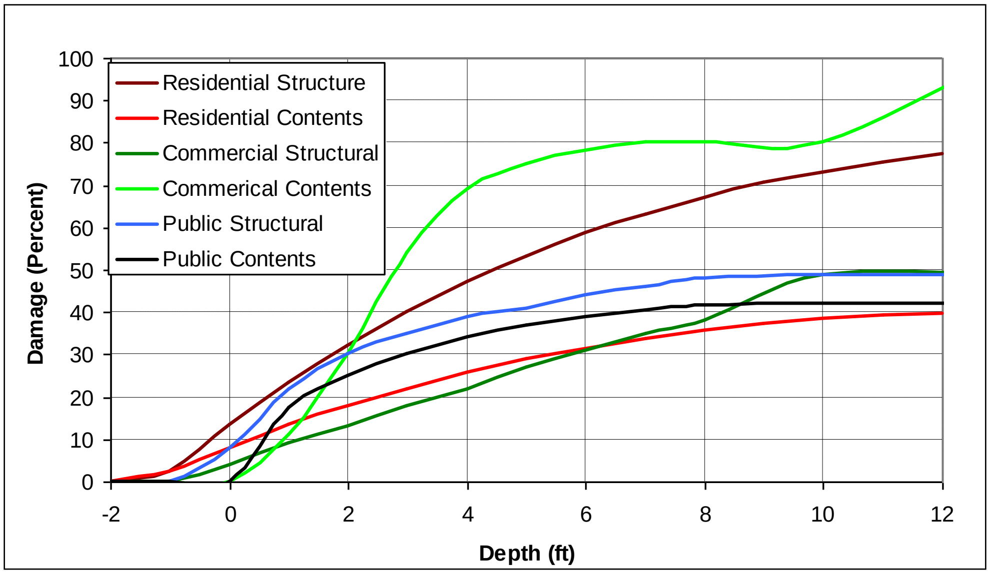
The flood damage analysis (FDA) was completed using HEC-FDA, a computer program developed by the USACE. HEC-FDA uses the stage and discharge data produced in HEC-RAS and structure information to develop stage-damage functions, discharge-exceedance probability functions, and stage-discharge relationships. These three functions are combined to create a damage-probability function.

To include uncertainty in exceedance probability, damage, and stage in the creation of the damage-probability function, numerical integration is required. HEC-FDA applies a Monte Carlo simulation to the damage-probability function to numerically integrate and derive expected annual damage. The Monte Carlo simulation uses an exceedance probability analysis of samples of the contributing variables (See HEC-FDA User's Manual for more in-depth description).

Depth-damage curves published by both USACE and FEMA were used in the FDA analysis (See USACE Economic Guidance Memorandum—EGM 04-01, *Generic Depth-Damage Relationships*, October 2003).

The depth-damage curves for residential, commercial, and public buildings are presented in Figure 3. The published depth-damage curves include standard deviations of error for residential structure and content damage curves.

Figure 3: Depth-Damage Curves used in Flood Damage Analysis



The structure value to content value ratio was assumed to be 0.50 for residential, commercial, and public buildings. Contents of structures may include equipment, furnishings, raw materials, and commercial inventory.

A value of plus or minus 0.5 feet was used for standard deviation of error of the 100-year stage for the existing conditions and with project scenarios to account for uncertainty in stage elevation. USACE EM 1110-2-1619, Engineering and Design-Risk Based Analysis for Flood Damage Reduction Studies, pg 5-5 lists a range of 0.3-0.6 feet for minimum standard deviations of error in stage when cross sections are based on field survey and Manning's n-value reliability is good to fair.

The results of the Flood Damage Analysis include estimates of total damage for various recurrence intervals and the equivalent annual damage.

5.2 Flood Damage Reduction Analysis of Potential Projects

The flood damage analysis was developed for existing condition as well as for the Projects C, I, L, and R and the combined damage reduction of the four projects. The limits of the flood damage analysis on Hangtown Creek are the same as the limits of the HEC-RAS model described in Section 3.0, namely, from the downstream limit of the Wastewater Treatment Plant, upstream to Smith Flat Road. Flood damage analysis outside of Hangtown Creek, including Cedar Ravine and Randolph Canyon tributaries, is beyond the scope of this study.

Project C, watershed storage near Placerville Drive and Highway 50 provides a 100-year peak flow reduction of 100 cfs. The potential project impacts only the parcels downstream from Highway 50.

Project L, the off-channel detention basin upstream from Wiltse Road, produces the largest 100-year peak flow decrease of 100 cfs. The potential project is configured to divert flow into the storage area for high-flow situations.

Project R, watershed storage behind the El Dorado County Irrigation District building produces a 100-year peak flow decrease of 50 cfs in Hangtown Creek. The peak flow reduction produced by Project R is essentially equivalent to the peak flow reduction produced by Project I, between Hawks Landing Court and Morrene Drive. The two projects are in the same vicinity near Mosquito Road in Randolph Canyon. For convenience, only Projects C, L, and R were evaluated with HEC-FDA, but the results for Project R can be applied equally to Project I along Hangtown Creek. Project I would be expected to have some additional benefit by reducing flooding between the project site and Hangtown Creek.

5.2.1 <u>Building Damage</u>

The flood damage analysis performed using HEC-FDA does not take into account non-building damages such as vehicle damages, roadway inundation damage, bridge overtopping, emergency response services, loss of business income, temporary relocation, transportation system disruptions, loss of public services, landscaping damages, and damages to other infrastructure such as sewer and power.

The flood damage to buildings and contents for the 10-, 25-, 50-, and 100-year events for existing conditions and Projects C, I or L, R and the combined effect of the four projects is presented in Table 16 and 17.

Table 16: Flood Damage by Recurrence Interval, Projects C and L

Hydrologic Event	Event Probability	Event Damage Existing Conditions	Event Damage With Project C	Project C Event Benefit	Event Damage With Project L	Project L Event Benefit
10-year	0.10	$ 4,202,000	$ 4,182,000	$ 20,000	$ 4,160,000	$ 42,000
25-year	0.04	$ 6,445,000	$ 6,421,000	$ 24,000	$ 6,311,000	$ 134,000
50-year	0.02	$ 7,696,000	$ 7,669,000	$ 27,000	$ 7,549,000	$ 147,000
100-year	0.01	$ 9,278,000	$ 9,239,000	$ 39,000	$ 9,014,000	$ 264,000

Table 17: Flood Damage by Recurrence Interval, Projects I or R and Combined Projects C, I, L, R

Hydrologic Event	Event Probability	Event Damage Existing Conditions	Event Damage With Project I or R	Project I or R Event Benefit	Event Damage With Projects C, I, L, R	Projects C, I, L, R Event Benefit
10-year	0.10	$ 4,202,000	$ 4,105,000	$ 97,000	$ 3,955,000	$ 247,000
25-year	0.04	$ 6,445,000	$ 6,336,000	$ 109,000	$ 6,062,000	$ 383,000
50-year	0.02	$ 7,696,000	$ 7,613,000	$ 83,000	$ 7,336,000	$ 360,000
100-year	0.01	$ 9,278,000	$ 9,168,000	$ 110,000	$ 8,934,000	$ 344,000

Over half of the total flood damage from Hangtown Creek occurs in the downtown corridor between Center Street and Bedford Avenue. For comparison, less than 10 percent of the total damage along Hangtown Creek occurs downstream from the Highway 50 crossing.

The equivalent annual damages for the existing conditions and the two project scenarios are shown in Table 18.

Table 18: Equivalent Annual Damage to Buildings

Scenario	Equivalent Annual Damage	Equivalent Annual Damage Reduced
Existing Conditions	$1,407,000	--
With Project C	$1,399,000	$8,000
With Project L	$1,390,000	$17,000
With Project I or R	$1,364,000	$43,000
With Projects C, L, I and R	$1,315,000	$92,000

The equivalent annual damage reduction from Project C is limited by the parcels that it benefits.

Project I or R has a higher equivalent annual damage reduction than Project L, despite Project L producing a higher 100-year peak flow rate reduction and resultant event damage reduction benefit. Project I or R has a higher 10-year peak flow reduction and resultant damage reduction benefit than Project L. Because the 10-year event happens more frequently, the equivalent annual damage is weighted to favor Project I or R over Project L.

It may be possible to reconfigure Project L to produce flow reduction benefit for higher probability events with lower flows on the order of the 10-year rate while sacrificing the detention volume that may be utilized for higher flows that occur with a lower frequency such as the 100-year event. The City would need to prioritize the apparently conflicting goals of reducing the 100-year peak flow rate and providing the largest equivalent annual damage reduction prior to pursuing Project L.

