A SALVADOR CREEK FLOOD MODEL

Prepared for
The Napa County Flood Control and Water Conservation District
and
The City of Napa

by

Paul Blank and Bob Zlomke
Napa County Resource Conservation District

J. Carter Borden and Jeremy Kobor
DHI Water & Environment, Inc.

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A NOTE ON THIS DOCUMENT

The present document is an enhanced version of *Final Report: Salvador Creek Flood Modeling/ Conceptual Plan, June 2006*, produced by Napa County Resource Conservation District (RCD) under Napa City contract no. 8904. It includes the complete text of the June 2006 report, with no changes, as *Part One* and adds an additional *Part Two*, which presents additional work done to December 2007 under new Napa City contract no. 9234 and contract no. 303 (FC) with Napa County Flood Control and Water Conservation District. The original pagination and numbering of figures and tables in *Part One* are preserved, *Part Two* having its own. The conclusions and recommendations in *Part Two* represent RCD's and DHI's best judgment to date on the matters studied and should supersede those in *Part One*.

Beginning with a simple channel-only model developed under earlier contracts, *Part One* describes a set of fundamental improvements to the model setup and a recalibration to the major storm of December 31, 2005. The work previous to *Part One* had the shortcoming that the floodplains of the creek were not represented at all. Remedying this lack of floodplain representation was the most important improvement to the model setup. In addition, the model's physical representation of the channel itself was improved by verifying selected channel cross sections and putting in all bridges that appeared likely to affect the 100-year discharge. At the same time, RCD added to the model the branches of the creek west of the highway.

Beyond making these improvements, *Part One* revisits the model calibration. There was a major storm on Salvador Creek (as elsewhere in the Napa River watershed) on December 31, 2005; RCD took the opportunity to document this storm carefully and made primary use of it in the recalibration. The calibrated model was used to study the 100-year storm on Salvador Creek and related scenarios, topics which were revisited in *Part Two*.

Part Two describes improvements to the distribution of storm runoff in the channel model, making the distribution agree with the City of Napa's *Storm Drain Master Plan*. It also describes revisions to the initial conditions of the 100-year storm as previously modeled, on the basis of the observed and modeled flood of December 31, 2005.

In *Part Two* we also report on a jamb elevation survey, which was carried out to determine the specific risk of flooding to houses. On the basis of information derived from that survey, the decision was made to develop a MIKE Flood model of Salvador Creek, which permitted much more realistic modeling of floodplain flows. The MIKE Flood model combines a one-dimensional model of the channel with a two-dimensional model of the floodplain, so that the actual paths of overland flow are modeled hydraulically. The calibration of the model was again revisited, on the basis of the storm of December 31, 2005, and the calibrated model used to study scenarios of interest.

Part Two was originally written in December 2007, and the present revised version was written in April 2008.

***PART ONE*:**

MODEL UPDATES AND CALIBRATION TO THE FLOOD OF DECEMBER 31, 2005

June 2006

PART ONE:

MODEL UPDATES AND CALIBRATION TO THE FLOOD OF DECEMBER 31, 2005

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PART ONE:

MODEL UPDATES AND CALIBRATION TO THE FLOOD OF DECEMBER 31, 2005

Napa County Resource Conservation District
June 2006

I. Introduction

The Salvador Creek Watershed, which lies in and near the city of Napa, California, has seen considerable change since the mid-twentieth century. A large part of its watershed and channel length currently lie within the City of Napa (Figure 1), and as the City has grown, the watershed and creek channel have undergone a number of changes. These changes included considerable residential development and associated re-engineering of the creek channel. Building new homes and constructing new roads can lead to an increased flooding hazard, by replacing permeable surfaces with impervious surfaces and by obstructing natural floodplains; and increases in water levels noted in recent years appear to be associated with the changes that have occurred.

At the same time, there has been an increasing recognition of the environmental value of the creek. The desire to care for the creek while addressing flood concerns, which was shared by residents and City officials alike, led the City in 2001 to contract with the Napa County Resource Conservation District (RCD) to provide stewardship support and hydrologic modeling services to the nascent Salvador Creek Stewardship Group, with the goal of developing both a flood conveyance study and an organized Stewardship group of concerned citizens to focus and guide the restoration process.

Beginning in 1999, the RCD had worked with the City of Napa to do hydrologic modeling and monitoring of local creeks. The initial contract was for monitoring and modeling of Napa Creek, and in subsequent years monitoring of flows on Milliken and Salvador Creeks was added. In 2001 we began to build an unsteady-flow hydraulic model of the creek on the basis of the MIKE 11 software developed by DHI Water & Environment. In a report prepared in June 2005, RCD reported on the development of the model and used it to study scenarios identified by the Stewardship Group (Zlomke et al., 2005). These included flood conveyance in the Garfield Lane bridge area, flood terraces above Jefferson and detention in Alston Park (Figure 1).

The present report describes modeling improvements made by RCD since June 2005, under contract no. 8904 with the City of Napa. The next section provides more modeling background and enumerates the specific modeling tasks carried out under the current contract.

II. Modeling Background

A. The MIKE 11 Model

The principal modeling tool used by the RCD for modeling Salvador Creek is the MIKE 11 model, an implicit finite difference model developed by DHI Water and Environment (formerly the Danish Hydraulic Institute). MIKE 11 is capable of modeling a network of one-dimensional channels, both for hydrodynamics and for a variety of water quality parameters. It is based on the St. Venant equations for one-dimensional unsteady flow, and as an unsteady flow model it is able to track the progress over time and space of floods, tidal events, and the interactions between the two, within the limits of a network of one-dimensional channels. These capabilities are particularly useful in the Napa River system, which is tidally influenced and experiences frequent flooding as well. In a similar fashion, specialized MIKE 11 models can track the progress of a "wave" of concentration of sediment or other water quality constituent. These capabilities permit the study of a large variety of scenarios in which time plays a role. In 2000, the MIKE 11 model was approved by FEMA for use in mapping floodplains within the National Flood Insurance Program.

The Salvador Creek work described in this report is focused on flooding and the effects of various interventions on it. For such work, the data needed by MIKE 11 fall into two broad categories:

- Topographical information, on stream channels, floodplains, and hydraulic structures
- Hydrometric data (principally rainfall and streamflow measurements), to drive the basic hydrodynamic model and the rainfall/runoff model

Section III of this report will provide details about the data used in the present work.

Two MIKE 11 models were used. The basic hydrodynamic (HD) model was employed, using the fully dynamic formulation of the governing equations; this model calculates water level and discharge throughout the model domain over the entire period simulated. One specialized model was employed as well, the *rainfall/runoff model* (RR), also called the *NAM* model.

The MIKE 11 RR model may be described as a continuous lumped-parameter model. A *continuous* model simulates the runoff from rainfall over an extended period, including the interactions between groundwater and other storage as the seasons change, in contrast to an *event* model like HEC-HMS, which simulates the runoff from a single storm and depends on assumptions about antecedent conditions. In effect, the entire land phase of the hydrologic cycle is modeled in MIKE 11. Storage volumes in groundwater, on the surface, and in the soil are continuously calculated and their contributions to baseflow, interflow, and overland flow determined, on the basis of parameters set by the user. These parameters are *lumped* in the sense that an average value is used in each subcatchment, and the values used are not directly derivable from observations but must be determined by calibration. Therefore, it is essential to have measurements of discharge to calibrate to.

B. Previous Modeling Work

Modeling work done prior to June 2005 was based on the initial model of Salvador Creek described in *Hydrologic Modeling and Stewardship of the Salvador Creek Watershed* (Jones & Sharp, 2003). The model was developed with two branches, Salvador Creek from Highway 29 to the confluence with the Napa River and the Napa River from Oak Knoll Avenue to the 3rd Street Bridge. The Salvador Creek channel was represented by cross sections obtained in a field survey by RCD staff, which generally did not include the floodplains.

The model was developed and calibrated using rain and discharge data for the 2003-04 rainy season. The rainfall data used to drive the model were derived mainly from two City ALERT system gages, no. 2271 (City Corporation Yard) and no. 2253 (Redwood Road at Mt. Veeder Road), and the discharge data used for calibration of the model were derived from a streamflow record at Big Ranch Road (chainage 2943), which the RCD established before the 2003-04 water year. This calibration was limited in value by the fact that it was based on a single year's hydrometric data. The model included only one bridge, that at Garfield Lane. Other bridges were surveyed but not included in the model.

The initial scenarios examined with the model involved flood improvements in the Garfield Park area. The 100-year storm showed Salvador Creek up around the top of bank at a number of locations between Highway 29 and the Napa River, especially in the area immediately downstream of Garfield Lane Bridge. In summary, the results showed that removal of the Garfield Lane Bridge (or its replacement by a footbridge) and the addition of a flood terrace in the park would offer a net reduction in flood peaks, but that most of the benefit would be in the park, where the need for flood peak reduction is slight.

Therefore, a number of residents asked whether such flood terraces might be considered elsewhere on the creek, particularly upstream of Jefferson Street. Accordingly, hypothetical flood terraces above Jefferson were modeled in order to get a rough idea of the possible reduction in flood levels that would result from the creation of overflow flood terraces along the creek there. This modeling exercise, carried out using a simplified method in MIKE 11, featured wider (75m or 250 ft) flood terraces in the remaining agricultural parcels above Jefferson Street. The results showed a reduction in water levels of about 0.15 m (0.5 ft) throughout the reach altered, with some benefit extending a considerable distance downstream.

By changing assumptions in the MIKE 11 rainfall/runoff model, a rough model of detention storage at Alston Park was created. Model results suggested strongly that detention in Alston Park would have no measurable effects downstream of the highway.

C. Objectives of the Present Work

The previous work, just described, had the shortcoming that the floodplains of the creek were not represented at all. Remedying this lack of floodplain representation was the most important objective of the present work. In addition, we hoped to improve our confidence in the model's physical representation of the channel itself, by verifying selected channel cross sections and putting in all bridges that appeared likely to affect the 100-year discharge. At the same time, we added to the model the branches of the creek west of the highway (Figure 2).

Beyond making these improvements, the present work set out to revisit model calibration using additional data which had become available since the earlier work was done. As it happened, there was a major storm on Salvador Creek (as elsewhere in the Napa River watershed) on December 31, 2005, which we took the opportunity to document carefully and included in our calibration.

The rest of this report is organized around the following tasks:

- Field-verify channel cross sections surveyed since 2001 by RCD and add information on bridge geometry to the model
- Update calibration of the MIKE 11 RR model
- Use the MIKE 11 RR model, along with a companion HEC-HMS model, to estimate the 100-year discharge on Salvador Creek
- Add bridges and floodplain cross sections to the MIKE 11 HD model
- Recalibrate the MIKE 11 HD model to the flood of New Years Eve 2005
- Model scenarios using the expanded floodplain HD model
- Prepare a conceptual plan for Salvador Creek, on the basis of modeled scenarios

III. Verifying and Extending Model Data

A. Channel Path Adjustment

The path of Salvador Creek used in the previous modeling effort was plotted over both a digital elevation model (DEM) of Napa County and 2002 aerial photos, and it was noted that the stream channel was not located properly with respect to the path indicated in these two data sources. Thus, the channel network used in the model was adjusted to better match the actual path and sinuosity of the stream.

B. Channel Cross Sections and Bridges

In November 2005, RCD staff reoccupied and resurveyed five channel cross sections, one each from the five original reach surveys carried out starting in 2001. Three of the five were quite close, but two cross sections, one near Jefferson and one near Big Ranch Road, led to further research and adjustments. The survey at Big Ranch was found to have a discrepancy traceable to use of a questionable benchmark, and since the original survey fortunately included a point of known elevation on the bridge, we were able to correct the original data to agree closely with the resurvey. This correction affected 2 cross sections.

The survey near Jefferson was more problematic. The first cross section resurveyed was approximately 0.2 m (0.7 ft) higher than in the old survey, which might have been attributable to deposition in the channel. However, as we added new cross sections around the Jefferson and Trower bridges (to be described below), we found discrepancies in top-of-bank elevations, which made us suspect datum inconsistencies in a reach survey carried out in 2002. Accordingly, a sample of the cross sections in that original survey were re-occupied as accurately as possible and used to correct the datums of the original cross sections. The following procedure was used:

- Six cross sections were resurveyed, distributed over the original survey (from above Jefferson through most of the high school campus);
- A desired adjustment was calculated for each, by comparison of the bank heights of the new and old cross sections;
- All the remaining old cross sections were adjusted by linear interpolation.

A total of 17 cross sections from this 2002 reach survey were either replaced by new ones or adjusted for datum in this manner.

Besides verifying the existing cross sections in our model, it was important to add new ones in the vicinity of all the bridges we intended to add to the model. The bridges added to the model are shown in Table 4 (in section V below), which shows basic information about all the bridges in the model. In general, we made sure there were two cross sections in the model associated with each bridge, one upstream of the bridge and one downstream, each separated from the bridge by a distance equal to the opening width.

The locations of all cross sections on the creek are measured by the longitudinal distance in meters from the upstream end of the channel, the *chainage*. In order to maintain the spatial locations of the existing cross sections, we adjusted the chainages to agree with the new channel path information derived from the relocating of the channel path based on the DEM and the aerial photos. The use of the DEM information to model floodplains will be discussed below; the revised chainages of branch ends and bridges are shown in Figure 2.

C. Floodplain Topography

For this project, DHI used a DEM of Napa County to derive long cross sections for the creek. This DEM was prepared in 2003 from LiDAR Ortho data collected in 2002 for Napa County. Further information on this dataset is available from the Napa County Information Technology Services (ITS) Department.

D. Hydrometric Data

The RCD has maintained a streamgage on Salvador Creek at Big Ranch Road (station SAL) since fall of 2003, so for this project virtually continuous discharge data were available at that point for the 2003-04 and 2004-05 water years and for a portion of the 2005-06 water year.

Rainfall records for 2003-04 and 2004-05 were available from a recent Napa County study (Jones & Stokes/EDAW 2005), for three gages in the vicinity of (although not within) the Salvador Creek watershed. Unlike the data we have used in previous years, these gage records represent essentially complete water years. The rainfall distribution used was identical to that used in the Napa County study.

In the spring of 2005, the RCD installed two recording rain gages in the Salvador Creek watershed to improve our field instrumentation. They were installed at two widely dispersed locations within the watershed, at Alston Park (station ALP) and at Vintage High School (station VHS), to provide a more representative rainfall record for future modeling purposes. These gages were in operation throughout the period from October through December 2005. The locations of these rain gages and the streamgage site are shown in Figure 3.

On December 31, 2005, Napa County experienced a major storm, by far the largest recorded storm on Salvador Creek. With the assistance of residents, RCD staff flagged high water marks along the creek from the vicinity of Highway 29 to a point past Big Ranch Road. The elevations of these high water marks were determined by level survey and the locations marked with GPS. The high water marks are shown in Figure 4. The discharge record at Big Ranch Road was complete during most of this storm, except that rising water caused a malfunction shortly before the peak, ending the record.

There is therefore some uncertainty about the peak discharge on December 31, because the record ends abruptly at 5:00 am on this day and there is the possibility of backwater from the Napa River affecting the stage-discharge relationship at the site around the time the record ends. Analysis of the rainfall and discharge data suggest that the end of the record is probably near the end of the rising limb but not quite at the peak. We do not know whether backwater from the river was affecting the gage site yet at 5:00 am, but various indicators suggest that it may have. So we have two sources of uncertainty with opposite bias. Reasoning that the potential for backwater is more important, we note that the observed record may overstate the actual discharge, so that if we take the peak value in our observed record as the actual storm peak, we will be conservative.

Since a portion of the Napa River is included in the Salvador model, discharge and level data for the river are required. For the period from October 2003 through December 2005, discharge data at Oak Knoll (USGS station no. 11458000, Napa River near Napa) are available to serve as upstream model boundary data. The river downstream boundary is provided by the predicted tidal record at Third Street in downtown Napa, for the same period.

IV. Rainfall/Runoff Modeling

A. MIKE 11 RR Model

We ran the MIKE 11 RR model using the division into four subbasins shown in Figure 2. The areas of these four subbasins are shown in Table 1. The MIKE 11 RR model was first applied to the period October 2003 through April 2005, using the County rainfall data and the RCD discharge record.

Table 1. Rainfall/Runoff Modeling Drainage Areas

Subbasin	Drainage area, km^2
North Branch (SALVO_NB)	7.571
South Branch (SALVO_SB)	2.105
Big Ranch Road (SALVO_BR)	4.761
Downstream (SALVO_DS)	4.268

In the case of a watershed like Salvador, with the RR calibration point downstream of much of the HD model, it is often necessary to run the two models in tandem to calibrate the RR model. This procedure allows the HD model to rout the RR results through the channel, which usually has the effect of attenuating the peaks somewhat. We initially chose to run the RR model alone, rather than coupling it with the HD model, because trial simulations indicated that the version of the HD model we developed last year did not attenuate the peaks more than about 5% for the period 2003-05.