The net combined equivalent annual damage reduction from the four projects does not equal the sum of the individual equivalent annual damage reduction due to timing of flows and non-linear damage curves.

5.2.2 Non-structural Damage

Factors can be applied to the equivalent annual damage values to account for various non-building damages, such as clean-up and other non-structural costs that can be considered to be proportional to structural damage. Some adjustment factors were taken from DWR Flood Rapid Assessment Model Development, November 2008 (F-RAM). These adjustments include:

1. Vehicle damage
2. Roadway inundation damage
3. Bridge and culvert damage due to overtopping
4. Emergency response services
5. Loss of business income
6. Temporary relocation
7. Transportation system disruptions
8. Loss of public services
9. Damage to landscaping
10. Damage to other infrastructure such as sewer and power systems

To provide a reasonable comprehensive estimate for the flood reduction benefit of each project, the equivalent annual damage for each scenario was increased by 50 percent. An additional 5 percent benefit for Project I was included to recognize benefits to areas between the project site and Hangtown Creek.

5.2.3 Overall Project Benefits

Table 19 presents the present value of the future benefits for Project L and Project I or R, assuming an analysis period of 50 years with a 6 percent discount rate.

Table 19: Present Value of Future Benefits

Project	Building Damage Reduction	Comprehensive Damage Reduction
C	$115,000	$173,000
I	$712,000	$1,068,000
L	$268,000	$402,000
R	$678,000	$1,017,000
C, I, L, R[1]	$1,445,000	$2,168,000

[1]Note the sum of the building damage reduction from the four individual projects is greater than the building damage reduction from the combination of the four individual projects due to the non-linear relationships of flood damage reduction, flood stage versus discharge, and discharge reductions not being additive.

5.3 Potential Project Costs

Concept-level planning cost estimates were developed for Projects C, I, L and R. These cost estimates were developed based on the conceptual project configurations presented in Section 4.1 of this report.

5.3.1 Project C Cost Estimate

Project C includes building an embankment about 10 to 12 feet high to detain flows from the tributary watershed. The outlet structure would be configured to passively control the release rate using an orifice and weir overflow configuration. The concept-level planning cost estimate for Project C is presented in Table 20.

Table 20: Project C Cost Estimate

Project C:	**Placerville Drive behind Polaris**			
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Excavation (Includes Placement as Fill)	5,000	CY	$ 25.00	$ 125,000.00
Rip-rap Slope Protection	2,000	CY	$ 100.00	$ 200,000.00
Outlet	1	LS	$ 20,000.00	$ 20,000.00
Subtotal Phase 1 Base Construction				$ 345,000.00
Mobilization (10%)				$ 34,500.00
Geotechnical (15%)				$ 51,750.00
Construction Management (10%)				$ 34,500.00
Contingency (15%)				$ 51,750.00
Administration (10%)				$ 34,500.00
Engineering				
Hydraulic Design Calculations				$ 20,000.00
Structural Design				$ 30,000.00
Civil Plans and Specifications				$ 50,000.00
Land Acquisition[1]	2	AC	$ 40,000.00	$ 80,000.00
Mitigation and Permitting				$ 130,000.00
Total				**$ 862,000**

[1]Land Acquisition costs based on land values of impacted parcels from 2010 El Dorado County Assessor.

5.3.2 Project I Cost Estimate

Project I includes construction of earthen embankments around the low area between Morrene Drive and Hawks Court Landing to create additional watershed storage. A passive orifice and weir outlet configuration would be constructed to provide flow control. Some excavation would also be required. Table 21 presents the concept-level planning cost estimate for Project I.

Table 21: Project I Cost Estimate

Project I : Between Morrene Drive and Hawks Landing Court				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Excavation	3,000	CY	$ 10.00	$ 30,000.00
Grading for Berms	6,000	CY	$ 20.00	$ 120,000.00
Erosion Control for Berms	3,000	SY	$ 5.00	$ 15,000.00
Outlet	1	LS	$ 15,000.00	$ 15,000.00
Subtotal Base Construction				*$ 210,000.00*
Mobilization (10%)				$ 23,000.00
Restoration Component (20%)				$ 46,000.00
Geotechnical (15%)				$ 34,500.00
Construction Management (10%)				$ 23,000.00
Contingency (25%)				$ 57,500.00
Administration (10%)				$ 23,000.00
Engineering				
Hydraulic Design Calculations				$ 30,000.00
Structural Design				$ 4,000.00
Civil Plans and Specifications				$ 90,000.00
Land Acquisition[1]	1.5	AC	$ 50,000.00	$ 75,000.00
Mitigation and Permitting				$ 150,000.00
Total				***$ 766,000***

[1]Land Acquisition costs based on land values of impacted parcels from 2010 El Dorado County Assessor.

5.3.3 *Project L Cost Estimate*

Project L would require excavation of approximately 8,000 cubic yards, most of which would need to be hauled offsite. It is estimated that about 25 percent of the excavated material could be used in construction to create berms around the proposed basin. The project includes a downstream outlet structure that would drain flows back into Hangtown Creek after the floodwave passes. Table 22 presents the concept-level planning cost estimate for Project L.

Table 22: Project L Cost Estimate

Project L :	**Off-channel Detention Basin Upstream from Wiltse Road**			
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Excavation	8,000	CY	$ 10.00	$ 80,000.00
Hauling	6,000	CY	$ 25.00	$ 150,000.00
Grading for Berms	2,000	CY	$ 20.00	$ 40,000.00
Erosion Control for Berms	5,000	SY	$ 5.00	$ 25,000.00
Outlet Structure	1	LS	$ 20,000.00	$ 20,000.00
Subtotal Base Construction				*$ 315,000.00*
Mobilization (10%)				$ 31,500.00
Geotechnical (15%)				$ 47,250.00
Construction Management (10%)				$ 31,500.00
Contingency (25%)				$ 78,750.00
Administration (10%)				$ 31,500.00
Engineering				
Hydraulic Design Calculations				$ 30,000.00
Structural Design				$ 4,000.00
Civil Plans and Specifications				$ 90,000.00
Land Acquisition[1]	1.5	AC	$ 30,000.00	$ 45,000.00
Mitigation and Permitting				$ 150,000.00
Total				***$ 855,000***

[1]Land Acquisition costs based on land values of impacted parcels from 2010 El Dorado County Assessor.

5.3.4 *Project R Cost Estimate*

Project R includes building an embankment about 15 feet high to detain flows from the tributary watershed. The outlet structure would be configured to passively control the release rate using an orifice and weir overflow configuration. As this project is located in a future developable area, the City may seek to construct the project in conjunction with a future subdivision project in the area. The concept-level planning cost estimate for Project R is presented in Table 23.

Table 23: Project R Cost Estimate

Project R:	**Behind El Dorado Irrigation District Building**			
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Excavation (Includes Placement as Fill)	5,000	CY	$ 25.00	$ 125,000.00
Rip-rap Slope Protection	2,000	CY	$ 100.00	$ 200,000.00
Outlet	1	LS	$ 20,000.00	$ 20,000.00
Subtotal Phase 1 Base Construction				*$ 345,000.00*
Mobilization (10%)				$ 34,500.00
Geotechnical (15%)				$ 51,750.00
Construction Management (10%)				$ 34,500.00
Contingency (15%)				$ 51,750.00
Administration (10%)				$ 34,500.00
Engineering				
Hydraulic Design Calculations				$ 20,000.00
Structural Design				$ 30,000.00
Civil Plans and Specifications				$ 50,000.00
Land Acquisition[1]	2	AC	$ 25,000.00	$ 50,000.00
Mitigation and Permitting				$ 130,000.00
Total				***$ 832,000***

[1]Land Acquisition costs based on land values of impacted parcels from 2010 El Dorado County Assessor.

5.4 Maintenance Costs

Maintenance costs for each of the proposed projects are estimated to be approximately $5,000 to $6,000 per year to cover periodic landscaping, debris cleanup and life-cycle costs to replace components such as outlet pipes. The net present value of the maintenance costs are included in the comprehensive project costs by adding $100,000 to the project capital costs.

6.0 Conclusions and Recommendations

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

New topographical and survey data were used to create a new hydrology model for the Hangtown Creek Watershed and a new hydraulic model for Hangtown Creek through the City of Placerville. Peak flow rates for the 10-, 25, 50-, and 100-year recurrence interval storm events are higher than the 1983 FEMA Flood Insurance Study, but lower than those indicated in other recent studies performed by others in the watershed. Though peak flows have increased due to development, the majority of the increase in peak discharge rates can be attributed to the more detailed methodology used in the Plan.

The hydraulic modeling of Hangtown Creek indicates more significant flood risk to structures in the stream corridor than shown on FEMA floodplain maps, primarily due to the higher discharge rates. Preliminary mapping was prepared as part of this study.

Eighteen potential projects were evaluated to determine the feasibility for reducing peak flow rates and flood damages. Only four projects, identified as Projects C, I, L and R, were determined to be potentially feasible. Project costs and expected benefits for these three projects were evaluated as presented in detail in Section 5.0. The project costs and expected present value of future benefits are summarized in Table 24.

Table 24: Project Cost and Project Benefit Summary

Project	Estimated Capital Costs	Comprehensive Project Costs	Present Value of Future Benefits
Project C	$862,000	$962,000	$173,000
Project I	$766,000	$866,000	$1,068,000
Project L	$855,000	$955,000	$402,000
Project R	$832,000	$932,000	$1,017,000
Project C, I, L R	$3,315,000	$3,715,00	$2,167,500[1]

[1] Combined comprehensive damage reduction is less than the sum of the individual projects.

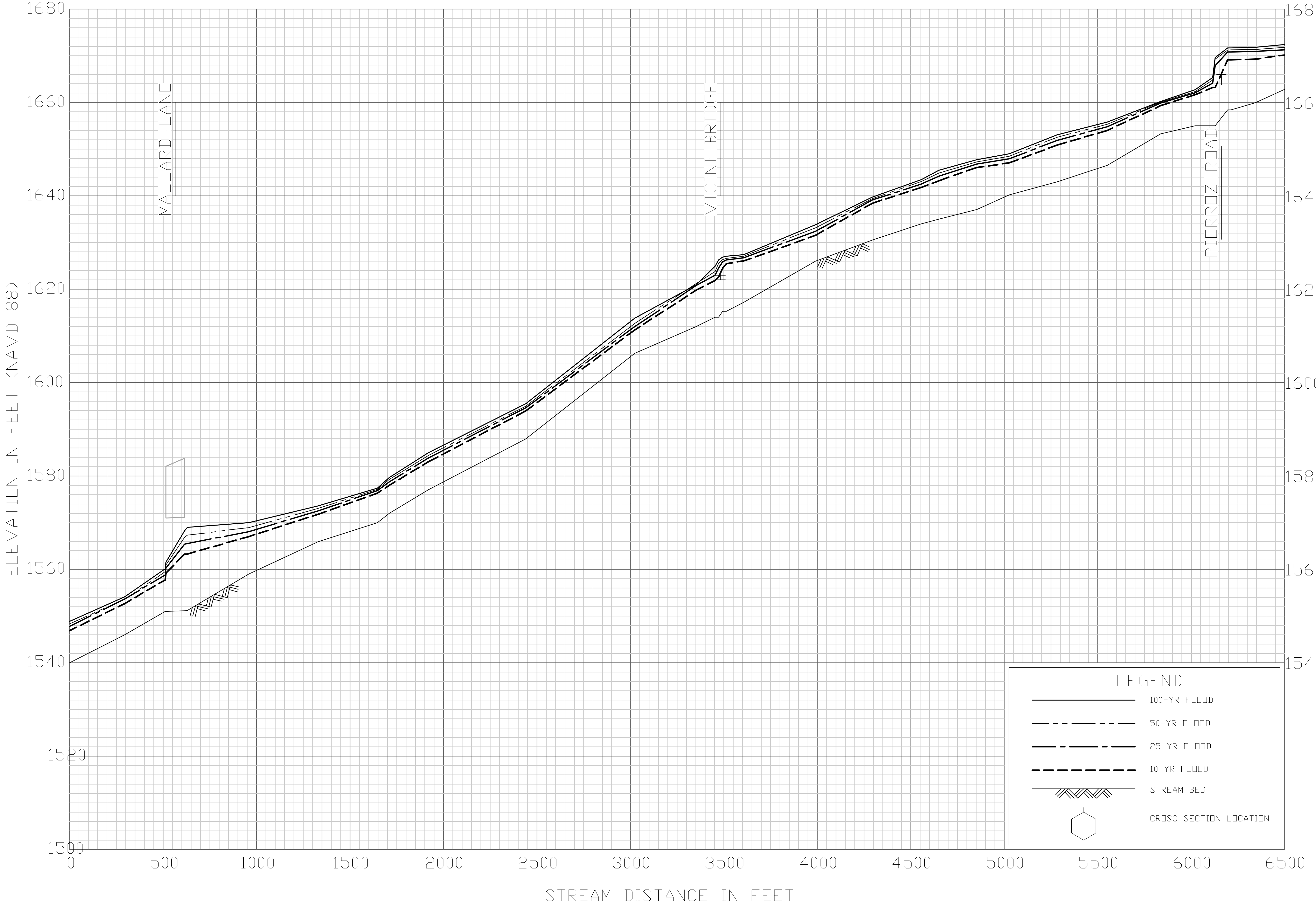
The analyses performed for the Plan indicate that Project I and R have benefit to cost ratios greater than one and are, therefore, justified economically. Project C and L each have the potential to reduce 100-year peak discharges by as much as Projects I and R combined; but, because Project L would not provide benefit during more frequent damaging flood events and Project C has a smaller area downstream from it, neither shows as great of future economic benefits. However, both Projects C and L may be worthwhile to pursue if a nexus is identified for development impacts. Additionally, potential benefit from flood risk reduction at the wastewater treatment plant was not included in the analysis but may provide project justification.

6.2 Recommendations

To reduce the flood risks along Hangtown Creek, the following recommendations are provided:

1. Enter into a Cooperating Technical Partnership (CTP) agreement with FEMA to facilitate completion of new flood mapping to make property owners aware of the flood risk from Hangtown Creek and to ensure that new and substantial remodeled structures have appropriate levels of flood protection.
2. Implement Low Impact Development policies to reduce the impact of future development.
3. Pursue funding for Projects I and R based on estimated economic benefit exceeding project costs.
4. Pursue Projects C and L when factors such as development impacts and/or benefit to the wastewater treatment plant indicate potential feasibility.
5. Support building elevation and buy-out programs, especially within the downtown corridor.