Initially we set out to take advantage of the RR model's ability to model the runoff response of the watershed through a period of several seasons, by matching the overall volume, timing and shape of all the storms in this period. However, this procedure had limited success, mainly because we were unable to match the flow during the extended dry periods which occur even in the rainy season, so that the volume calibration contained a considerable error which detracted from the modeling of individual storms. Instead, the model was calibrated to match the peaks of all storms over 10 m^3/s (350 ft^3/s), since the goal of this modeling effort is to predict large storms. We did this by noting the difference between modeled and observed peaks for all storms meeting that definition, taking the average of the absolute values of this quantity, and calibrating the model to minimize this average.

The RR model thus calibrated was applied to the October – December 2005 data, including the New Years Eve storm, and was found to under predict the storm peaks in this period, especially the two highest peaks (both on December 31), which are also the two highest peaks of record. Reasoning that we should calibrate to a storm of the magnitude we are most interested in, we revised the calibration with a specific view to matching the two largest peaks. This is the approach used also in the application of the HEC-HMS model described in the next section. The final calibration of the MIKE 11 RR model will be discussed further below.

B. HEC-HMS Model

RCD performed an analysis of the Salvador Creek watershed using HEC-HMS, a software program developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) for rainfall-runoff modeling projects. The purpose of the analysis was to estimate the peak flow of Salvador Creek at the Big Ranch Road Bridge during the 100-year design storm.

RCD used an ArcView extension called HEC-GeoHMS, and the Napa County LiDAR DEM, to define the watershed boundary of Salvador Creek from the confluence of Salvador Creek with the Napa River. The resulting area was derived solely from topography. It looked good through the hills and vineyards, but we immediately realized it was not well placed through the heavily storm-drained urban areas of the watershed. Using the City of Napa Storm Drain Master Plan, we modified the drainage boundary to reflect actual conditions. The watershed area was then divided into four subbasins as listed in Table 1 above. The HEC-HMS analysis uses the same subbasins as the MIKE 11 RR model.

HEC-HMS Basin Model

Once the subbasins were delineated, RCD constructed the basin model. We selected the Soil Conservation Service (SCS) models (USDA-NRCS, 1986) to generate hydrographs for each subbasin. The SCS curve number (CN) method was selected to model the runoff volume. The parameter for this model is the weighted CN for each subbasin. To compute the CNs, we created a land use layer in GIS by delineating the agricultural, forested, grassland, urban, rural residential, and open water areas on 2002 aerial photographs. These areas were superimposed on the soil type data from the Napa County Soil Survey (USDA-SCS, 1978). Soil types included hydrologic soil groups B, C, and D. The area of each soil type within each land use category was measured, and RCD assigned a CN to each area using the SCS reference tables. Weighted average CNs were then computed for each subbasin. The selected CNs, areas, and weighted CNs are included in Appendix B.

RCD selected the SCS unit hydrograph as the transform model. The sole input parameter for this model is the SCS lag time (t_{lag}), which is calculated as 0.6 times the time of concentration (t_c). The t_c of each subbasin is the sum of the time spent as sheet flow, shallow concentrated flow, and channelized flow, along the longest flow path. Each of these times is estimated using equations provided by the SCS, which compute the times using length, slope, and estimates of hydraulic radius (R) and roughness (n). We used the DEM to create topographic profiles of the longest flow paths of the subbasins, divided the profiles by slope into sections, and computed t_c and t_{lag} using estimates of R and n. The measured lengths and slopes, estimates of R and n, and computed t_c and t_{lag} values are included in Appendix B.

The hydrographs for each subbasin were then added and routed toward the watershed outlet. RCD created two routing reaches, one to move the water generated by the subbasins west of the highway through the Big Ranch Road subbasin, and one to move the water generated by the three upstream subbasins through the downstream subbasin to the outlet of the watershed. The flow network is shown in Figure 5. RCD selected the Muskingum-Cunge model for channel routing. This model uses the length, slope, bottom width, side slope, and roughness of the reach to compute outflow hydrographs given an inflow hydrograph. RCD estimated average channel dimensions using the surveyed cross sections collected for MIKE 11 modeling, the DEM, and selected estimates of roughness for each reach. The values selected are included in Appendix B.

To complete the basin model, we added a constant baseflow ranging from 2 to 4 ft^3/s (0.05 to 0.1 m^3/s) to each subbasin. These values were based on observations of Salvador Creek between storms by RCD staff, and scaled proportionally based on the characteristics of each subbasin.

The SCS recommends a computation interval of less than or equal to 0.29 times t_{lag} of the smallest subbasin. This equaled 13.8 minutes for the South Branch subbasin; therefore, we selected a computation interval of 10 minutes.

Model Calibration

Although the SCS models can be used for analysis of ungaged basins, hydrometric data are available for the Salvador Creek watershed, as described in Section 3C. As is recommended practice for calibration of event models such as this, we selected the storm of record nearest in size to the storm of interest. Since this model will be used to estimate the flow resulting from the 100-year rainfall, a very large event, we selected the largest storm of record, the December 30-31, 2005 event, for model calibration.

Rainfall data from RCD gages ALP and VHS and streamflow data from RCD streamgage SAL for December 30 and 31, 2005 were loaded into HEC-HMS. Rain data from ALP were used for subbasins west of the highway, and data from VHS were used for the subbasins east of the highway.

The simulated outflow hydrograph for the junction point at Big Ranch Road was then compared to flow data from SAL. Model parameters (CN, t_{lag}, and Manning's *n* for the routing reaches) were then adjusted through an iterative process until the simulated hydrograph most closely matched the observed hydrograph (Figure 6).

100-Year 24-Hour Storm

The calibrated model was then used to estimate the flow at Big Ranch Road resulting from the 100-year 24-hour rainfall. The RCD selected the SCS hypothetical storm as the meteorological model, for which the inputs are SCS temporal distribution type and rainfall depth. We selected SCS Type IA rainfall distribution, which is appropriate for our region. Using the coordinates of the centroids of the subbasins, and the NOAA Atlas 2 website located at http://www.nws.noaa.gov/oh/hdsc/noaaatlas2.htm , we obtained the 100-year 24-hour rainfall depth for each subbasin. We set up a fictitious rain gage for each subbasin with the corresponding hyetograph.

Results

The results of RCD's HEC-HMS watershed runoff analysis of the Salvador Creek watershed are presented in Table 2.

Table 2. HEC-HMS Analysis Results

Location	100-year Peak Flow (ft^3/s)
Big Ranch Road	3,213
Napa River Confluence	3,910

C. Final MIKE 11 RR Calibration

The HEC-HMS event model just described provided a useful model of the creek response, which is a good basis for modeling the 100-year discharge on Salvador Creek. Since the HEC-HMS model includes its own routing component, these results are complete in themselves. To complete the MIKE 11 RR calibration, however, it was necessary to route the RR results through the HD model, so that attenuation within the channel could be taken into account.

As we began to calibrate the HD model to the high water marks of the New Years Eve storm, we found that the increase in roughness and the other steps taken to match the high water marks had the effect of reducing peak discharges, so it was necessary to make considerable further changes to the RR calibration. This required a kind of back-and-forth between the two MIKE 11 models, until we achieved a satisfactory finished HD calibration. The final RR model parameters, compared with the values used previously, are shown in Table 3.

Table 3. Comparison of MIKE 11 RR model parameters, current work vs. previous

Parameter	*U_{max} (mm)*	*L_{max} (mm)*	*CQOF*	*CKIF (hr)*	*CK1, 2 (hr)*	*TOF*	*TIF*
previous	15	150	0.6	1000	0.01	0.1	0.1
current	5 to 6	40 to 60	0.95	500	0.5	0.2	0.2

The current work uses different values of U_{max} and L_{max} for the various subbasins. The table shows the range of values for each parameter.

The striking changes in several parameters require comment. The changes in U_{max} (maximum contents of surface storage), L_{max} (maximum contents of rootzone storage), and *CQOF* (overland flow coefficient) represent an aggressive attempt to capture volume of runoff, which proved to be necessary to achieve a good HD calibration. All three are at or near their usual limits, even for a largely urbanized watershed like this one.

The changes in TOF and TIF (threshold values for overland flow and interflow, respectively) cause a slightly later start of overland flow/interflow in the fall, although the start is still rather early. The previous value for CK1,2 was far outside the normal range: it made for an extremely spiky hydrograph. The new value, while less extreme, is still outside the usual range of 3-48 hours. Finally, the change in CKIF is relatively minor and probably could be reversed.

Since the final MIKE 11 RR calibration is so intimately involved with the detailed HD calibration, no separate RR results are presented here.

V. Updating the Hydrodynamic Model

The most important improvement to the HD model was the addition of long floodplain cross sections. DHI added these by "cutting" them from the County DEM. This method combines the strengths of two very different data sources for maximum benefit: the high local accuracy of field channel surveys and the breadth of coverage of the DEM. The DEM has a high degree of resolution, given the size of the dataset, but it nevertheless fails to represent the channel banks and bottom as well as the field surveys. Since the two data sources used different vertical datums, it was necessary to convert the DEM (in NAVD 88) to the datum used in all RCD modeling (NGVD 29), using a datum adjustment of 0.799 m (2.65 ft) for conversion. Generally speaking, the elevations of the tops of banks and near-channel floodplains derived from the two data sources agreed well. However, the channel widths and depths were not represented in the DEM-derived cross sections as accurately as in the surveyed cross sections. The existing surveyed cross sections were thus imbedded in the DEM-derived cross sections by aligning the lowest elevation point from each data source, and removing the DEM-derived data from a swath equivalent to the surveyed cross section width. The resulting long cross sections are a hybrid of the two data sources, with DEM data representing the floodplains and surveyed data representing the channels. The locations of long cross sections are illustrated in Figure 7.

These long cross sections allow the one-dimensional MIKE 11 model to take the floodplains into account in a somewhat realistic manner. It is possible to implement different roughness values for the floodplains, but variations in velocity and level over the cross section are not modeled. Additionally, since this is a one-dimensional model, flow on the floodplains is not modeled explicitly.

The other major HD improvement was putting in bridges. Eight additional bridges were added to the model, making nine in all; they are listed in Table 4, along with two other bridges over the creek shown only for reference. These two, like the bridges over the North and South Branches west of the highway, are not modeled because they do not appear to constrict the flow. All the bridges except one were represented using the FHWA WSPRO method provided in MIKE 11, which is the method used for the Garfield Bridge in last year's effort. The lone exception is the Highway 29 bridge, where it seemed most appropriate to use the MIKE 11 culvert/weir combination, because the flow does pass through a pair of long culverts and because the FHWA WSPRO method seemed to restrict the flow too much. The bridges were added to the model one at a time, and the interactions among all the bridges were carefully scrutinized before proceeding. The completed model with all bridges was satisfactorily stable, with Trower Bridge the least stable one, probably because of its position immediately downstream from Jefferson. The model with bridges shows a considerable tendency to attenuate high flows, because of the backwater effects that occur when some of the bridges are submerged.

Table 4. Salvador Creek Bridges

Structure	Center Chainage, m	Modeled as	Submergence/Overflow levels, m NGVD
Railroad Bridge*	3.73		
Highway 29 Bridge	27.13	MIKE 11 culvert/weir	22.07 22.49
Biale Bridge	590.07	FHWA WSPRO	19.73 20.04
Bridge to Nowhere	982.90	"	18.15 18.47
Jefferson Bridge	1128.55	"	17.78 18.45
Trower Bridge	1200.02	"	17.47 18.18
HSB1	1408.36	"	16.95 17.31
HSB2	1537.47	"	16.43 16.72
Garfield Bridge	2019.26	"	13.42 13.72
Zerba Bridge	2615.71	"	10.93 11.22
Big Ranch Road Bridge*	2957.80		

** Bridge not included in model, shown here for reference.*

VI. Hydrodynamic Model Calibration

Since this modeling study is focused on flooding during large events, we chose the period from October to December 2005 for HD calibration. This period included a variety of storms, including the major New Years Eve storm for which good high water mark coverage was available. The main calibration was for the New Years Eve storm. In carrying out the calibration, our goal was to maximize the ability of the model to predict the effects of a possible future large flood. For that reason, we established calibration criteria that focused on peak water levels and discharges, paying less attention to smaller peaks, timing issues and overall volume. We used a high water criterion of 0.25 m (0.82 ft) or less; that is, we aimed for the average of the absolute values of the error at each high water mark (simulated minus observed water level) to be less than 0.25 m. At the same time, we tried to keep the discharge peaks within 15% of the observed values. This latter criterion was applied to each of the two separate peaks on New Years Eve.

Calibration to the New Years Eve high water marks was complicated by the high water level in the Napa River at the peak of this flood. Our first simulations indicated that the water surface slope in the lowest part of the creek, where we had good high water mark coverage, was flat at the peak of the storm, indicating a strong backwater influence from the river. Since the river was not the focus of this study, we felt justified in raising Manning's n for the river sufficiently (to a value of 0.94) to match the high water marks in the lower creek. Incidentally, the use of a predicted tidal record for the downstream river boundary appeared not to affect the results at Salvador; the tidal signal was largely overwhelmed by the freshwater flood and was invisible except in the vicinity of the boundary.

With the downstream level established in this manner, we adjusted Manning's n by reach to match the high water marks and found that higher values were required than we had expected. The roughness varied over a range from 0.045 to 0.1; Table 5 shows the values by reach. Two reaches show an especially high value of Manning's n, one around Biale Bridge (400 – 875) and a second, longer reach in the lower part of the creek. Although these values are primarily the result of the calibration process, there is some reason to think they are physically appropriate as well. The Flood District staff reports that there are extensive gravel bars in the vicinity of Biale Bridge, associated with a large storm drain outfall and the bridge pier, which have a lot of aquatic primrose growing on them (Mike Forte, pers. comm.). With regard to the lower section of the creek (from 1625 to 3811), there is some evidence of high roughness in at least a portion of this reach (Riechers, 2001).

Table 5. Manning's n values for Salvador Creek, by reach

Chainage	Manning's n
0 – 400	0.06
400 – 875	0.10
875 – 1300	0.06
1300 – 1550	0.045
1550 – 1625	0.06
1625 – 3811	0.10

In addition to adjusting roughness by reach, we varied roughness within individual cross sections. Table 6 shows the multipliers used to calculate the increased roughness for flow out of the main channel. For each reach indicated, the roughness multiplier shown is applied to high flow areas of all cross sections within the reach. For a cross section in the vicinity of Biale Bridge (chainage 400 – 875), for example, the roughness in the main channel (0.1) is increased by a factor of 1.5 to 0.15 in the high-flow portions of the cross section. As the table shows, roughness is increased by 50% in high flow areas in two reaches, with no increase in the rest of the creek.

Table 6. Variation of roughness within cross sections

Chainage	High Flow Multiplier
0 – 27	1.0
27 – 983	1.5
983 – 2019	1.0
2019 – 3811	1.5

In addition to the roughness calibration, we made two further changes to improve the HD calibration. First, we adjusted the coefficient of discharge (*Cd*) for the submergence condition at several bridges. Reducing *Cd* reduces discharge through the bridge opening when it is submerged, and increasing *Cd* does the opposite. For the bridges at the high school and further downstream, *Cd* was reduced considerably below the default, and for the Biale Bridge it was increased somewhat. Second, we revised the RR calibration. As was described above, the increase in roughness and the other steps taken to match the high water marks had the effect of reducing peak discharges, so we made further changes to the RR calibration to compensate for these reductions.

The results of the calibration are illustrated in Figures 8 and 9. Figure 8 shows the calibration to the high water marks along the creek, and Figure 9 shows the simulated vs. observed hydrograph at Big Ranch Road. In Figure 8, the maximum water level during the simulated storm is plotted on the vertical axis against chainage along Salvador Creek, with the corresponding observed high water marks shown as points for comparison with the continuous simulated line. The average error falls just short of our success criterion, at 0.257 m; however, the criterion was met at 20 out of 25 individual locations. The discharge results illustrated in Figure 9 do meet the discharge criterion, with peak discharge errors of 12% and 7% for the two New Years Eve peaks.

This calibration is aggressively focused on matching high water marks, and it is possible that some of the high water observed may be associated with episodic debris blockage and other one-time events that are not necessarily associated with the typical response of the channel. In that sense, some of the roughness values or bridge coefficients may be too high. However, episodes of debris blockage and the like are real possibilities in the case of a large flood, and it seems appropriately conservative to us to try to match the observed effects of this storm as faithfully as possible.

DHI prepared a flood map for the New Years Eve storm using the results of the MIKE 11 simulation and the DEM, which appears in Figure 10. The flood map interpolation/extrapolation routine takes the modeled water levels at MIKE 11 cross section locations, interpolates water levels between cross section locations, and extrapolates the water levels out onto the floodplain based on the DEM topography to derive a map of the inundated area. Thus it should be borne in mind that MIKE 11 flood maps are not a direct representation of modeling results; rather, they utilize the 1-d modeling results to develop 2-d flood maps, and the model does not explicitly simulate flow on the floodplains or return flow back to the channel.