Flood Profiles



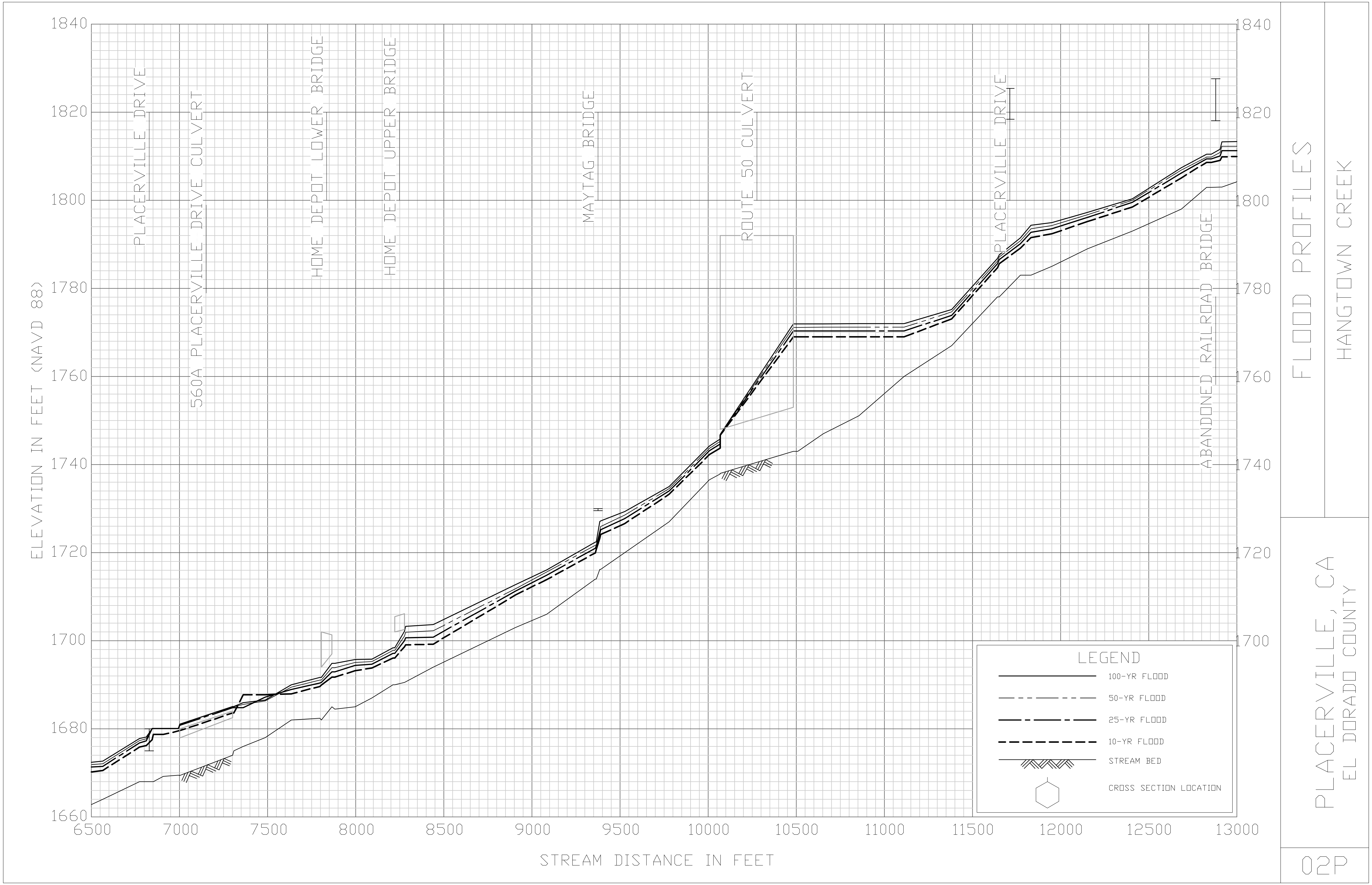
FLOOD PROFILES

HANGTOWN CREEK

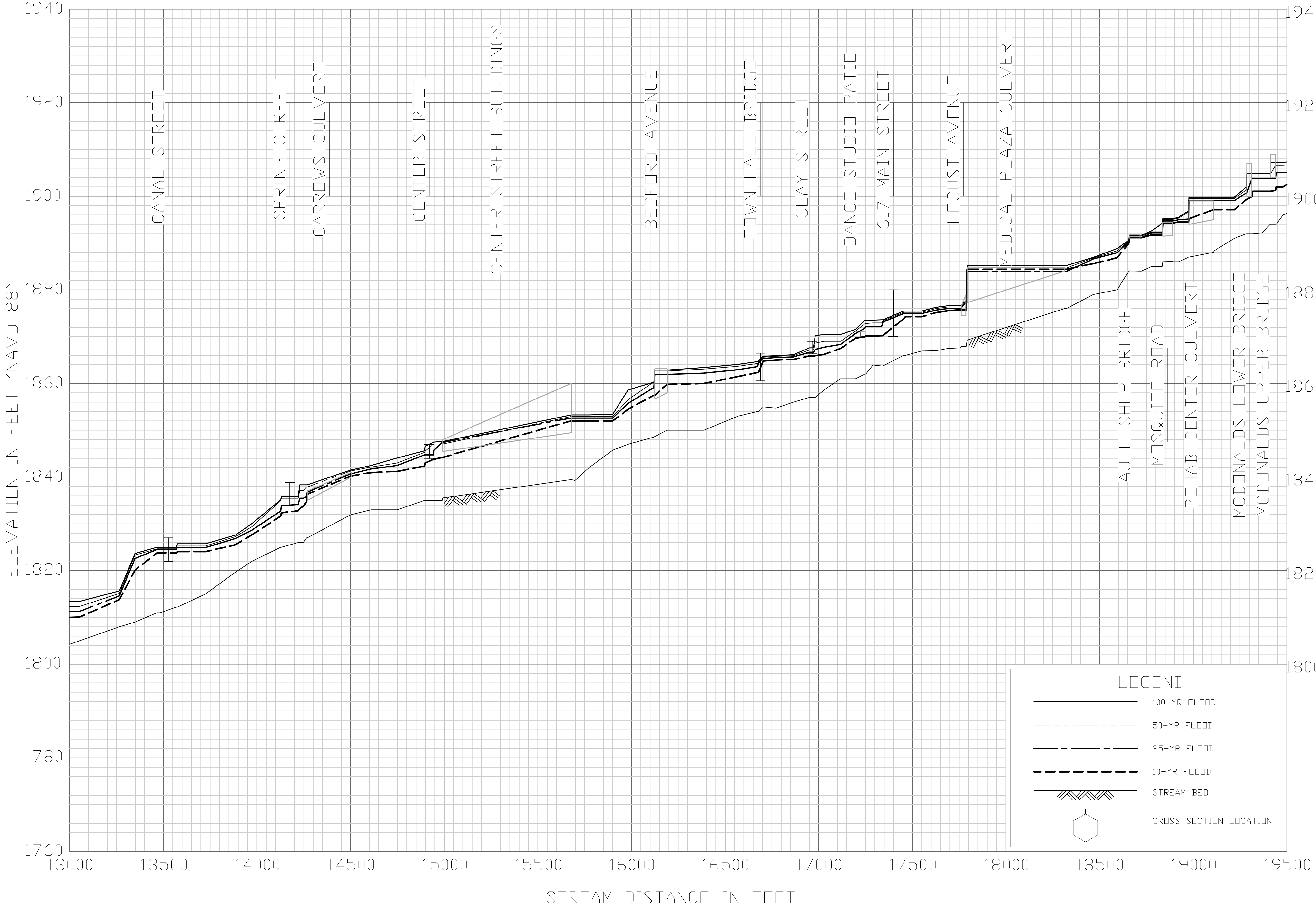
PLACERVILLE, CA

EL DORADO COUNTY

01P



FLOOD PROFILES
HANGTOWN CREEK
PLACERVILLE, CA
EL DORADO COUNTY
02P
ELEVATION IN FEET (NAVD 88)
STREAM DISTANCE IN FEET
1840
1820
1800
1780
1760
1740
1720
1700
1680
1660
6500
7000
7500
8000
8500
9000
9500
10000
10500
11000
11500
12000
12500
13000
PLACERVILLE DRIVE
560A PLACERVILLE DRIVE CULVERT
HOME DEPOT LOWER BRIDGE
HOME DEPOT UPPER BRIDGE
MAYTAG BRIDGE
ROUTE 50 CULVERT
PLACERVILLE DRIVE
ABANDONED RAILROAD BRIDGE
LEGEND
100-YR FLOOD
50-YR FLOOD
25-YR FLOOD
10-YR FLOOD
STREAM BED
CROSS SECTION LOCATION