The flood map is a useful tool for visualizing and evaluating the model results. There is one area where the extent of flooding seems definitely underpredicted: the creek reach between the Garfield and Zerba bridges. As one might expect, this reach is also one where the modeled water level remains furthest below the surveyed high water marks (see Figure 8). In other areas, the flood map is believable. There is some flooding west of the highway, which gets onto the railroad tracks but not the highway. There is some flooding at the edges of the vineyards, and the backwater flooding from the river on the right bank downstream of Big Ranch Road is about right.

The low modeled levels between the Garfield and Zerba bridges may have to do with the distribution of runoff entering the creek between Highway 29 and Big Ranch Road. In the model this runoff is distributed linearly, whereas in fact the distribution of storm drains may be rather different. Adjusting the RR inputs to the HD model, to make them correspond to the actual storm drain layout, would be a worthwhile step to take in the future.

VII. MIKE 11 Model Applications

A. 100-year Storm

We modeled a 24-hour design storm for a recurrence interval of 100 years, using the same SCS type IA storm and the same rainfall depth as for the HEC-HMS model described above. We used both RR and HD models as calibrated above.

In modeling a short synthetic storm like this, the choice of initial conditions is extremely important no matter what the model. With the MIKE 11 model, the most important decisions concern the initial relative moisture content in both the surface zone (U/U_{max}) and the root zone (L/L_{max}). To determine these values in a systematic way, we made use of our calibrated model for the first part of water year 2005-06: we looked at hourly values of relative moisture content throughout the portion of the high flow season modeled (December through early January) and calculated an average for both U/U_{max} and L/L_{max} for each subbasin of the Salvador Creek watershed. These values, shown in Table 7, were used as an initial condition for the model.

Table 7. Average relative moisture in surface and root zones, Salvador Creek, December 2005 and early January 2006

	North Branch	South Branch	Big Ranch	Downstream
average relative moisture, L/L_{max}	0.66	0.72	0.72	0.65
average relative moisture, U/U_{max}	0.60	0.58	0.57	0.60

The peak water level along Salvador Creek for the 100-year storm is shown in Figure 11. For comparison, the high water marks and modeled water level for the New Years Eve storm are shown in the same figure. The variation in peak discharge along the creek is shown in Figure 12. The figure shows a striking degree of attenuation of the flood peak along Salvador Creek: a peak value of about 73 m^3/s (2578 ft^3/s) at the highway is attenuated to about 55 m^3/s (1942 ft^3/s) at Big Ranch Road, even though the entire runoff from the Big Ranch Road subbasin is entering the creek evenly over the reach from the highway to Big Ranch Road. The volume of runoff is all there; it is simply being slowed down by attenuation.

We considered several possible explanations for this high degree of attenuation. We found that the results had very little sensitivity to cross section length and channel roughness, so these model features do not seem to be causing the strong attenuation observed in the model results. The main reasons seem to be (1) the relatively high degree of overbank flow spilling onto the floodplain along the upper reaches of the creek and (2) the effects of flow backing up and in some cases overtopping the numerous bridges in the model. A simulation run with all bridges removed shows about half as much attenuation, so that both these factors seem to be important. It is important to note that this is a synthetic storm with a brief period of very intense rainfall, producing an extreme response which may be particularly sensitive to attenuation by the effects of structures and floodplains. The distribution of rainfall during an actual storm event of a similar magnitude may be of a lower intensity and a longer duration than the synthetically generated event used here.

The MIKE 11 and HEC-HMS results for the 100-year storm on Salvador Creek make an interesting contrast, which raises more questions than it answers. The HEC-HMS results are relatively straightforward and consistent with other modeling done by RCD on Tulucay (Blank, 2005) and Napa Creeks (Zlomke et al., 2005). However, the MIKE 11 results show us that there may be more to the story for this particular urban creek, which has the unique situation of having considerable hydraulic attenuation from the moment it acquires a defined main channel.

The MIKE 11 RR model alone produces a considerable amount of rainfall, but the HD model attenuates a great deal of it. To some degree this is an artifact of the relatively brief synthetic design storm we are considering; the historical storm of New Years Eve does not have nearly as much attenuation. However, it is also an indication of the importance of hydraulic factors associated with bridges and channel-floodplain interactions, which HEC-HMS does not readily take into account. The MIKE 11 model indicates that these elements have a strong influence on attenuation and flooding during large magnitude storm events in this system, so their inclusion is of particular importance in simulating such events. While recognizing the value of the HEC-HMS work, we therefore believe that the MIKE 11 work reported here provides a better basis for considering improvements on Salvador Creek. It is important to note the limitations of the MIKE 11 HD model, however, and in particular the fact that the model does not consider the 2-d flow on the floodplains or the flow of overbank water returning to the channel.

Figure 13 shows a flood map for the 100-year event, as modeled in MIKE 11. There are two areas of evident increased flooding over the New Years Eve storm, the junction of North and South Branches west of the highway (chainage 0 on the main channel) and the area just upstream of the Biale Bridge (chainage 590).

B. Other Scenarios

We used the 100-year simulation as a platform to test the effects of various scenarios.
First we considered the creation of flood terraces along the creek between Highway 29 and Jefferson Street. We began with a continuous flood terrace where the current maintenance road is, within the space of the parcels/easements owned by the Napa County Flood Control and Water Conservation District. The Flood District has control, either by outright ownership or easement, of a continuous band 66 ft (20.1 m) wide, from near Highway 29 to Jefferson. We modeled a continuous flood terrace at the level of the two-year 24-hour storm, taking all available space outside the creek within this 66-foot band. This level was defined by running a two-year storm, specifically the SCS type I-A 24-hour storm as in section IVB, with the rainfall depth adjusted accordingly. Since the two-year water level was near or over the top of bank, we adjusted the two-year water level down at several cross sections, to a level consistently below the top of bank.

The reduction in water level under this scenario was slight, so we extended the idea by adding 250-foot (approximately 75-meter) wide farmable flood terraces along the left-bank vineyards in this reach, at the same adjusted two-year water level. These flood terraces would be bounded by a rolling levee on the side away from the creek, to provide more secure flood protection there in partial compensation for the increased risk of flooding on the terraces themselves. We also considered combining these terraces with removal of the two farm bridges (Biale, Bridge to Nowhere) in this reach, which may become superfluous or be replaced by pedestrian bridges in the future. Both these simulations are considered in a spirit of exploring hypothetical possibilities; the landowner has not to our knowledge been consulted about them.

Figure 14 shows the effects of two scenarios on peak water levels, terraces alone and terraces in combination with the bridge removals, along with both the 100-year and the unadjusted two-year water levels for comparison. The figure shows a small improvement in water level from the terraces, but most of the improvement is after chainage 700, where the flood map does not show flooding. Removal of the two bridges removes their backwater effects, but even with the terraces the peak water level is increased somewhat just downstream of the bridges.

Since the 100-year flood map showed considerable flooding west of the highway, we modeled an increase in the capacity of the Highway 29 bridge by the addition of a third culvert like the two existing ones. Figure 15 shows the results of this scenario on the main channel of the creek. There is a dramatic reduction in peak water level at the beginning of the main channel just upstream of the highway (chainage 0), but there is a slight increase beginning just before the Biale bridge.

It is important to note that looking at the change in peak water level as a result of a given scenario alone may not reveal the full impact of the scenario implementation. For example, the implementation of rolling levees may lead to minimal reductions or even increases in peak water levels while at the same time decreasing flood extents by preventing the lateral spreading of flow out onto the floodplains.

These scenarios suggest that the bridge improvements modeled have definite local benefits, but that they have modest negative consequences downstream, which are not likely to be compensated for by the flood terraces modeled. At the same time, there is reason to think that these bridge improvements may be made in the future, and it is important to consider what may be done to ensure 100-year flood protection to businesses and residents along the creek. The next section considers this question in light of these modeling results.

VIII. Conceptual Plan

The City of Napa has requested a conceptual plan that would provide for 100-year flood protection for residents and businesses along the creek while preserving and enhancing its associated environmental amenities. While the scenarios considered in this study do not explicitly describe a plan for 100-year flood protection, they do offer some useful hints. We have therefore prepared the following notes toward a conceptual plan to assist the City in its planning efforts.

These notes are organized under two headings, *Opportunities and Constraints* and *Recommended Elements*.

A. Opportunities and Constraints

The 100-year flood map (Figures 13a – 13e) shows several areas of flooding concern. The most evident one is the area just upstream of the highway, where the North and South Branches meet at the so-called "Solano Y." The inundation is largely confined to open fields, but some structures may be flooded in the 100-year event. Of course, the DEM used to represent the floodplain topography does not include finished floor elevations, so the extent of any structural flooding is not shown in the flood map. In addition, it is important to note that this area may become more intensively developed as the City of Napa grows, with the potential for greater flood risk.

A second area of flooding concern is the low area around Lassen Street, where the flood map shows extensive local street flooding. Of course, street flooding in the case of a 100-year storm is appropriate, since the streets are intended to function as a secondary drainage system during rare events like this. Other areas of flooding shown on the flood map appear to be relatively minor, although a note of caution is warranted. There is at least one flooding area not supported by the model but reported to have experienced flooding, the area around Garfield Bridge, which should be considered a third area of flooding concern.

The area downstream of Big Ranch Road, which flooded in the New Years Eve storm, appears to be strongly dependent on the water level in the Napa River, so we do not see a risk for flooding in this area from flow in Salvador Creek.

While the areas of flooding concern noted are real, it is important to remember that the modeled extent of inundation during the 100-year event is relatively small compared to that associated with the Napa River and Napa Creek.

Addressing these areas of flooding concern requires taking into account the particular hydrology of Salvador Creek, which has an interesting mix of features characteristic of an urban stream:

- Much of the watershed is urban, with the extensive roofs and pavement that make for more and quicker runoff. The most recent development has been accompanied by flood attenuation structures, but there are relatively few of these and they are sized to be effective only in the 100-year storm.
- The creek channel between the highway and Jefferson was relocated in the 1960's and the public easement has limited width.
- The creek experiences significant attenuation of flood peaks (especially design floods) beginning relatively high in the watershed, at the beginning of the main channel just west of the highway. The modeled 100-year peak discharge actually decreases from there in the downstream direction.
- The level of the two-year flood is surprisingly high. Residents report that the creek is up around the top of bank in every big storm, and the model bears this out. To the extent that this is due to high discharge, it means that shear stress at the toe of bank is often high and that bank instability will likely be a problem.
- According to the calibrated model, roughness values are very high in some reaches of the creek.

Because of the hydrology of the creek, several problems and constraints arise. Perhaps most important, the high level of the two-year flood suggests that the creek will tend to have bank stability problems at critical points, which are particularly important because the public easement is so narrow. This is not news to the Flood District staff, who have responsibility to maintain the creek banks and are doing a conscientious job of repairing damaged spots with bioengineered solutions. These repairs have the potential to increase roughness and therefore increase water levels, which already show the effects of high roughness in some places.

Managing the creek to reduce areas of elevated roughness would be desirable to reduce flood levels, but this has the potential to increase discharge (and therefore velocity) and therefore to increase shear stress on the lower banks. Although reducing water levels might be thought to reduce stress on banks, in fact shear stress is proportional to the velocity gradient in the vertical direction, so that anything that is increasing the mean velocity while reducing depth (as happens when roughness is increased) increases shear stress. Therefore, it appears that reducing channel roughness should be avoided if it would affect the hydraulic regime in areas where bank stability is a problem. This is the case in the Lassen Street area.

The bridges between the highway and Big Ranch Road play an important double role. They tend to become submerged in large storms and to cause much higher water levels immediately upstream, but when this happens they reduce both water levels and discharge downstream by "metering" the flow. No one likes to see submerged bridges in a storm, but any bridge improvements made have the potential to increase flooding downstream.

B. Recommended Elements

We recommend that the City of Napa consider the following as elements of a conceptual plan for the creek:

Continue to maintain creek banks with bioengineering methods. Manage the roughness in the creek so as to maintain but not increase the existing roughness where bank stability is a problem. Elsewhere, consider managing the creek to reduce high roughness values.

Retrofit attenuation structures installed with recent development, so that the two- to ten-year floods are attenuated as well as higher-return-interval events.

Relieve bottlenecks to the flow caused by bridges at the highway and along the vineyards.

- Relieve flooding west of the highway by increasing the capacity of the Highway 29 Bridge. The 100-year flood shows considerable inundation here, including part of the highway, and any plan for the creek should include at least an increase in bridge capacity from two culverts to three.
- Remove the Bridge to Nowhere and either remove the Biale Bridge or replace it with an elevated footbridge, above the 100-year water level. These bridges have the potential to become submerged and exacerbate flooding in the problem reach around Lassen Street.

Consider a combination of flood terraces and limited flood walls to contain the 100-year flow in the vicinity of Lassen Street and to make sure that there is no net increase in water levels at Jefferson Street or beyond. If the bridge improvements described above are carried out, some such measure is essential, to avoid increasing flood risk downstream. Consider the following:

- Flood terraces where the maintenance road is between Highway 29 and Jefferson, using the available width of the public easement. It will probably be necessary to set the terraces considerably lower than the two-year flood level, as has been proposed for Napa Creek under the current Napa River/Napa Creek Flood Protection Project, to increase capacity. This will mean that the terraces will be wetted more often, and they may have stability issues. Because of the limited width of the public easement, these terraces will probably still have to be supplemented by farmable terraces in the vineyards.
- Flood terraces in the vineyards, separated by the remaining land by rolling levees; they would still be farmable.

If the combined effects of flood terraces and bridge removal show continued flooding around Lassen Street under the 100-year flood, it may be necessary to determine whether structures will in fact be flooded. If that turns out to be so, consider flood walls to contain the 100-year flow around Lassen Street. These raise the problem of interior drainage, and they must be designed to avoid increasing water levels downstream, so they are the measure of last resort.

The flooding concern in the vicinity of Garfield Bridge was not addressed in this study, but it would be appropriate to consider the measures modeled previously to lower levels in that area (Zlomke et al., 2005).

Figure 1. Salvador Creek Watershed Location Map

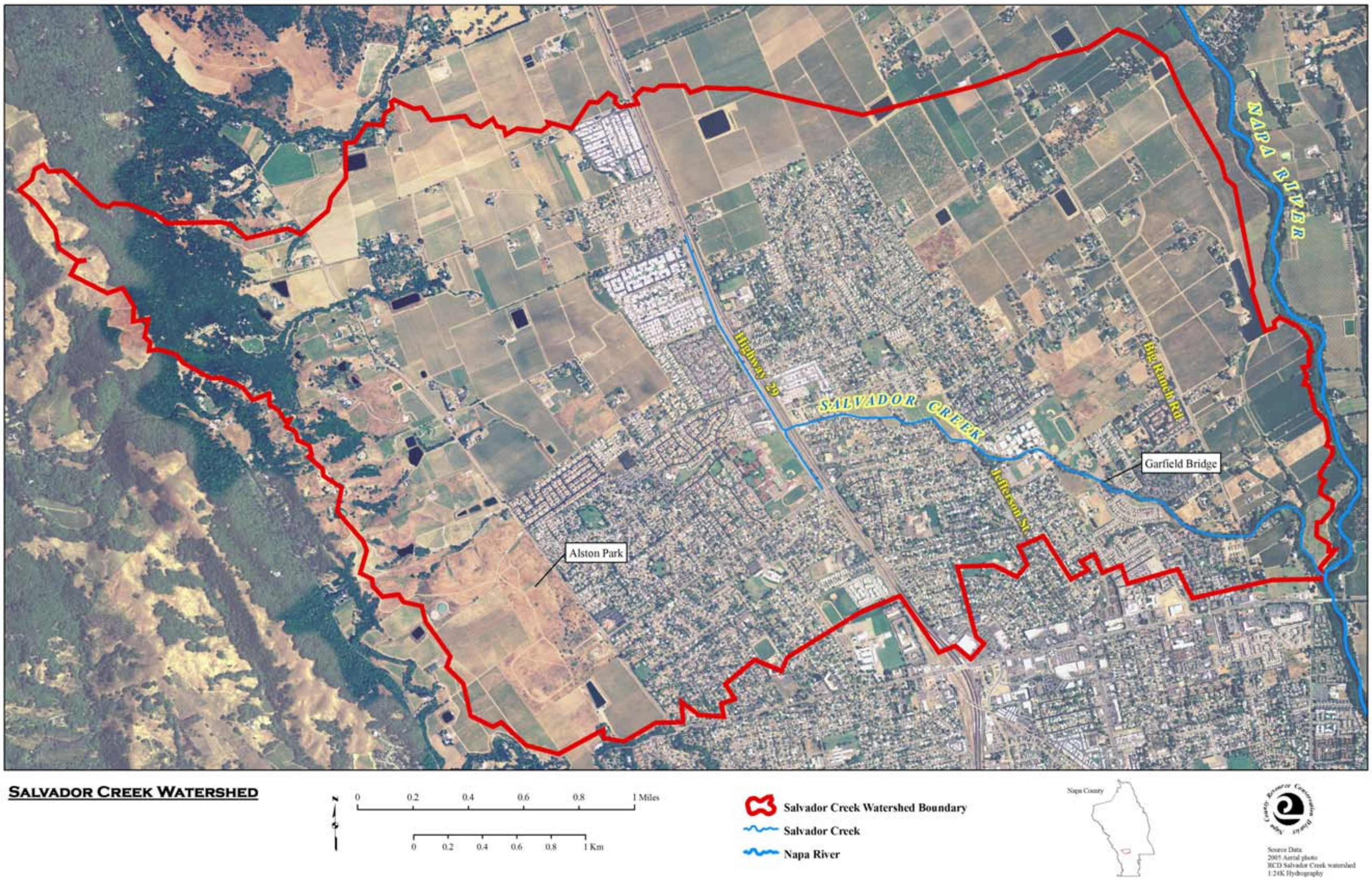


Figure 2. Improved Salvador Creek Model

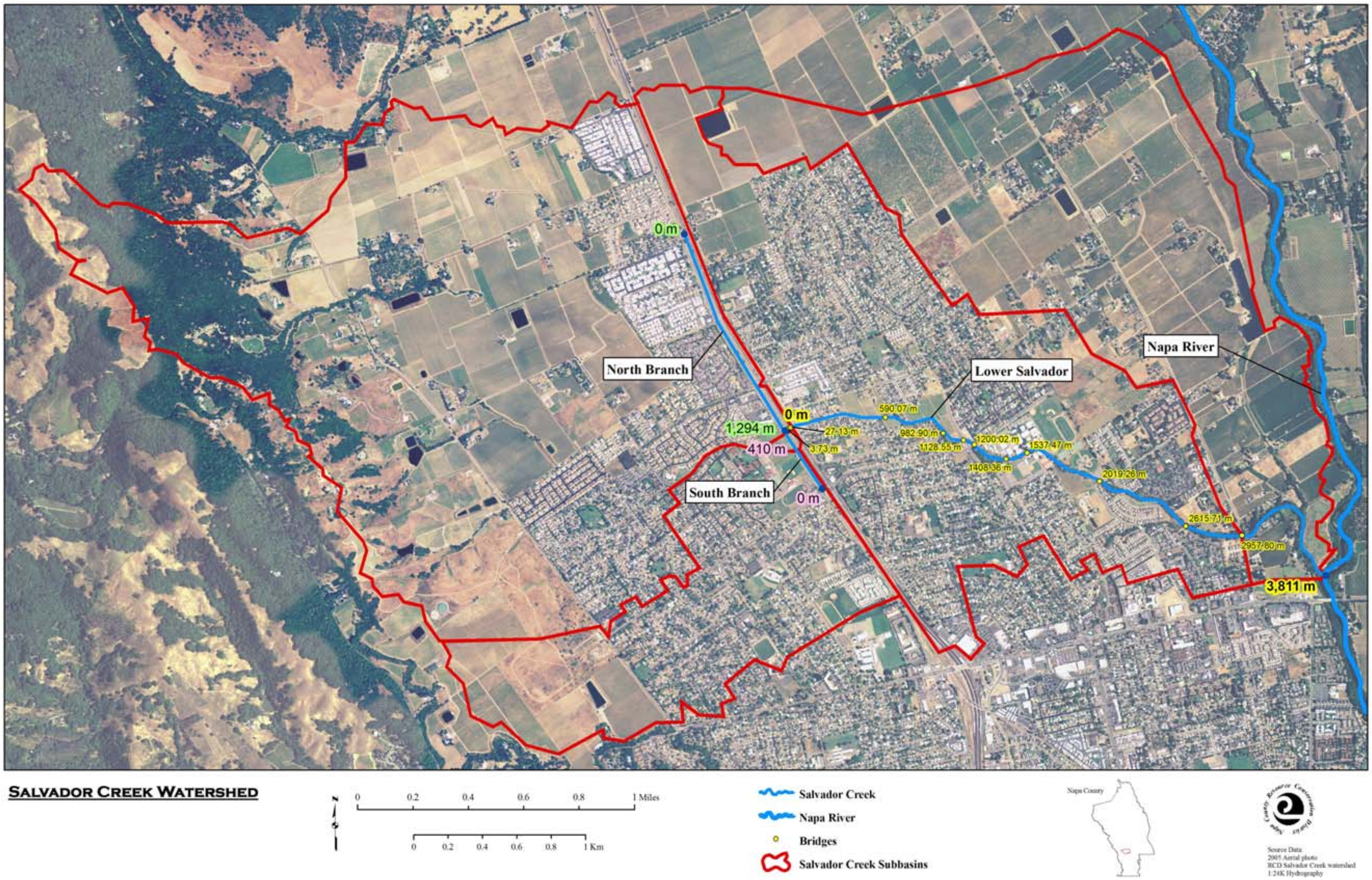


Figure 3. Salvador Creek Field Instrumentation

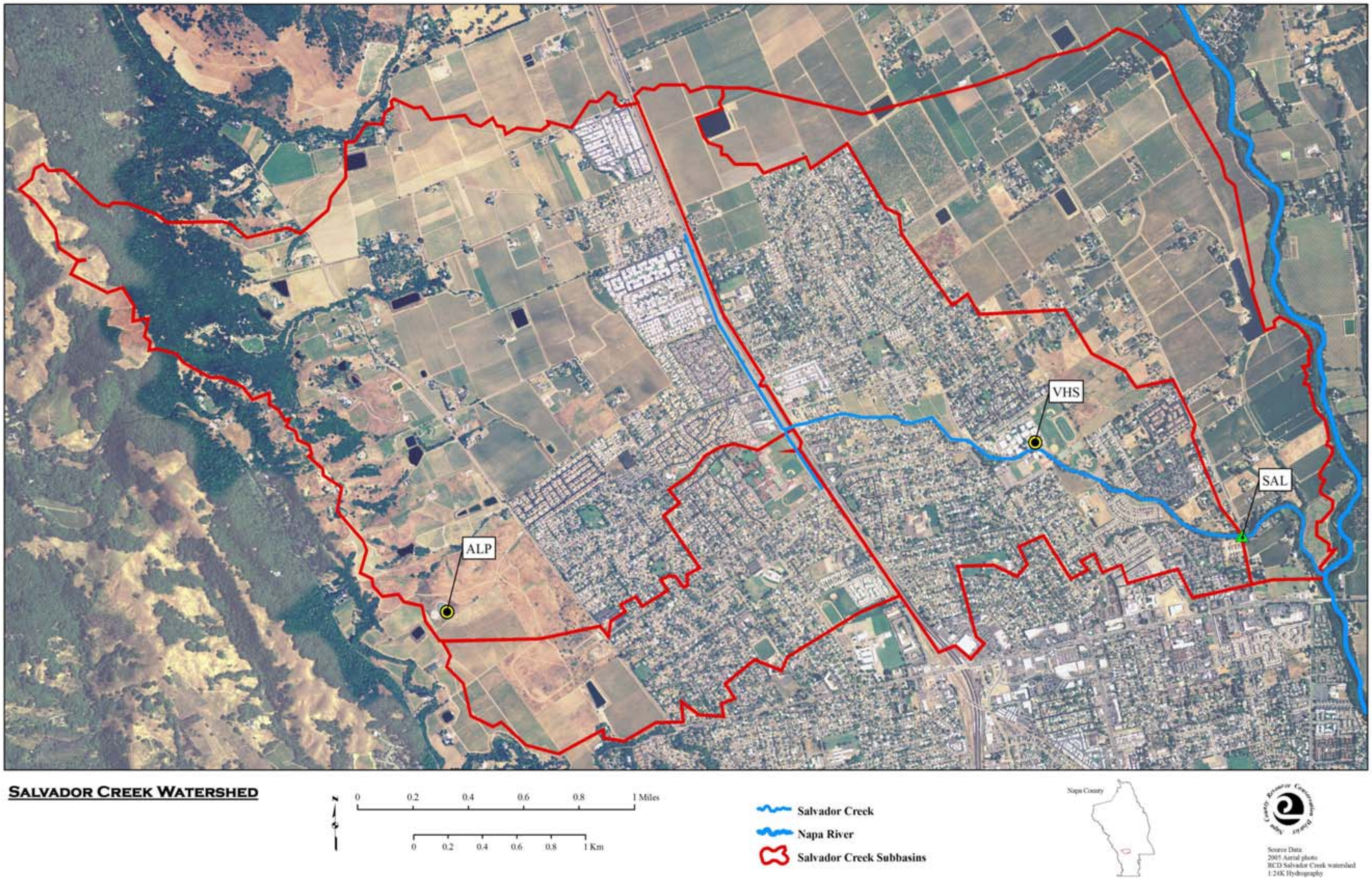


Figure 4. New Years Eve Storm: High Water Marks

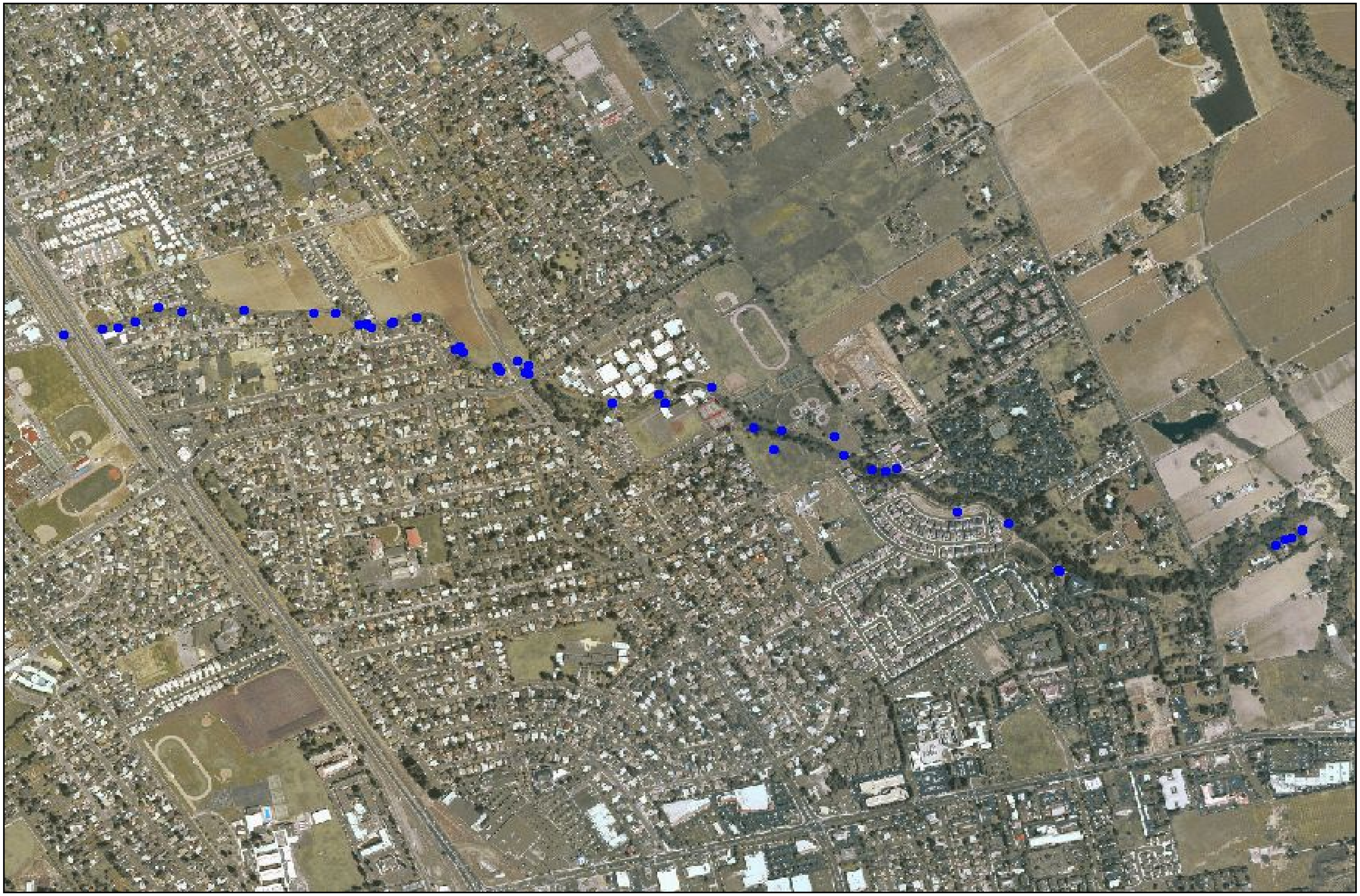


Figure 5. HEC-HMS Basin Model and Flow Network

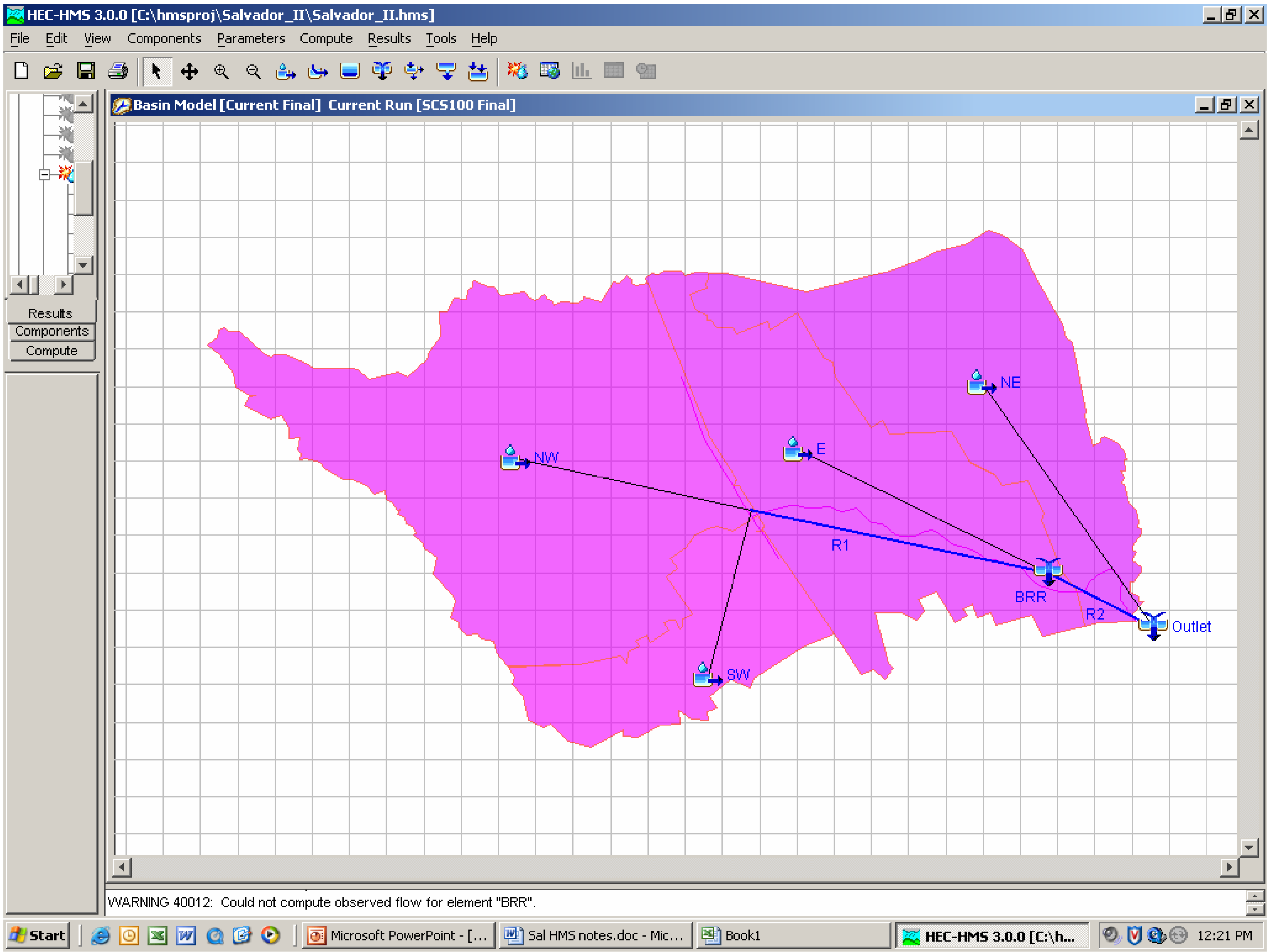


Figure 6. HEC-HMS Calibration, New Years Eve Storm

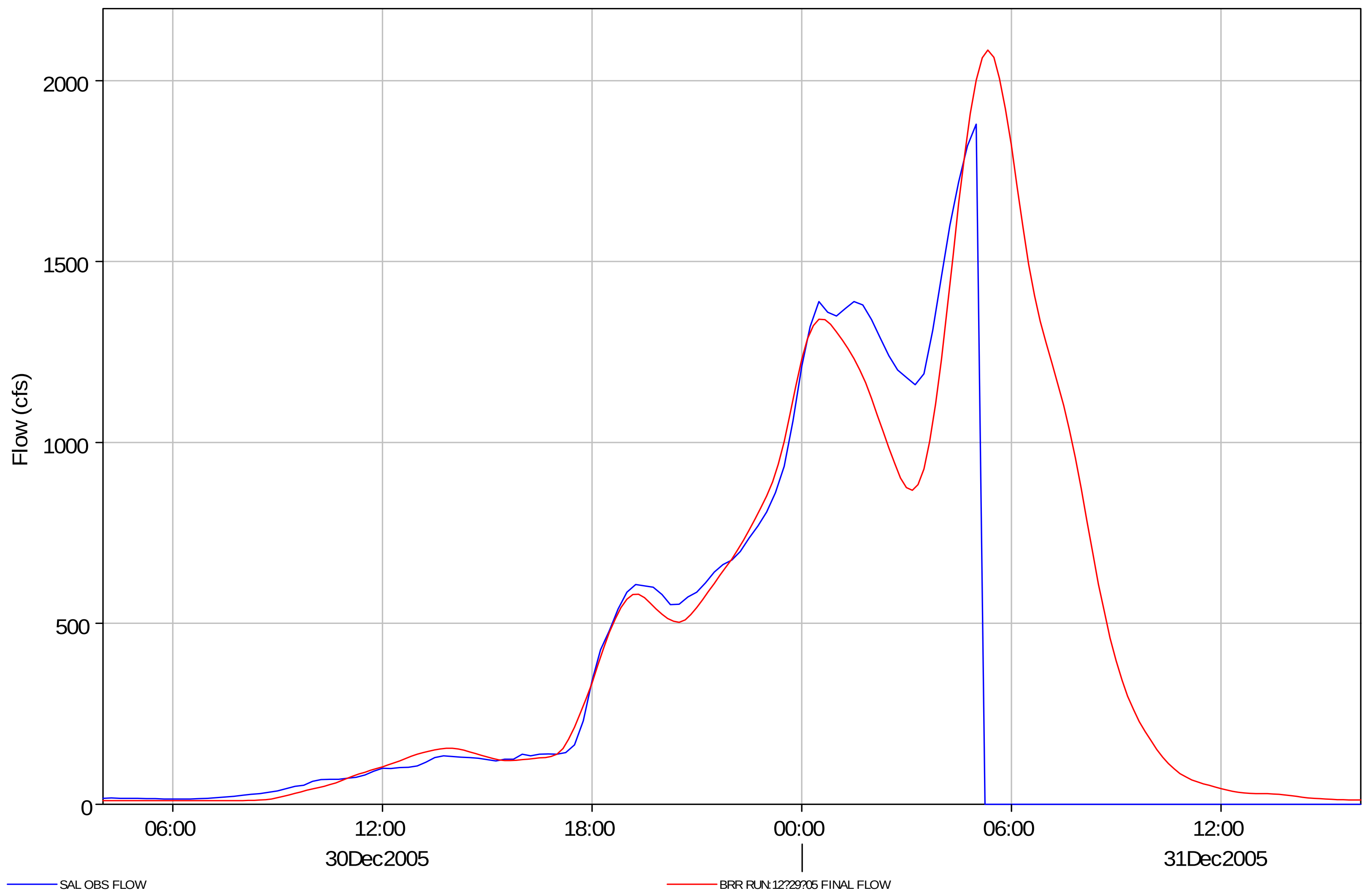


Figure 7. Locations of Long Cross Sections

Figure 8. HD Calibration: Peak Water Level, New Years Eve Storm
Blue symbols indicate observed high water marks