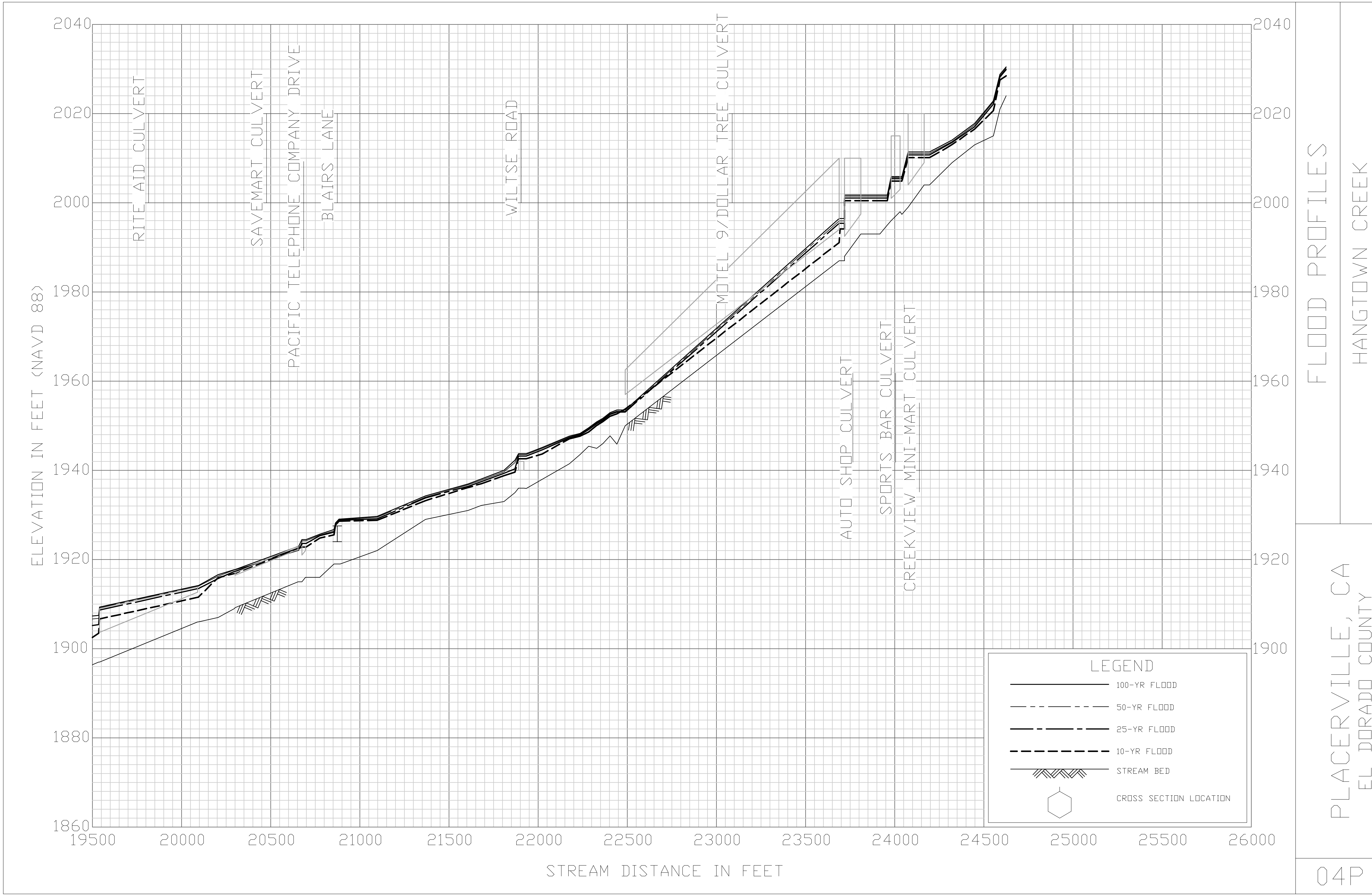
FLOOD PROFILES

HANGTOWN CREEK

PLACERVILLE, CA

EL DORADO COUNTY

03P



FLOOD PROFILES
HANGTOWN CREEK
PLACERVILLE, CA
EL DORADO COUNTY
04P
ELEVATION IN FEET (NAVD 88)
STREAM DISTANCE IN FEET
RITE AID CULVERT
SAVEMART CULVERT
PACIFIC TELEPHONE COMPANY DRIVE
BLAIRS LANE
WILTSE ROAD
MOTEL 9/DOLLAR TREE CULVERT
AUTO SHOP CULVERT
SPORTS BAR CULVERT
CREEKVIEW MINI-MART CULVERT
LEGEND
100-YR FLOOD
50-YR FLOOD
25-YR FLOOD
10-YR FLOOD
STREAM BED
CROSS SECTION LOCATION
1860
1880
1900
1920
1940
1960
1980
2000
2020
2040
19500
20000
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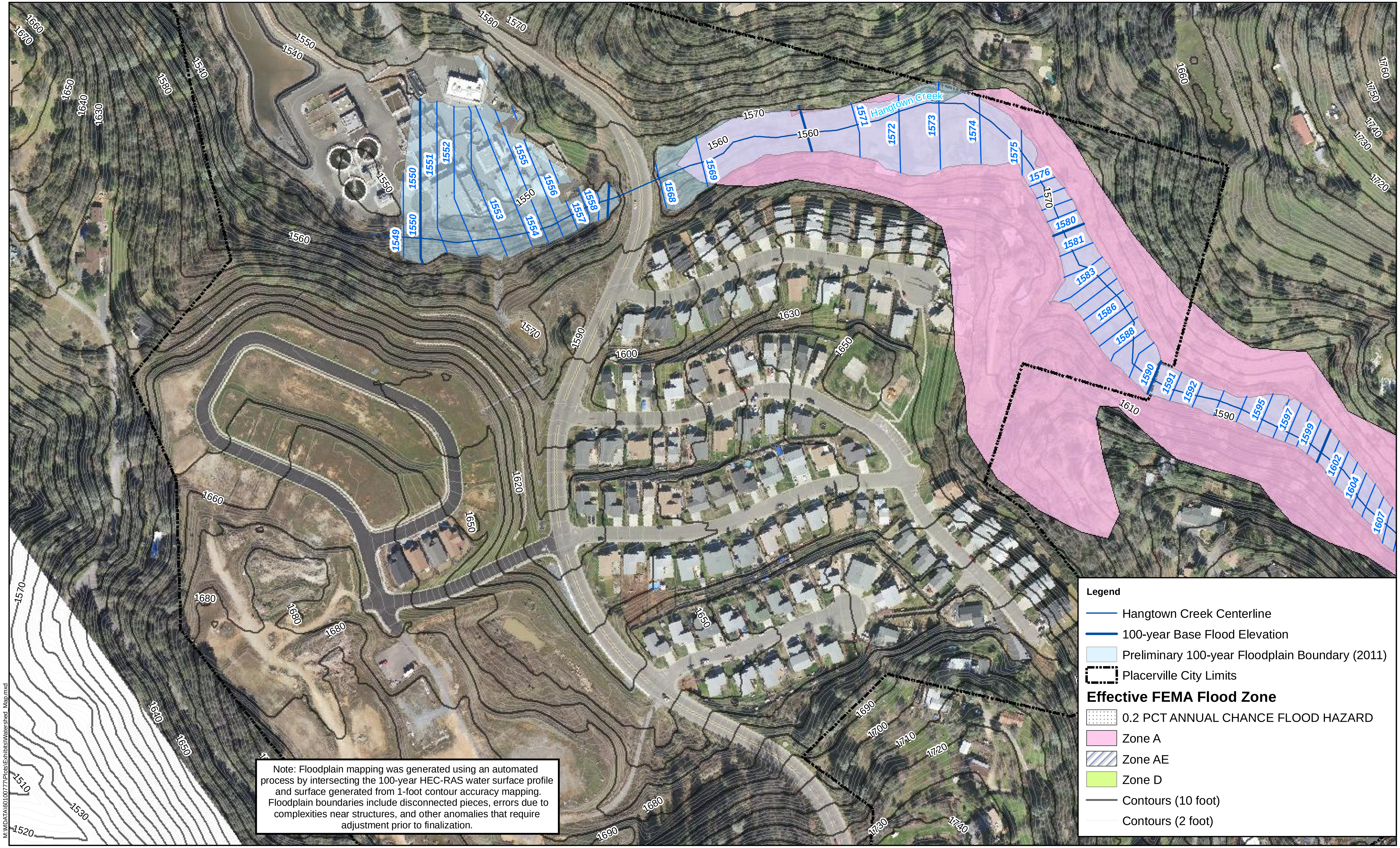
25500
26000