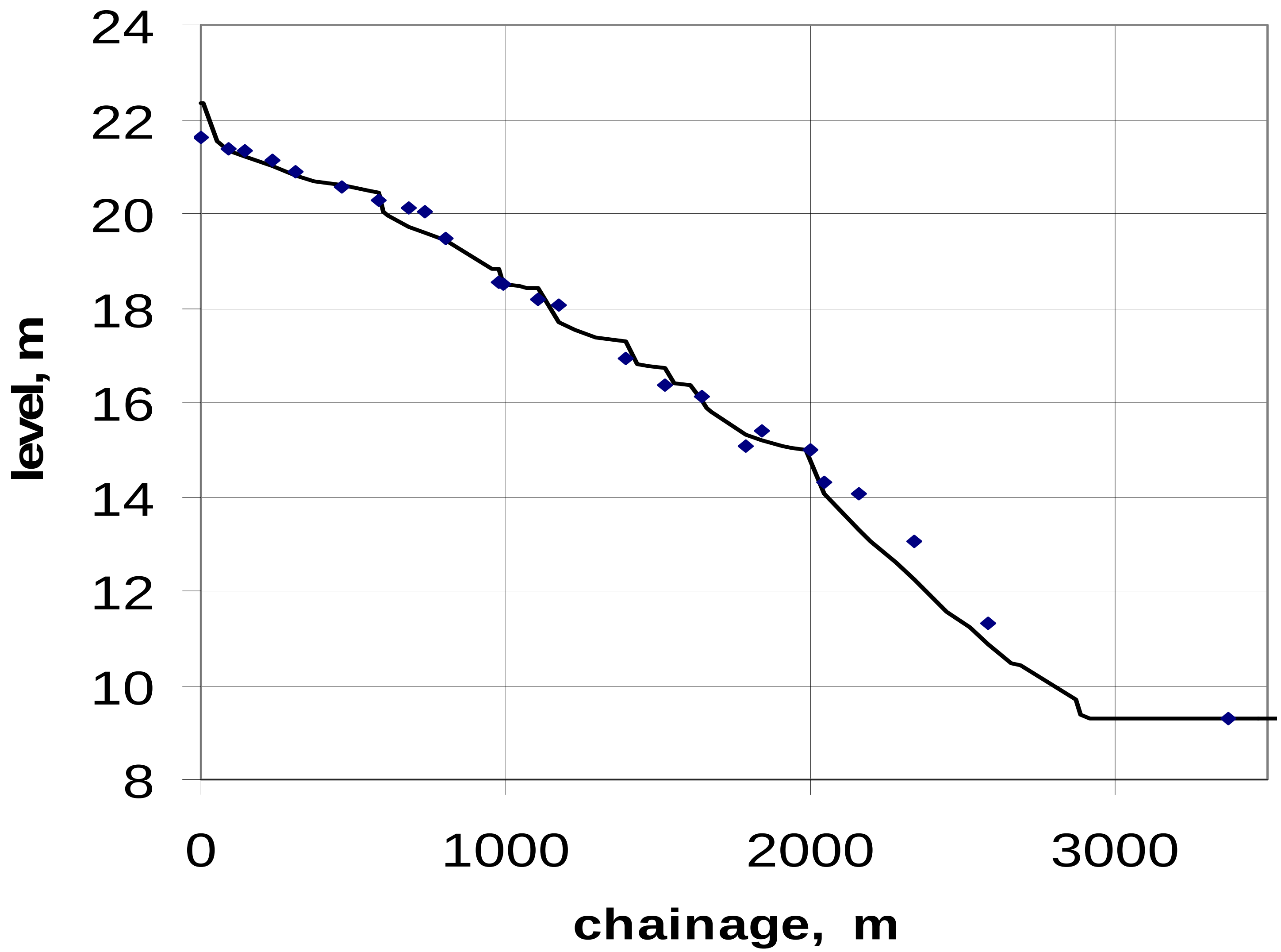
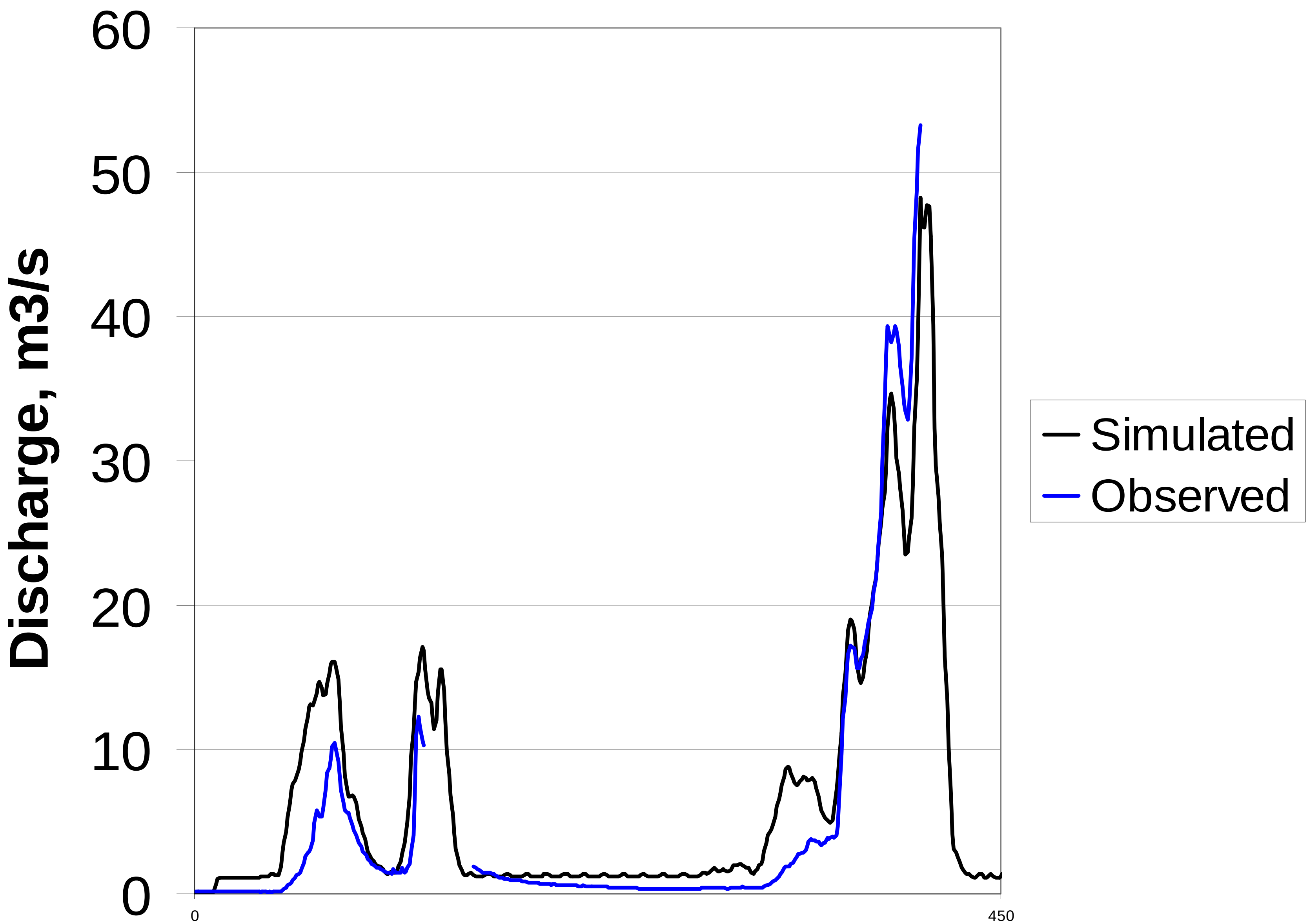


Figure 9. HD Calibration: Discharge at Big Ranch Road
December 27 – 31, 2005



Figures 10a-10e. New Years Eve Storm Inundation Maps
Figure 10a.

Figure 10b.


Railroad Bridge
Hwy 29 Bridge
Biale Bridge
Bridges
High Water Marks
Stream Network
Inundated Area (12/31/05)
500 250 0 500 Feet

Figure 10c.

Biale Bridge
Bridge to Nowhere
Jefferson Bridge
Trower Bridge
HSB1
Bridges
High Water Marks
Stream Network
Inundated Area (12/31/05)
500
250
0
500 Feet

Figure 10d.

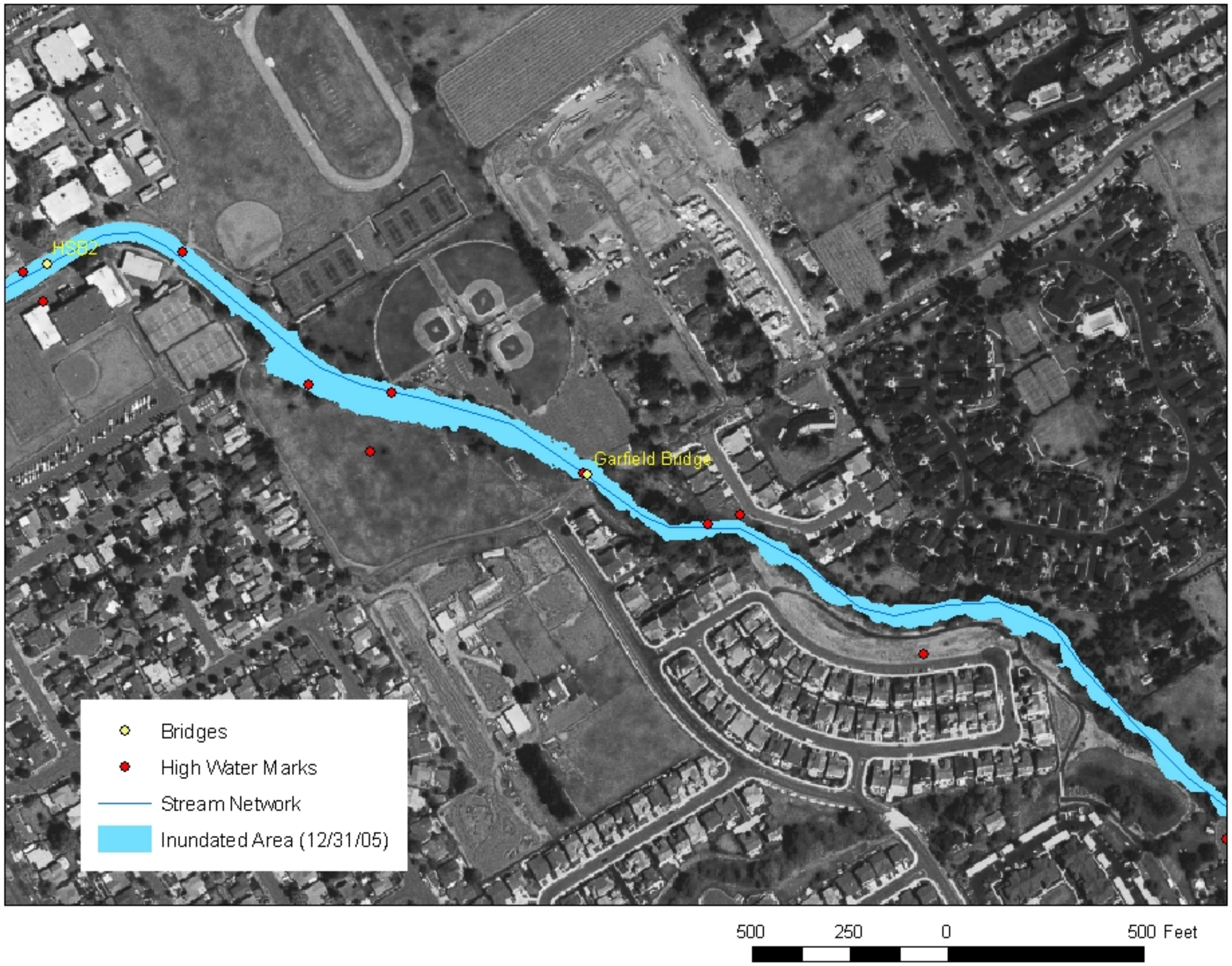

HSB2
Garfield Bridge
Bridges
High Water Marks
Stream Network
Inundated Area (12/31/05)
500
250
0
500 Feet

Figure 10e.


Big Ranch Road Bridge
Bridges
High Water Marks
Stream Network
Inundated Area (12/31/05)
500
250
0
500 Feet

Figure 11. 100-year Storm Peak Water Level

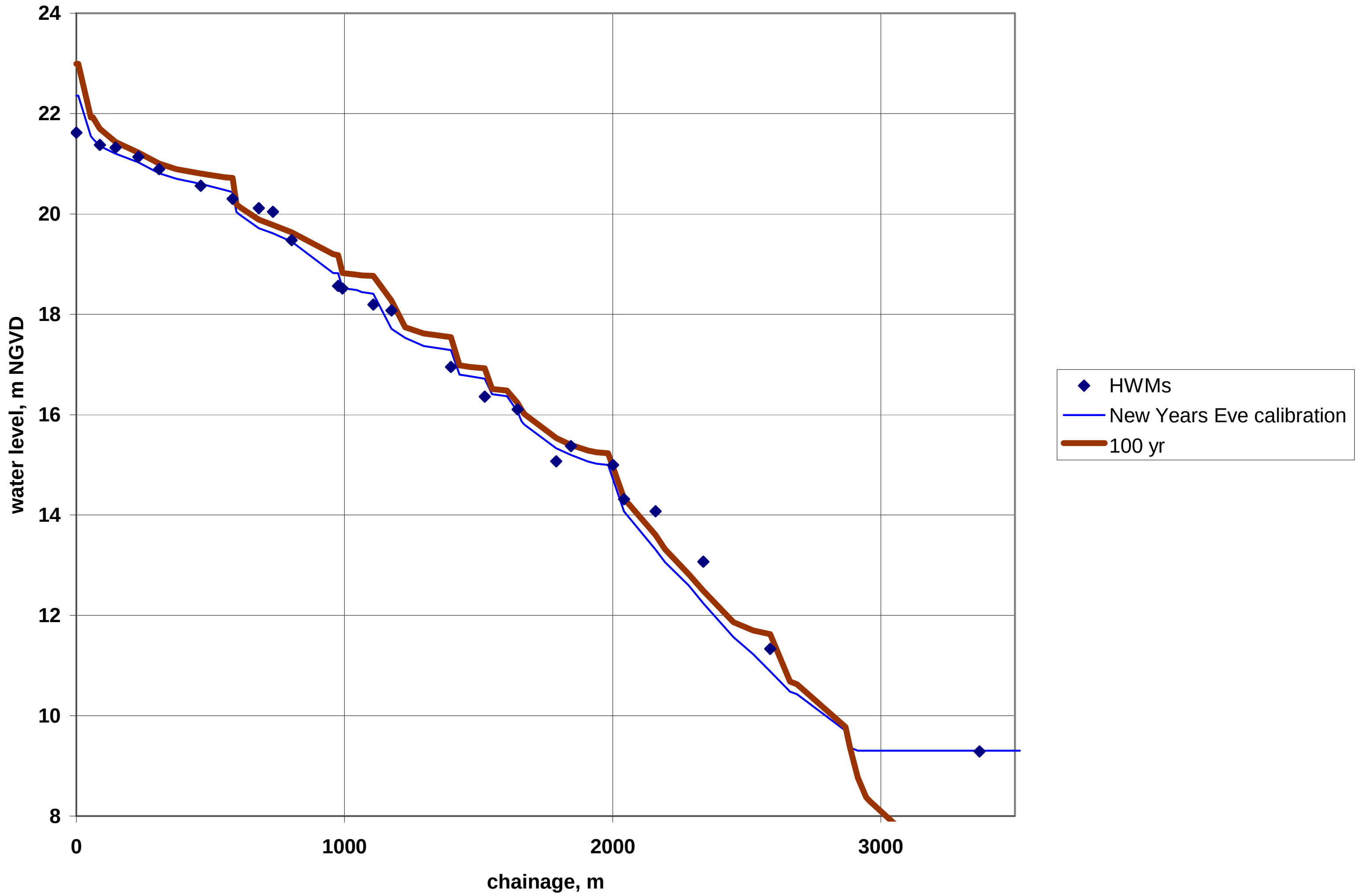


Figure 12. 100-year Storm Discharge

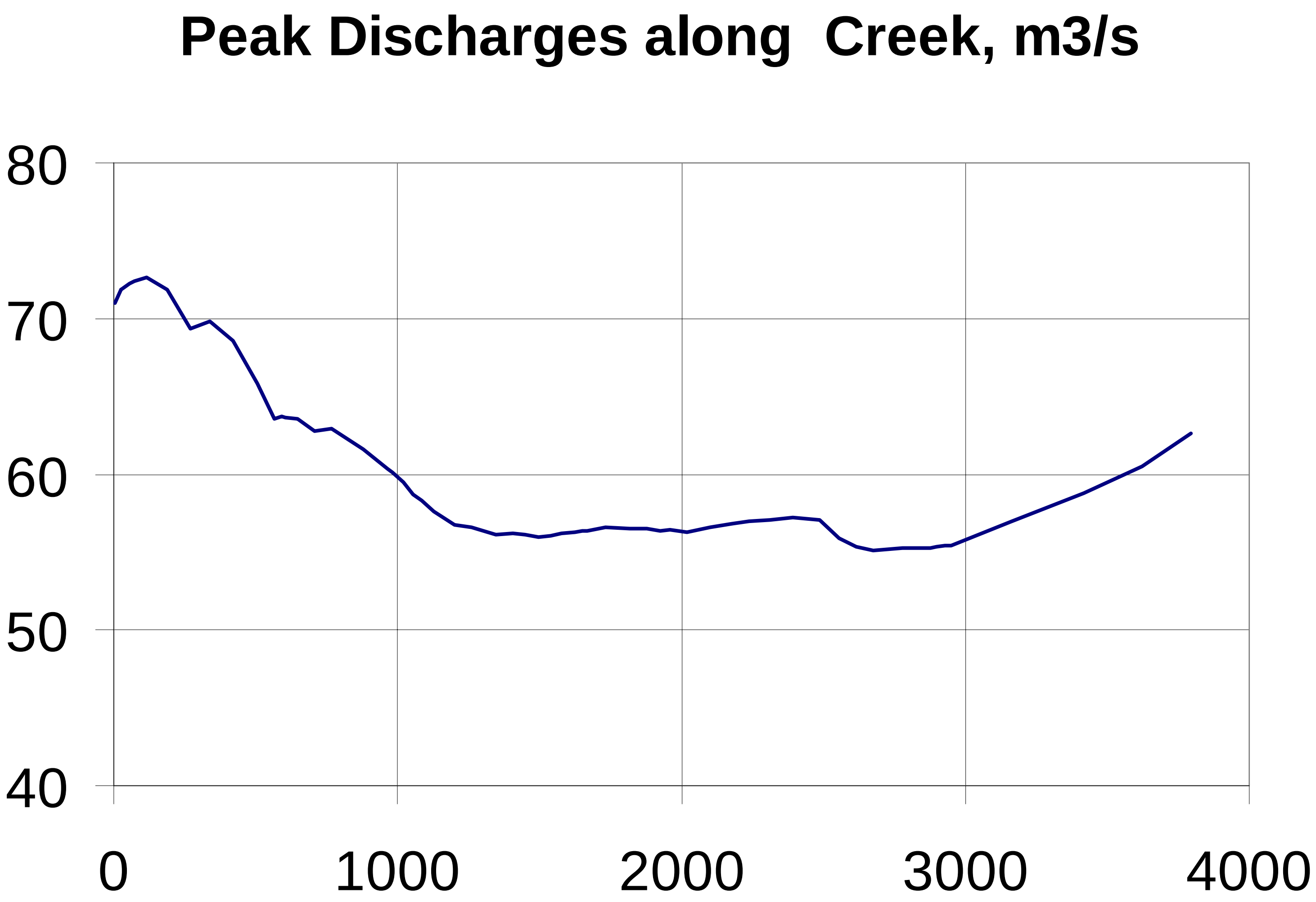


Figure 13a-13e. 100-year Storm Inundation Maps
Figure 13a.

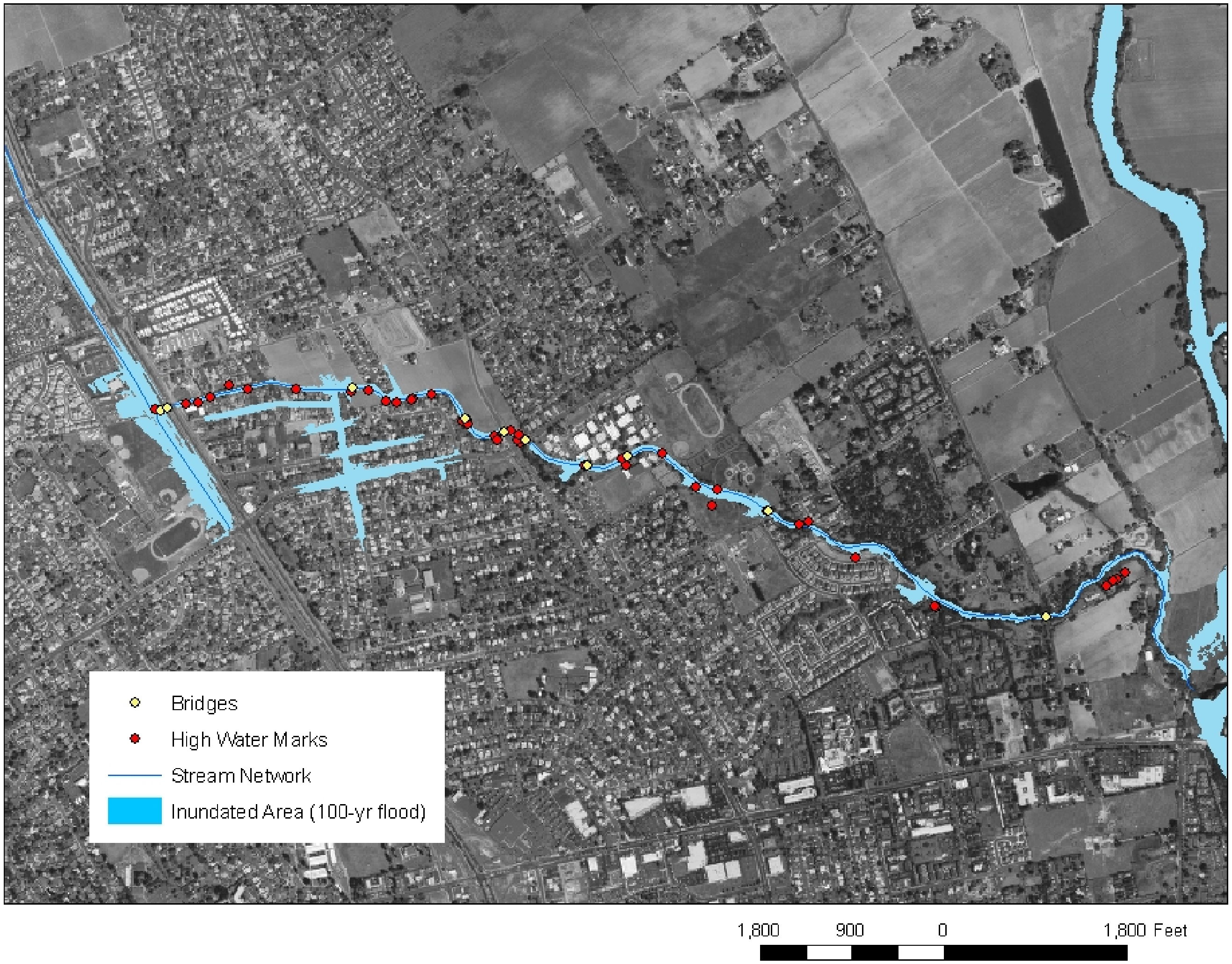


Figure 13b.


Biale Bridge
Hwy 29 Bridge
Railroad Bridge
Bridges
High Water Marks
Stream Network
Inundated Area (100-yr flood)
450
225
0
450 Feet

Figure 13c.


Bridge to Nowhere
Jefferson Bridge
Trower Bridge
HSB1
HSB2
Bridges
High Water Marks
Stream Network
Inundated Area (100-yr flood)
450
225
0
450 Feet

Figure 13d.

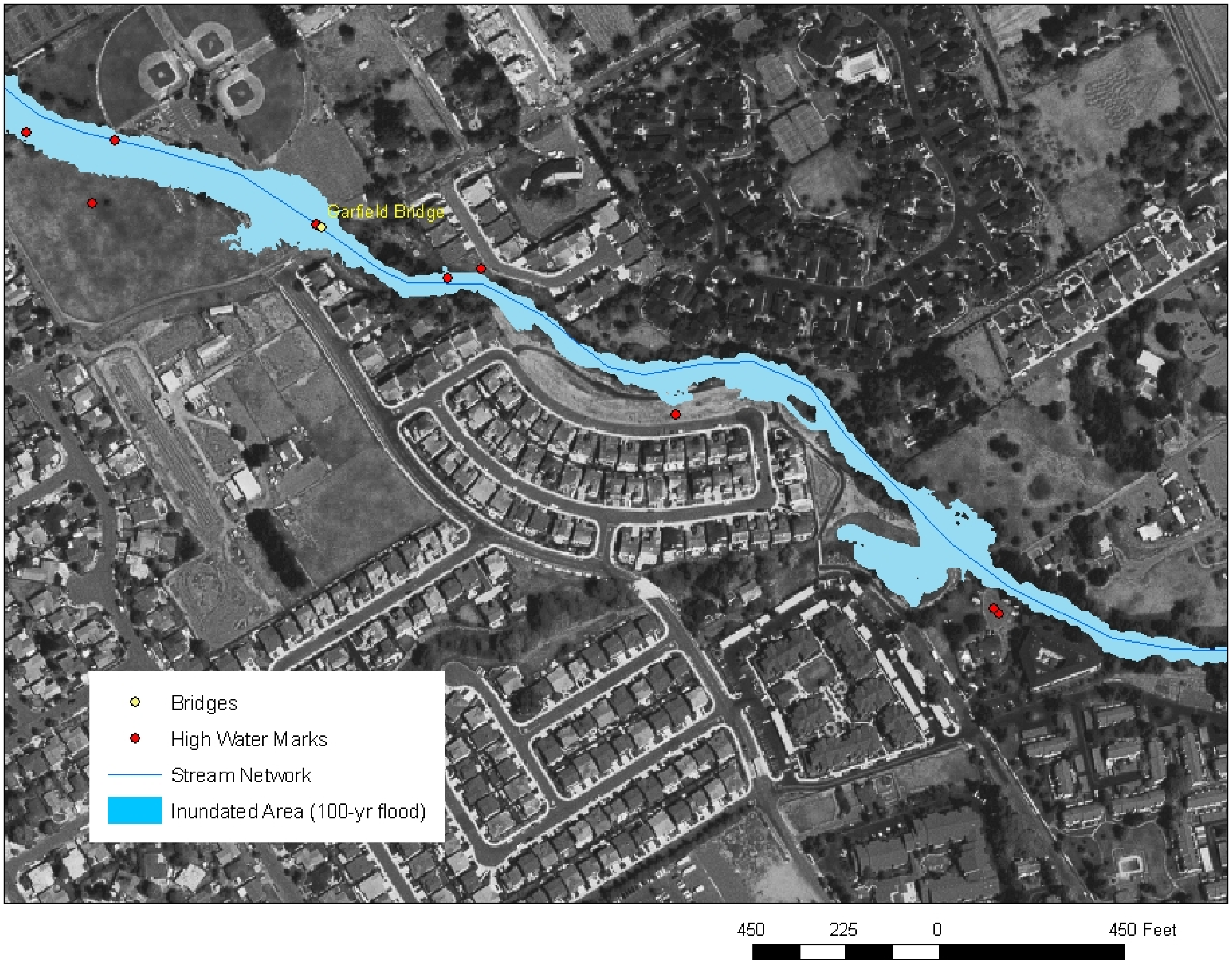

Garfield Bridge
Bridges
High Water Marks
Stream Network
Inundated Area (100-yr flood)
450
225
0
450 Feet

Figure 13e.

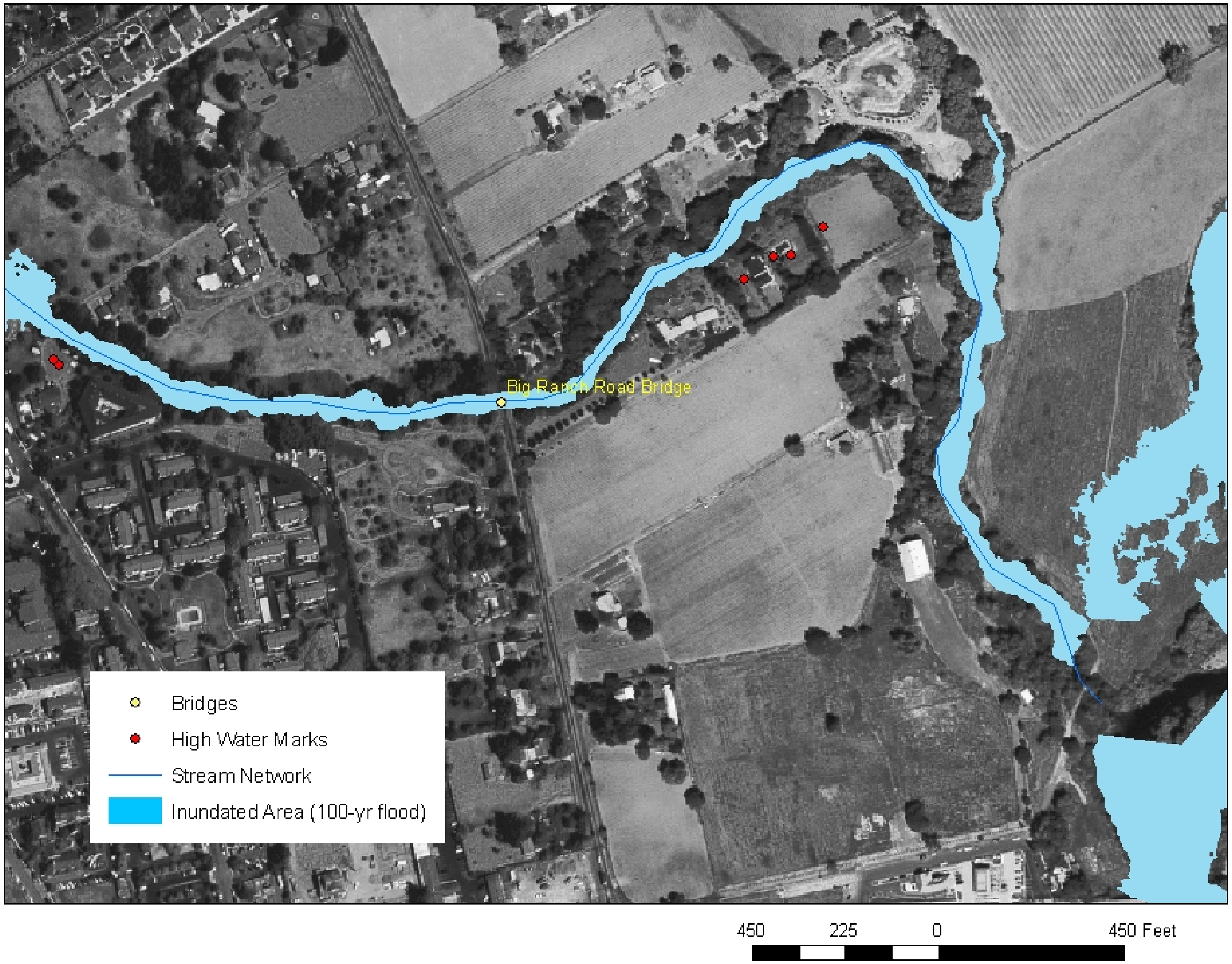

Big Ranch Road Bridge
Bridges
High Water Marks
Stream Network
Inundated Area (100-yr flood)
450
225
0
450 Feet