Preliminary Floodplain Maps



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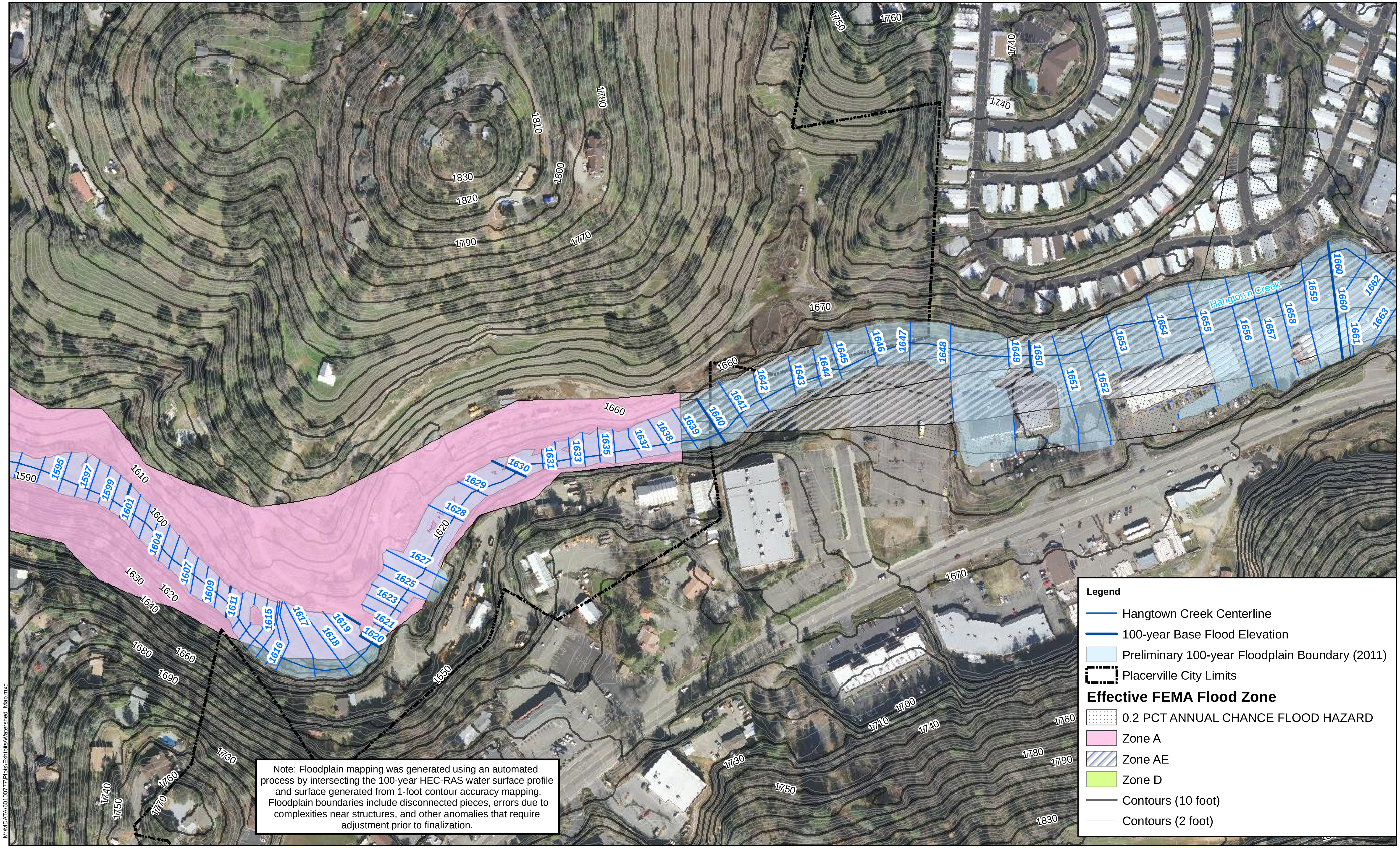
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping

Hangtown Creek 1



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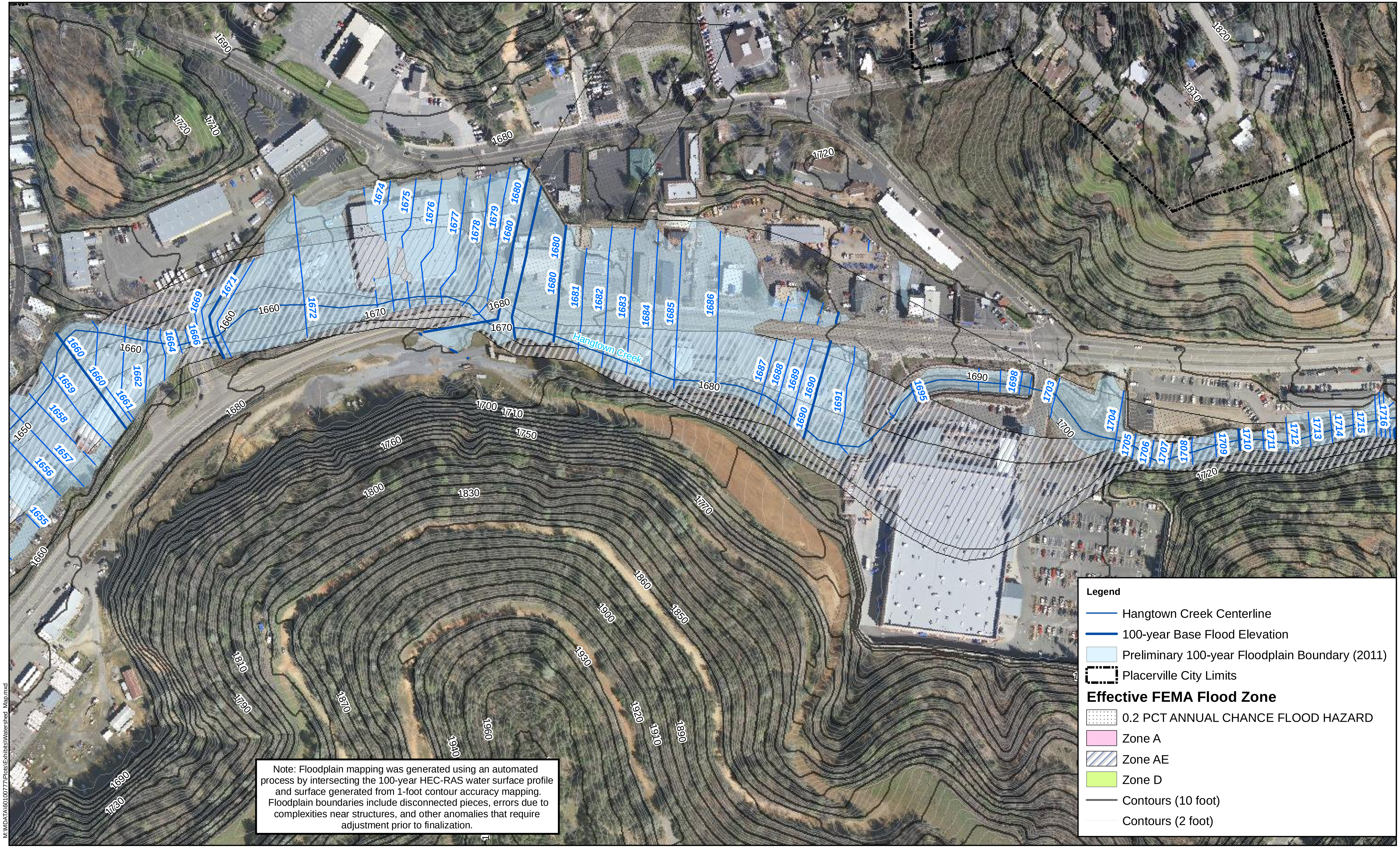
PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping

Hangtown Creek 2



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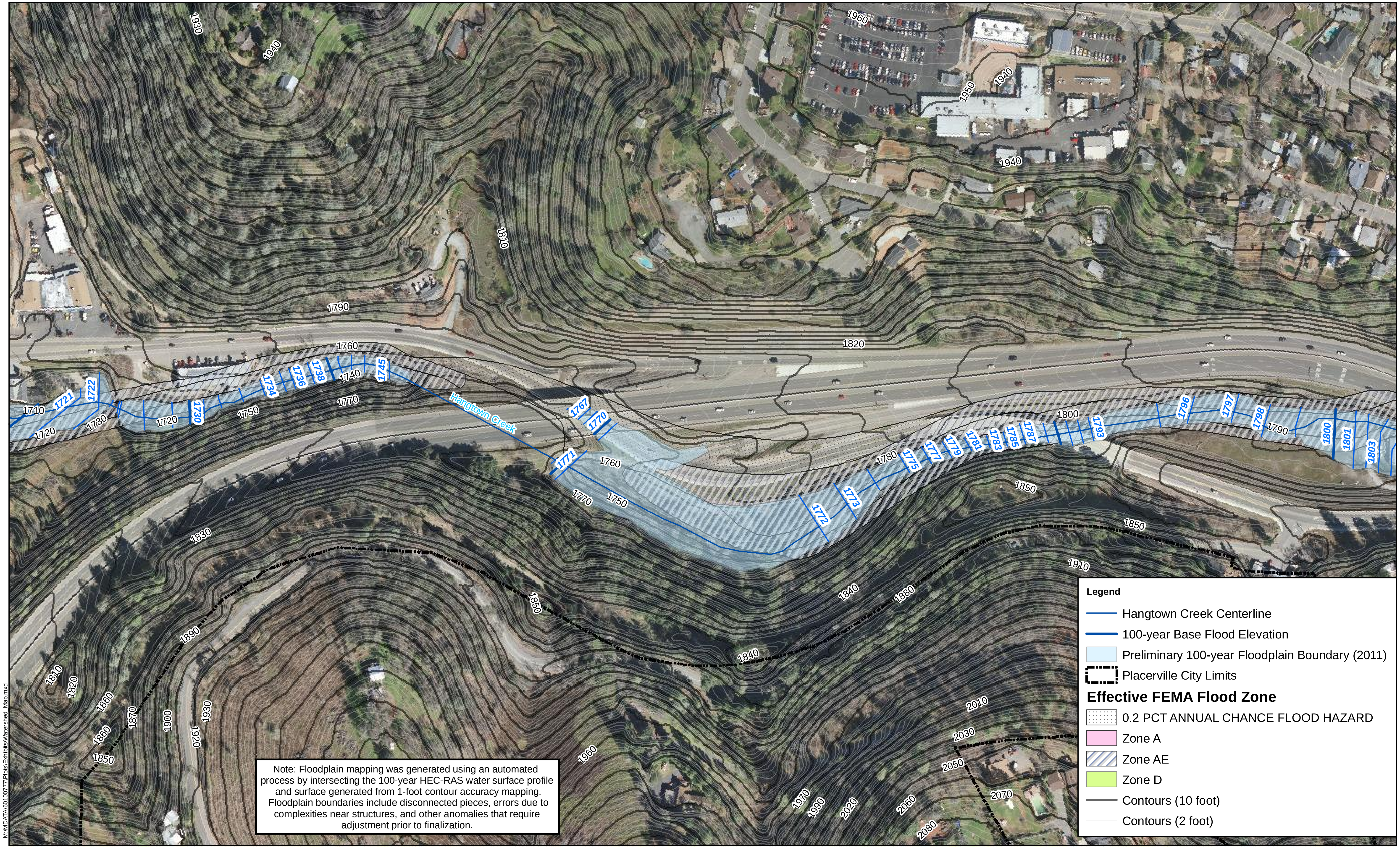
PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping

Hangtown Creek 3



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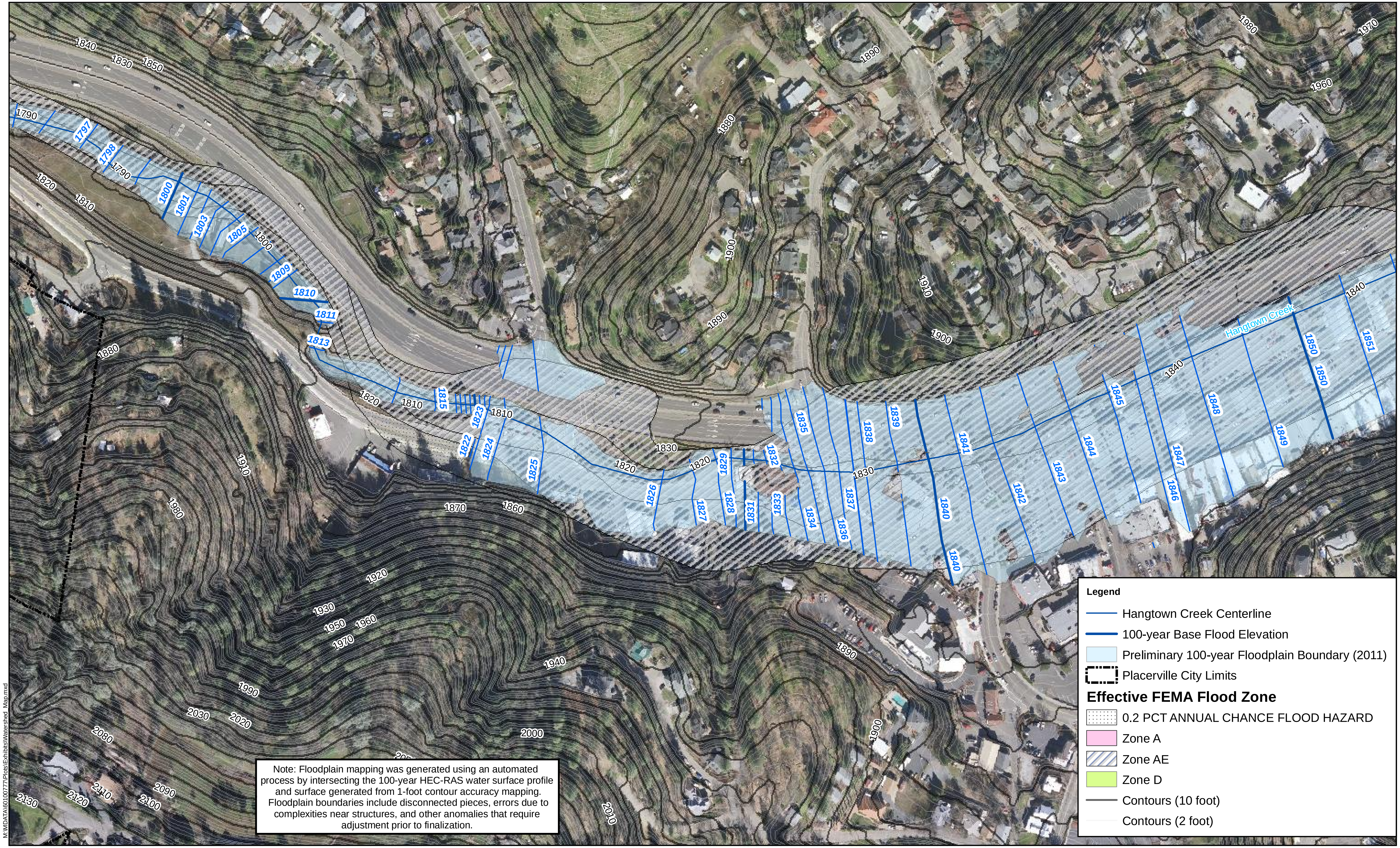
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping

Hangtown Creek 4



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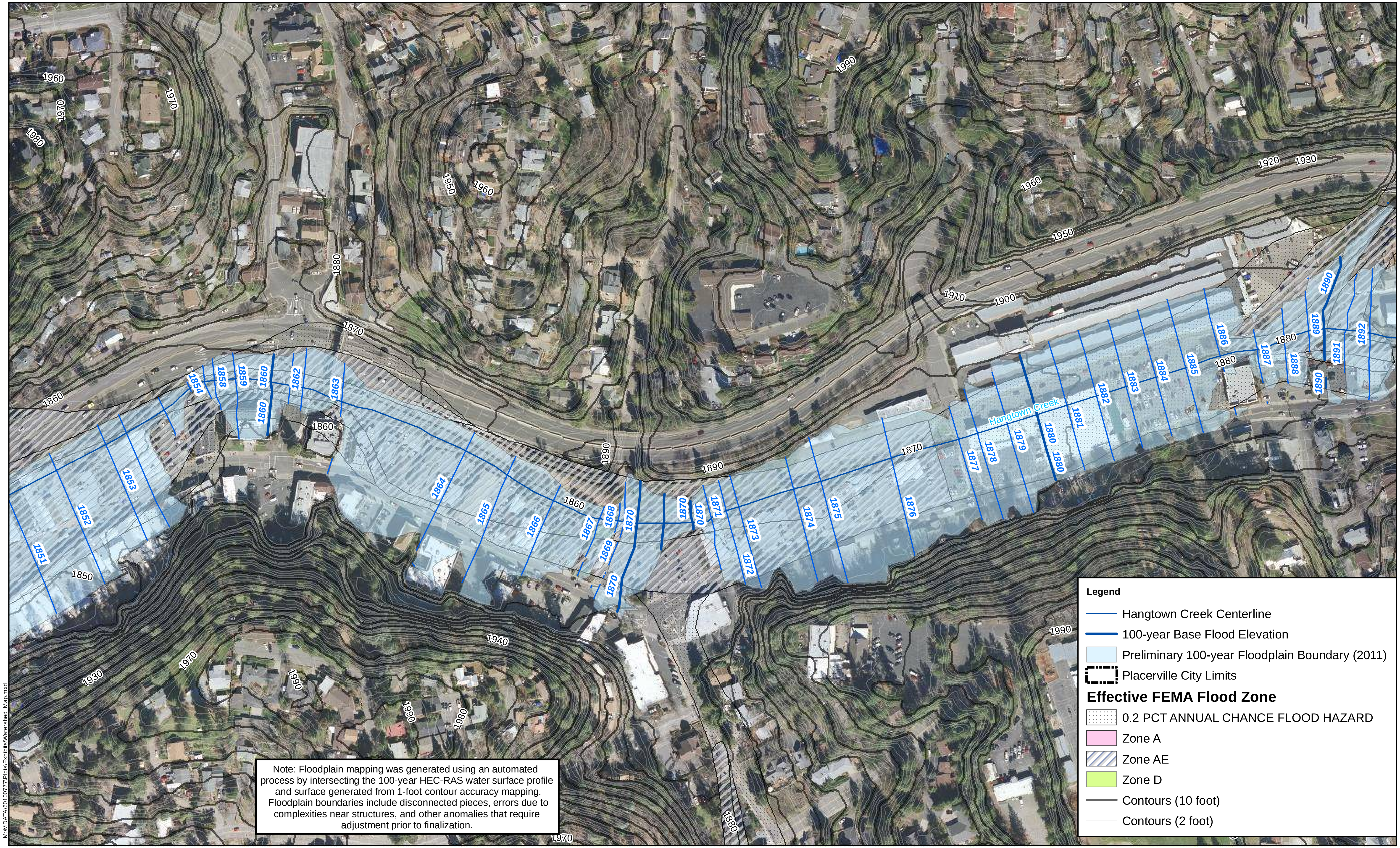
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping



Feet
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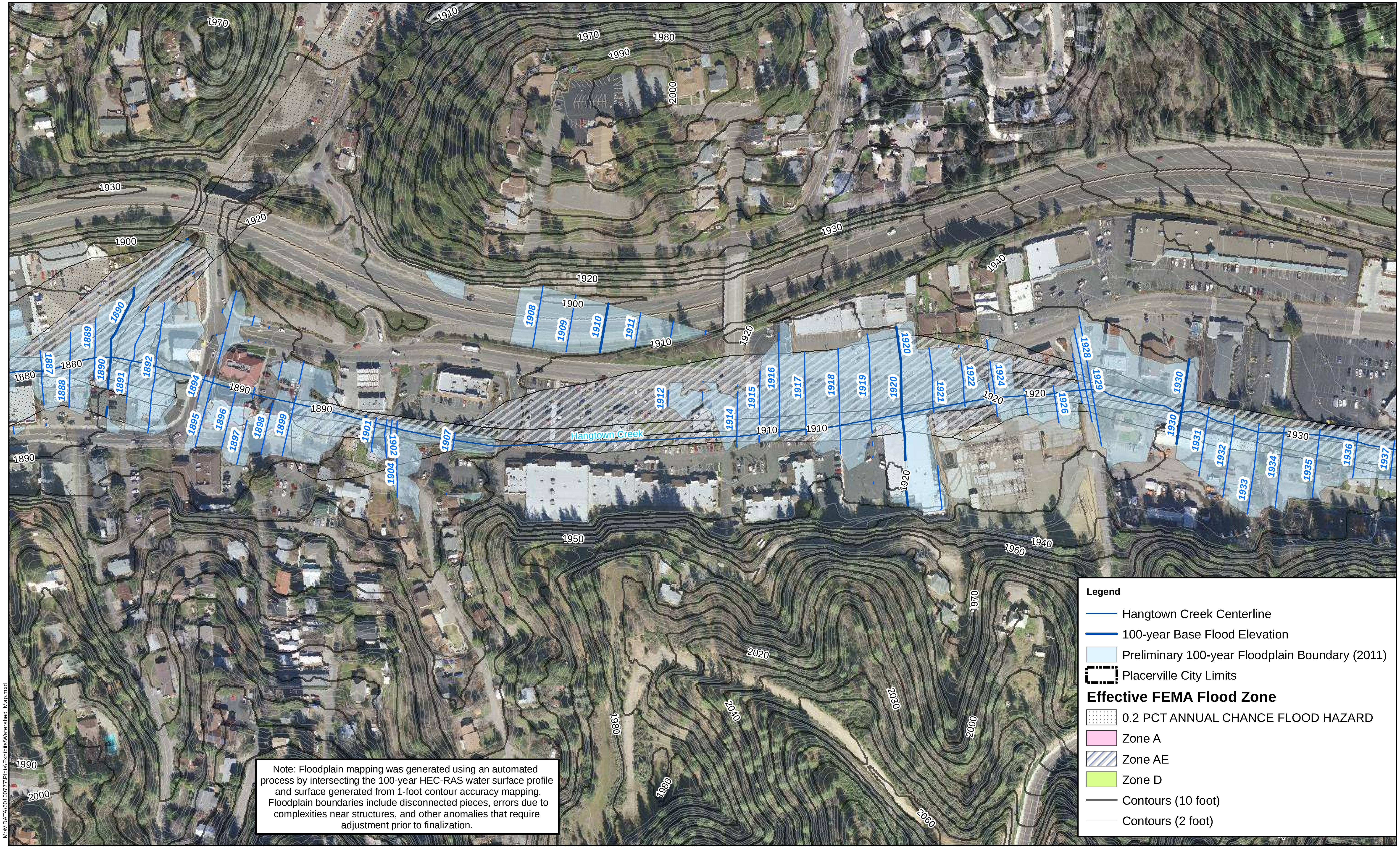
Source:
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping



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Source:
Date: 06/23/11

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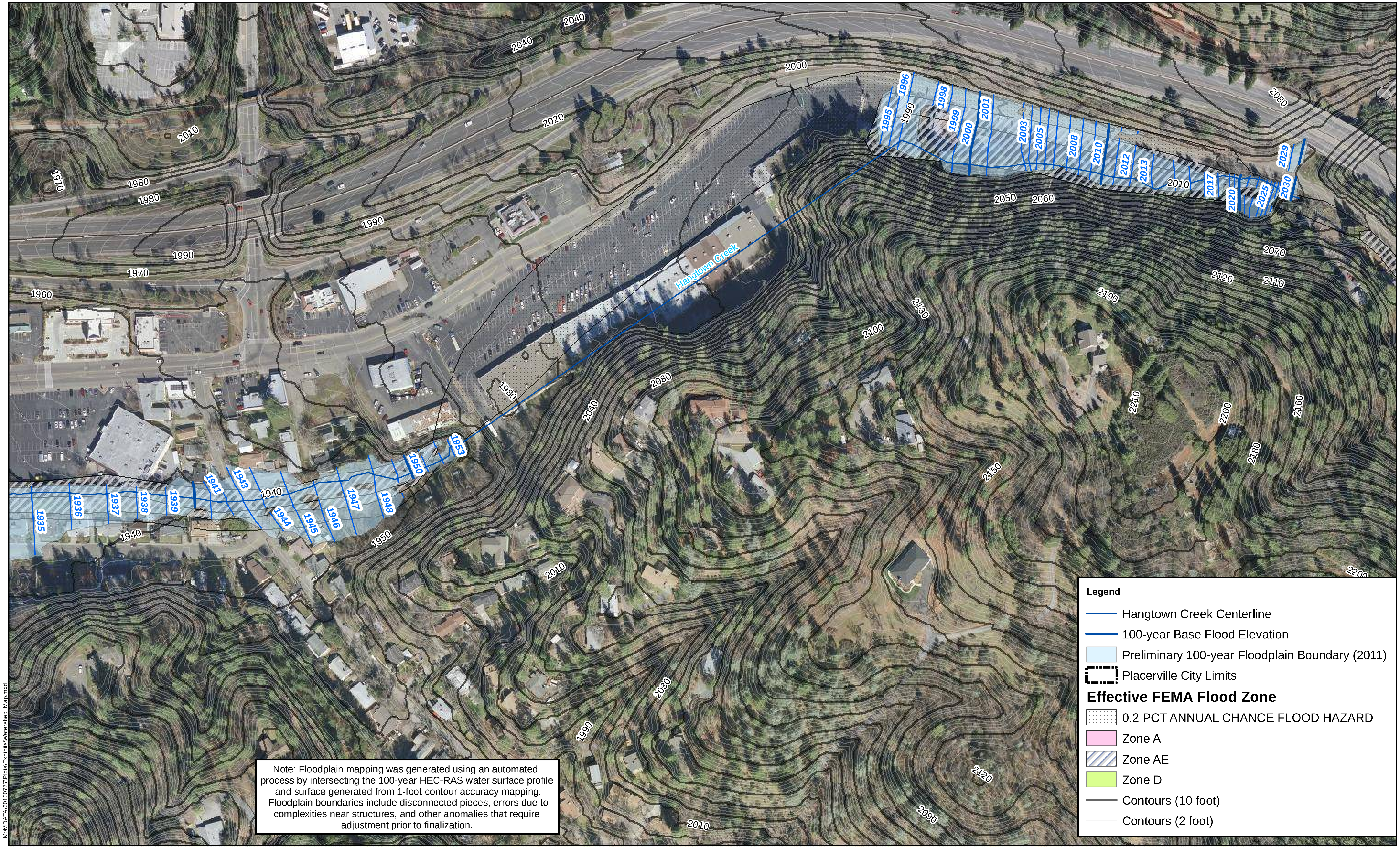
PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping

Hangtown Creek 7



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Feet
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1"=200'

Source:
Date: 06/23/11

PLACERVILLE HANGTOWN CREEK COMPREHENSIVE WATERSHED PLAN

Preliminary 100-year Floodplain Mapping

Hangtown Creek 8

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