Figure 14. Effect of Flood Terraces on 100-year Water Level

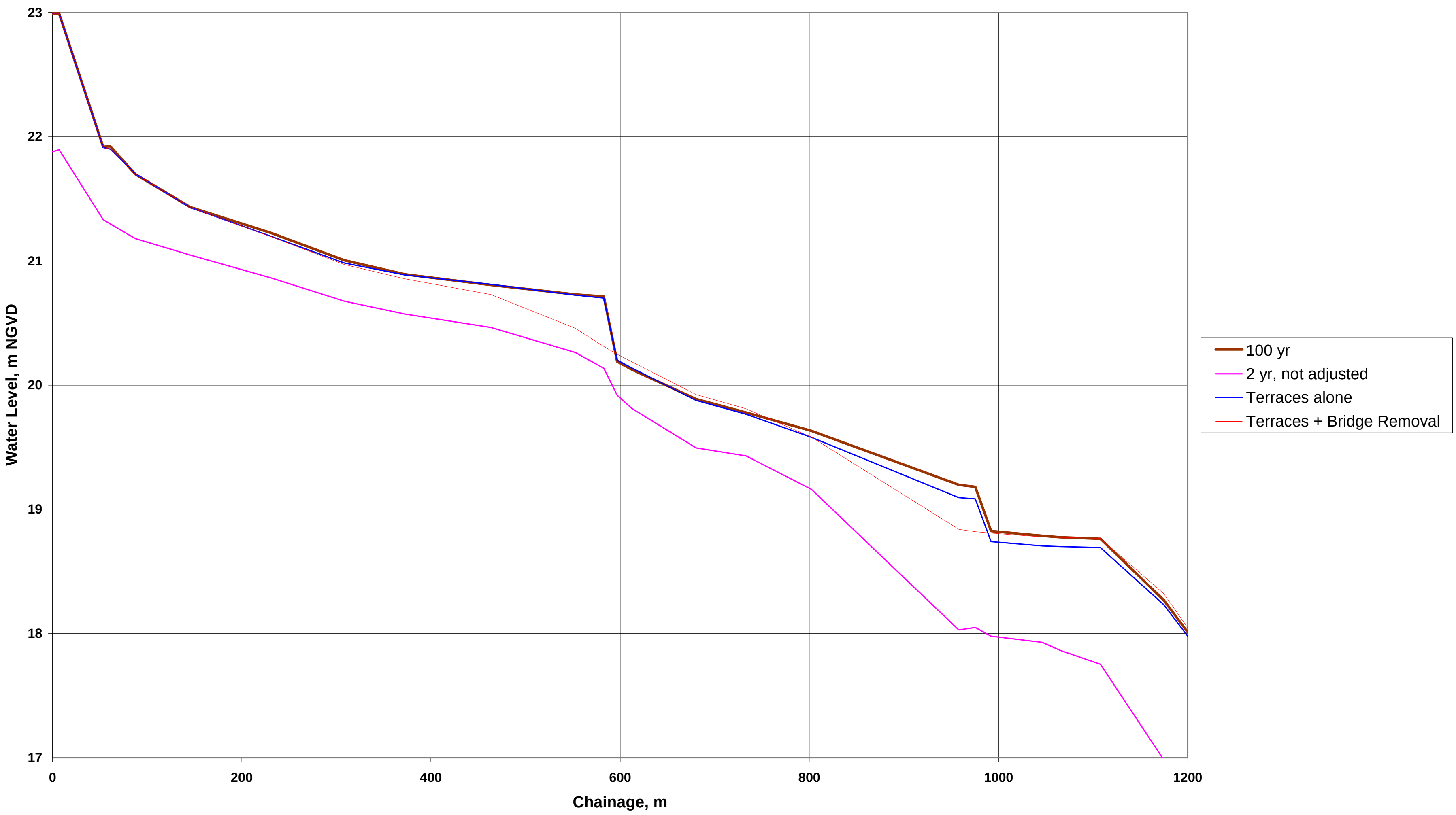
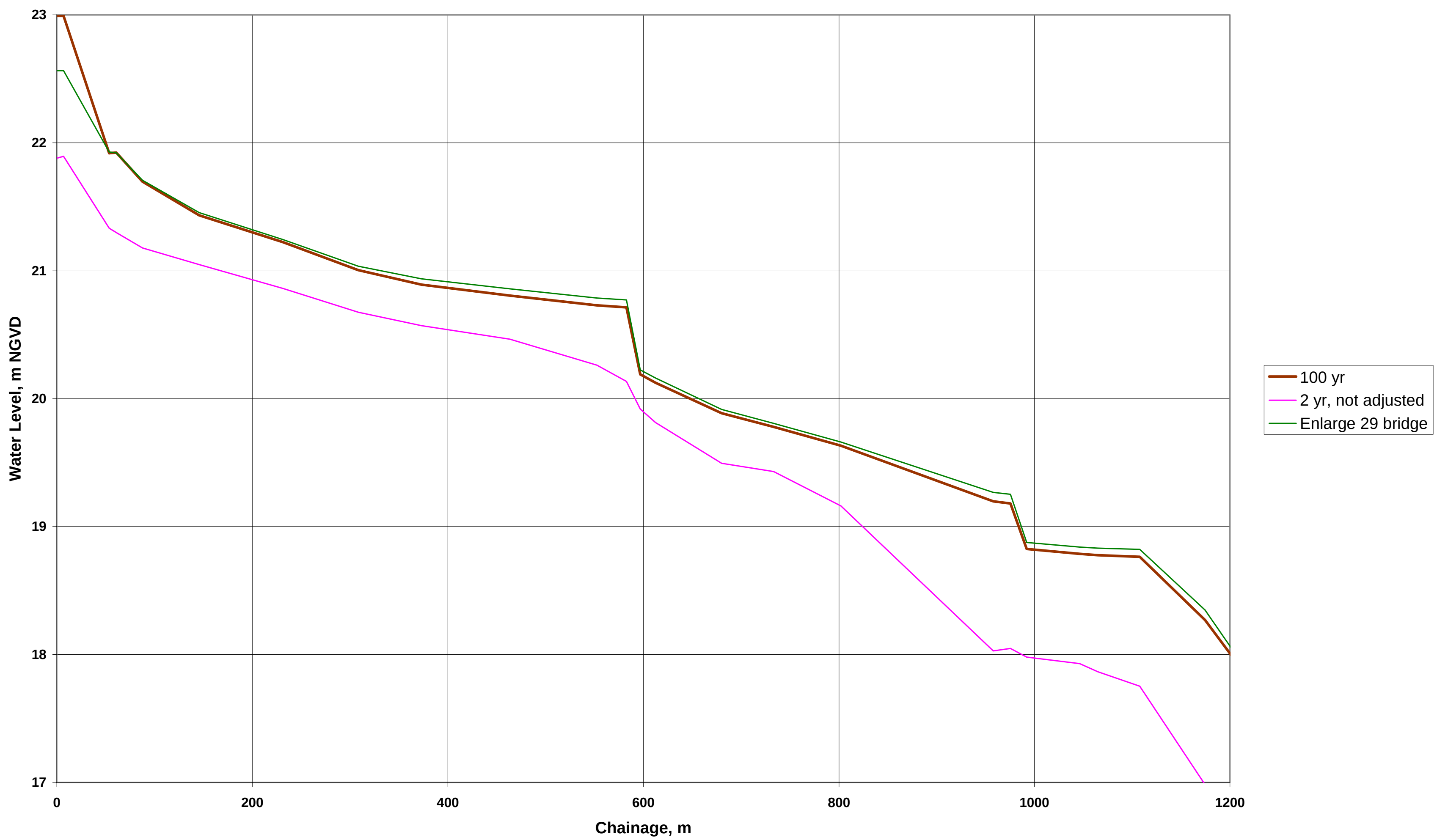


Figure 15. Effect of Highway Bridge Improvement on 100-year Water Level



PART TWO:

SALVADOR CREEK MIKE FLOOD MODEL

December 2007
Revised April 2008

PART TWO:

SALVADOR CREEK MIKE FLOOD MODEL

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Figure 20. Maximum inundation depths (ft) for the 100-year flood event for Scenario 4

Figure 21a. Upstream view of maximum extent of inundation for Scenario 4 (pink) compared with the baseline inundation extent (light blue) for the 100-year flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 4

Figure 21b. Downstream view of maximum extent of inundation for Scenario 4 (pink) compared with the baseline inundation extent (light blue) for the 100-year flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 4

Figure 22. Comparison of the maximum inundation depths for the 100-year flood event for Scenario 4 with the surveyed finished floor elevations, inundated parcels are shown in blue

PART TWO:

SALVADOR CREEK MIKE FLOOD MODEL

December 2007

I. Introduction

Part One of this report described a major improvement to earlier modeling work on Salvador Creek, which improved the representation of floodplains and made other improvements to the representation of physical reality in the model and also included an improved calibration, using data from the storm of December 31, 2005. The work described in *Part Two* makes several further improvements to that model, of which the most important is the addition of a 2-dimensional representation of the floodplain using MIKE FLOOD.

After the completion of *Part One* it became clear that the model might be further improved by adjusting the rainfall runoff output to the channel, to make it consistent with the *City of Napa Storm Drain Master Plan* (West Yost & Associates, 2001). The storm drain master plan details the specific locations of storm drain outlets in the Salvador Creek watershed, along with the associated drainage areas. This information was added to the model and the calibration was revisited. In the course of that review, RCD collected further information on the experience of local agency staff in recent floods and reconsidered the calibration more generally.

An additional question arose about the 100-year storm as modeled on Salvador Creek. Comparison with modeled results on other local creeks made it clear that the amount of surface runoff, and therefore the magnitude of the peak of this storm, depended strongly on the assumed initial condition. In *Part One*, the method used to determine the initial condition was to use the model to determine an average wet-season condition and use that as the initial condition for the 100-year storm; however, subsequent modeling on another local creek suggested that other, more conservative methods might be preferable. Various methods were identified and compared with both City standards and local practice, and an updated 100-year storm model was produced.

RCD also carried out a jamb elevation survey of finished floors and garages in areas shown as inundated by flood maps, in order to verify the flood risk. A comparison of these finished floor elevations with the maps led to the discovery that the DEM used in the model was consistently over a foot higher than the field survey. Further checking revealed an error in the elevation values in the DEM resulting from a unit error at the time of the conversion to NGVD29. When this error was corrected and the 100-year storm redone, it became clear that the resulting flooding was too complex to be represented adequately by a one-dimensional channel model. In particular, the pattern of inundation suggested that during a large magnitude flood event along Salvador Creek, split-flows may occur with streets acting as preferential flow paths. It is difficult to adequately simulate this effect with a 1-dimensional model because 1-dimensional modeling requires the modeler to define these principal flow paths prior to executing a simulation.

The decision was made to develop a MIKE FLOOD model, which joins a 2-dimensional floodplain model of the floodplain to a 1-dimensional model of the main channel. A MIKE FLOOD model of

Salvador Creek was constructed and calibrated to the flood of December 31, 2005, and the model was used to evaluate several scenarios of interest.

II. Review Model Calibration using Storm Drain Master Plan Data

A. Rainfall Runoff Model Revision

In the rainfall runoff model developed in *Part One*, the Salvador Creek watershed was divided into four subbasins, and the runoff from each was input to the creek linearly along the appropriate reach. The runoff from the North Branch subbasin, for example, was distributed evenly along the length of the North Branch of the creek. This commonly used procedure is a simplification of the *actual* distribution pattern along each creek reach, which depends on the physical layout of the storm drain system. As a check on this feature of the model, we decided to study this further, and we revised the model according to the actual storm drain layout. The Storm Drain Master Plan includes maps showing the locations of all storm drain outfalls to the creek, with annotations of the drainage areas associated with the storm drains and of the remaining areas which drain directly to the creek via surface runoff. These were all transferred to the model after the data were re-aggregated to give the totals associated with each point inflow or remaining area of distributed inflow. Three of the previous subbasins were redistributed in this manner; only the most downstream subbasin remained unchanged. The redistribution of runoff is summarized in Table 1.

Table 1. Redistribution of Runoff to Match Storm Drain Master Plan

Branch & Outfall	Chainage, m	Area, km^2		Total areas by subbasin, km^2
		Point Inflows	Distributed Inflows	
SALVO_NB 1	0	2.011		
SALVO_NB 2	551	1.888		
SALVO_NB 3	1293	3.527		
SALVO_NB 4	0 - 996		.261	7.687
SALVO_SB 1	0	1.728		
SALVO_SB 2	324	0.350		2.078
SALVO_BR 1	373	0.250		
SALVO_BR 2	552	0.086		
SALVO_BR 3	1065	0.225		
SALVO_BR 4	1140	1.443		
SALVO_BR 5	1560	0.180		
SALVO_BR 6	2450	0.647		
SALVO_BR 7	2687	0.937		
SALVO_BR 8	0 - 2958		.992	4.760
SALVO_DS	2958 - 3811		4.268	4.268
Totals		13.272	5.521	18.793

Rerunning the rainfall runoff model using the calibration of *Part One* resulted in aggregated results at Big Ranch Road that were very close to the previous results, with only minor differences that may be attributed to the slight variation in total subbasin areas.

B. Hydrodynamic Model Recalibration with Storm Drain Data

When the new rainfall runoff results were inputted to the previously calibrated hydrodynamic (HD) model, the effect on the HD results was surprisingly minor. Although we experimented with varying Manning's n, in the end no adjustments were made to the calibration parameters to model the storm of December 31, 2005, which was the main focus of the previous calibration. The new calibration is compared to the old in Table 2. The average of the absolute values of *residuals* is calculated for each calibration (a residual being defined as the difference between the average of all high water marks associated with a computational point and the peak simulated level there), as is the percent error in the modeling of the two peak discharges at Big Ranch Road on December 30-31, 2005. The new calibration shows marginal improvement in discharge but falls off slightly in matching the high water marks.

Table 2. Comparison of New Hydrodynamic Calibration with Previous Calibration

Calibration	Average of absolute values of residuals, m	Percent error in modeled Peaks	
		First peak	Second peak
Uniform runoff distribution (previous calibration)	0.257	12	7
Runoff distributed according to Storm Drain Master Plan (new calibration)	0.265	9.7	8.9

III. Review 100-year Storm Model

A. Importance of Initial Conditions

The Federal Emergency Management Agency (FEMA) recommends several methods for determining appropriate peak 100-year discharge values to use in performing hydraulic analyses for flood hazard zone delineations. If a long-term gaging station is available, a flood frequency analysis is generally the chosen option. In the absence of a long-term gaging record, empirical relationships between contributing drainage area and peak discharge are often used to infer a peak discharge for an ungaged tributary based on the peak discharge from a gaged main-stem river. Various types of rainfall-runoff models are also commonly used for the purpose of determining the 100-year discharge, and when properly calibrated, rainfall-runoff models can provide more reliable 100-year peak discharge values compared with more simple empirical methods. Some rainfall-runoff models, such as the MIKE 11 rainfall-runoff model, have the additional advantage of allowing the determination of a transient flood hydrograph as opposed to the simple steady-state value that is commonly used in FEMA analyses. Because FEMA analyses are generally simple steady-state solutions, FEMA has not developed any concrete guidelines regarding appropriate assumptions to use when applying rainfall-runoff models other than generating a list of acceptable model codes.

Various assumptions must, however, be made when applying a continuous rainfallrunoff model to determine the 100-year hydrograph; perhaps most significant is the choice of initial conditions. The antecedent moisture conditions at the onset of a large storm event may exert a strong influence on the resulting peak discharge. The most commonly used approach to determining appropriate initial conditions is to examine historical moisture conditions during past high flows. One approach is to use an average wet season condition as the initial condition, as was done in *Part One* of this study. Another approach is to use the initial conditions at the onset of past large flood events. In order to choose the most appropriate initial conditions to use in this study, we first considered City standards. Although it is the City's policy to provide 100-year flood protection to structures in the City, City standards do not address the initial condition to be used in modeling the 100-year flood, and when asked about this issue City public works staff referred to the hydrologic modeling done for the Napa River/Napa Creek flood protection project (Sam Jones, pers. comm.).

The final General Design Memorandum for this major local project reports hydrologic modeling in HEC-1 using initial infiltration loss rates of 1.0 to 1.6 in. (~25 to 41 mm) and constant infiltration loss rates of 0.11 to 0.20 in./hr (~3 to 6 mm/hr) (USACE, 1998). The MIKE 11 rainfall runoff model calculates infiltration, and examination of these results in the 100-year model created in *Part One*, using the average wet season condition, shows that the infiltration rate is uniformly less than the lower USACE value of 0.11 in. and that the total modeled infiltration is less than 0.5 in. (~ 13 mm). It appears from this that this MIKE 11 model is more "conservative" (i.e. tends to predict more runoff) than the USACE work. However, it is difficult to make explicit comparisons between such different types of models.

B. *Choice of Initial Conditions*

Although it appears that the 100-year MIKE 11 model created in *Part One* may be conservative compared to the USACE work, experience modeling other local creeks subsequent to the USACE study has suggested that a more conservative assumption might be warranted. In order to investigate this possibility, it is helpful to review the MIKE 11 rainfall runoff model as developed in *Part One*.

The state of saturation of the watershed at the beginning of a simulation, which is so important to a 24-hour design storm, is represented in the MIKE 11 rainfall runoff model by the average relative moisture in two storages, one representing the upper or surface zone (U/U_{max}) and one representing the lower or root zone (L/L_{max}) (DHI, 1992). In the work reported in *Part One*, these values were obtained by running the model for an extended period and calculating averages of both quantities throughout an extended portion of the high flow season (December 2005 through early January 2006). Figure 1 compares the 100-year 24-hour hydrograph modeled in *Part One* with two (admittedly unlikely) extremes, a totally dry condition in which both storages are at zero and a fully saturated one in which both are completely full. As the figure shows, the average wet season initial condition produces a peak approximately midway between the two extremes.

To further explore the range of possibilities, the calibrated model was run for the entire hydrologic year 2005-06 and the variation of relative moisture through the flood season was observed. Table 3 shows the average values of relative moisture for the flood season, here taken to be from December 30, 2005 through March 31, 2006. For comparison, it also shows the modeled state of saturation of the watershed at the onset of serious rain on December 30, 2005, immediately before the major New Year's Eve storm which was used to calibrate the MIKE 11 model, as well as the values used in Table 7, Part One above.

Table 3. Modeled Relative Moisture in Surface and Root Zones, Salvador Creek

	North Branch	South Branch	Big Ranch	Downstream	Average of all
Flood Season (December 30, 2005 through March 31, 2006)					
average relative moisture, L/L_{max}	0.823	0.811	0.791	0.806	0.808
average relative moisture, U/U_{max}	0.539	0.513	0.496	0.516	0.516
Values at onset of serious rain December 30, 2005					
average relative moisture, L/L_{max}	0.840	0.888	0.888	0.840	0.864
average relative moisture, U/U_{max}	0.993	0.992	0.984	0.986	0.989
Values from Part One					
average relative moisture, L/L_{max}	0.66	0.72	0.72	0.65	0.688
average relative moisture, U/U_{max}	0.60	0.58	0.57	0.60	0.588

It is interesting to note that the average relative moisture values are different from the values obtained in Part One; L/L_{max} is greater and U/U_{max} is less. Since the new values are based on the entire flood season of 2005-06, they are probably a better guide to likely initial conditions for any storm during the flood season. More striking, however, are the values at the onset of serious rain on December 30. Both L/L_{max} and U/U_{max} are greater than the flood season average values, especially U/U_{max}, which is virtually equal to 1 (fully saturated).

It is important to be "conservative" in the assumptions made for a flood study in order to avoid any underprediction of the 100-year flood risk, and at the same time it is also important not to be so overly "conservative" that the study does not provide a realistic picture of the flood risk. This dilemma is made more complicated by recent challenges to previous thinking about water management in the face of predicted changes in climate, and Milly et al. (2008) argue that the assumption inherent in water management that natural systems fluctuate within an unchanging envelope of variability is no longer valid in the face of anthropogenic-induced changes in climate. FEMA has taken note of these challenges and has begun making recommendations for modernizing their flood hazard mapping program to include the effects of future-conditions hydrology (FEMA, 2001); however, much work remains to be done in order to fully address this issue.

Climatologists generally agree that climate change in the western U.S. will result in an increase in the variability of precipitation with more extreme events and consequently higher flood risk. This means that the statistical peak 100-year flood will likely become larger in the coming decades and also points to the need to be cautious about understating flood risk in hydrologic analyses. The New Years storm is the storm of record on Salvador Creek, and thus moisture conditions prior to this event arguably give us the best picture of what the moisture conditions may be like at the onset of a future large event such as the 100-year flood. Thus, we decided to use the initial conditions at the onset of serious rain on December 30, 2005 as the initial conditions for modeling the 100-year 24-hour storm in MIKE 11, with U/U_{max} rounded to 1.0. This choice strikes a balance between the need to avoid understating flood risk and the need to be realistic and is in our opinion the most appropriate choice for this work.

IV. Salvador Jamb Survey

After the 100-year storm was rerun using the new initial condition described in the previous section, RCD carried out a survey of the elevations of front-door jambs of properties in or near the areas of inundation in the 100-year flood map in order to evaluate the potential risk of flooding to houses. We included garage floor elevations and a limited number of street and gutter shots as well, and at the request of individual homeowners we surveyed secondary entrances as well. The results of the jamb survey are shown in Table 4.

The jamb survey provided a useful test of model results; as was mentioned in section I above, it led to discovery of a datum error in the DEM used to represent floodplain topography. Since correction of that error led to construction of a new MIKE FLOOD model (to be described in the next section) and the production of a new flood map, the jamb survey does not necessarily include all properties shown as potentially threatened on the new flood map.

Table 4. Surveyed Elevations of Properties Shown as Possibly Inundated on 100-year Flood Map

Street Name	Street Number	Elevation (feet above NGVD29)		
		Main Entrance	Secondary Entrance	Garage Floor
ARCADIA	1650	63.61		62.05
BIG RANCH	2123	41.30	38.58	39.91
BRYCE	1600	65.70		64.10
BRYCE	1601	65.91		64.31
BRYCE	1607	65.99		64.37
BRYCE	1608	65.95		64.30
BRYCE	1615	66.13		64.52
BRYCE	1616	65.95		64.26
BRYCE	1623	66.42		64.71
BRYCE	1624	66.27		64.63
BRYCE	1631	66.52		64.91
BRYCE	1632	66.40		64.80

BRYCE	1639	66.65		65.01
BRYCE	1640	65.09		64.51
BRYCE	1647	66.84		65.26
BRYCE	1648	66.75		65.09
BRYCE	1655	67.18		65.55
BRYCE	1656	67.10		65.46
BRYCE	1663	67.46		65.81
BRYCE	1664	67.34		65.65
BRYCE	1671	67.71		66.06
BRYCE	1672	67.63		65.92
BRYCE	1680	67.79		66.15
DIABLO	3917	64.28		62.36
GARFIELD	73	47.24	46.53	
GLACIER	3936	64.65		63.08
GLACIER	3942	64.78		63.14
GLACIER	3950	65.01		63.36
GLACIER	3956	65.16		63.53
GLACIER	3972	65.33		63.67
GLACIER	3978	65.45		63.84
GLACIER	3984	65.54		63.87
GLACIER	3990	65.67		64.05
JEFFERSON	3908	64.32		62.58
JEFFERSON	3912	64.69		
LASSEN	3824	69.58		67.96
LASSEN	3825	69.84		68.28
LASSEN	3835	69.81		68.21
LASSEN	3836	69.38		67.77
LASSEN	3848	69.23		67.64
LASSEN	3857	67.94		67.40
LASSEN	3860	68.98		67.34
LASSEN	3867	69.29		67.77
LASSEN	3872	68.77		67.24
LASSEN	3879	69.23		67.57
LASSEN	3884	68.64		67.10
LASSEN	3889	68.79		67.21
LASSEN	3900	66.10		
LASSEN	3901	68.64		67.09
LASSEN	3909	67.14		66.55
LASSEN	3917	68.58		66.96
LASSEN	3925	67.94		66.20
LASSEN	3930	66.04		65.60
LASSEN	3958	68.40		66.13
LASSEN	3966	68.38		66.20
LASSEN	3967	68.79		67.01
LASSEN	3968	69.07		67.15
LASSEN	3969	69.60		67.66
LASSEN	3970	68.61		66.87
MOSS	100	69.03		67.51
MOSS	106	69.89		67.62
MOSS	112	70.13		68.24
MOSS	118	70.66		68.75
MOSS	124	69.75		67.96
OLYMPIA	1700	62.09		61.58
SOLANO	4075	74.25		
SOLANO	4123	76.18		
TROWER	1502	60.63		
TROWER	1506	61.07		60.43
TROWER	1512			60.67
TROWER	1518			60.75
TROWER	1530	63.55		61.95
TROWER	1531	63.63		62.03
TROWER	1540	63.68		62.10
TROWER	1543	63.91		62.29
TROWER	1580	64.17		62.59
TROWER	1583	63.92		62.29
TROWER	1600	64.59		62.93

TROWER	1601	64.26		62.59
TROWER	1625	64.32		62.67
TROWER	1642	64.60		62.89
TROWER	1651	64.42		62.71
TROWER	1682	64.77		63.20
TROWER	1700	64.94		63.40
TROWER	1701	64.72		63.11
TROWER	1724	65.26		63.59
TROWER	1725	64.87		63.39
TROWER	1753	63.73		63.17
TROWER	1756	65.38		63.81
TROWER	1775	65.39		63.75
TROWER	1780	65.74		64.05
TROWER	1787	65.60		64.00
TROWER	1800	66.05		64.48
TROWER	1801	65.75		64.10
TROWER	1808	64.45		63.83
TROWER	1819	65.94		64.32
TROWER	1822	65.95		64.39
TROWER	1825	65.99		64.40
TROWER	1828	66.39		64.79
TROWER	1839	65.09		64.62
TROWER	1842	66.42		64.95
TROWER	1845	66.83		64.98
TROWER	1848	66.64		64.99
TROWER	1859	66.12		65.22
TROWER	1862	66.82		65.09
TROWER	1865	67.12		65.46
TROWER	1868	66.87		65.30
TROWER	1879	67.12		65.62
TROWER	1882	66.95		65.24
TROWER	1893	67.33		65.75
TROWER	1900	67.75		66.15
TROWER	1904	66.87	66.92	66.71
TROWER	1912	67.29		
TROWER	1918	67.51		
TROWER	1924	67.93		
TROWER	1930	68.29		
TROWER	1942	68.44		
TROWER	2001	75.24		
TROWER	2019	76.75		75.21
TROWER	2035	75.72		75.23
TROWER	2047	77.40		75.88
TROWER	FIRE STA	74.76		74.23
UNWIN	1862	68.92		66.87
UNWIN	1875	67.28	66.08	66.15
UNWIN	1878	69.04		67.22
UNWIN	1881	68.28		66.50
UNWIN	1884	69.10		66.95
UNWIN	1887	68.60		66.94
UNWIN	1890	68.76		66.83
UNWIN	1900	68.91		67.09
UNWIN	1901	69.52		67.32
UNWIN	1904	68.95		67.55
UNWIN	1905	69.10		67.64
UNWIN	1908	69.46		67.93
UNWIN	1909	69.29		67.75
VALENCIA	1904	69.84		67.81
VALENCIA	1905	69.05		67.41
VALENCIA	1906	70.14		68.36
VALENCIA	1907	69.31		67.56
VALENCIA	1908	70.27		68.77
VALENCIA	1909	69.77		67.76
VALENCIA	1910	68.84		68.52
VALENCIA	1911	69.67		67.87
VALENCIA	1913	68.31		68.06

VALENCIA	1914	68.41		68.18
VALENCIA	1915	68.34		68.00
VALENCIA	1916	69.92		68.29
VALENCIA	1917	68.15		67.98
VINTAGE	1	59.42	59.69	
VINTAGE	2	59.05		
VINTAGE	3	58.47		
VINTAGE	4	57.80	58.03	
VINTAGE	5	57.04		
VINTAGE	6	56.71		
VINTAGE	7	55.93		
VINTAGE	8	61.78		
VINTAGE	9	61.06		
VINTAGE	10	55.43		
VINTAGE	11	55.01		
VINTAGE	12	57.46	53.85	52.91
VINTAGE	13	55.42		
VINTAGE	14	54.17		
VINTAGE	15	55.89	55.73	
VINTAGE	16	56.82		
VINTAGE	17	56.73		
VINTAGE	18	53.91		
VINTAGE	10A	55.37		
VINTAGE	11A	54.52		
WISE	1600	64.96		63.38
WISE	1601	64.95		63.47
WISE	1642	63.72		63.36
WISE	1643	65.08		63.42
WISE	1684	65.75		64.16
WISE	1685	65.29		63.62
WISE	1700	65.84		64.16
WISE	1701	65.39		63.75
WISE	1722	64.13		63.78
WISE	1725	65.70		63.88
WISE	1751	65.46		64.24
WISE	1754	66.10		64.58
WISE	1775	64.09		63.82
WISE	1786	66.20		64.71
WISE	1800	66.55		64.96
WISE	1801	65.69		64.40
WISE	1809	66.10		64.81
WISE	1817	66.45		64.86
WISE	1820	65.22		64.67
WISE	1825	66.60		65.20
WISE	1826	65.51		
WISE	1832			65.02
WISE	1839	67.20		65.35
WISE	1845	65.87		65.15
WISE	1875	67.56		65.95
WISE	1880	65.60		
WISE	1883	67.86		66.37
WISE	1884	66.74		65.20
WISE	1887	68.05		66.37
WISE	1888	66.90		65.37
WISE	1890	67.16		65.73
WISE	1892	68.34		65.83
WISE	1893	68.16		66.47
WISE	1900	67.60		66.19
WISE	1901	67.84		67.30
WISE	1902	67.25		66.81
WISE	1904	67.62		67.00
WISE	1906	68.03		
WISE	1907	70.15		68.28

V. MIKE FLOOD Model

A. Model Construction

The MIKE 11 model described in Part One of this report was used as the basis for the 1-dimensional component of the MIKE FLOOD model. The only change made to the MIKE 11 model was to clip the cross sections back so that they extend only to the top of the channel banks. This was performed because the MIKE 11 model simulates only the active channel portion of the MIKE FLOOD model domain, and the floodplain portion is simulated in 2-dimensions using MIKE 21. A polygon of the active channel was delineated by connecting the end points of the clipped MIKE 11 cross sections lines based on interpretations of the extent of the active channel derived from aerial photography and a Digital Elevation Model (DEM).

The 1.0-meter resolution DEM was aggregated to a resolution of 2.5 meters for use in the MIKE 21 model. This resolution was chosen in order to represent the floodplain topography in as great a detail as possible while keeping within the computational restrictions of the model engine. Based on previous modeling experience, MIKE 21 models should not exceed 1,000,000 active grid cells. The 2.5-meter resolution used for this model results in 728,318 active grid cells within the Salvador model domain, which represents a good balance between computational burden and a detailed representation of the floodplain topography.

The extent of the MIKE 21 domain was established based on the floodmapping performed for the stand-alone MIKE 11 modeling described in Part One of this report. The goal here was to include the full anticipated area of inundation under the 100-year event but not to include any unnecessary areas so that the resolution could be as fine as possible without exceeding the computational limits of the model engine. The area within the polygon representing the active channel was removed from the MIKE 21 domain because this area was simulated in MIKE 11: it needed to be removed from the MIKE 21 domain in order to avoid double-counting the conveyance in this area. The MIKE 21 topography and the extent of the MIKE 11 and MIKE 21 domains are shown in Figure 2.

The links between the MIKE 21 and MIKE 11 portions of the MIKE FLOOD model were established such that the MIKE 11 h-point water levels are compared to the elevations of the MIKE 21 grid cells immediately adjacent to the area represented in the MIKE 11 domain (in MIKE 11, h-points are the computational points at which water level is calculated). During the simulations, this comparison provides the basis for the exchange of flows between MIKE 11 and MIKE 21. A uniform Manning’s n of 0.033 was used to represent the floodplain resistance in the MIKE 21 model.

The inflows for the December 31, 2005 (New Years storm) and 100-year flood events that were determined previously with the rainfall-runoff models developed for the stand-alone MIKE 11 model were retained for the MIKE FLOOD model. See the preceding sections IIA, IIIA, and IIIB for a description of the rainfall-runoff modeling.

B. Model Calibration

The MIKE FLOOD model was calibrated to observed high water marks (HWMs) from the December 31, 2005 flood event. Channel roughness values were adjusted in order to minimize the differences between the observed HWMs and the model simulated water levels. Table 5 shows the final longitudinal variation of Manning’s n values used in the model. Manning’s n was not varied vertically within the cross sections as it was in the stand-alone MIKE 11 model, because the MIKE 11 portion of the MIKE FLOOD model only covers the active channel portion of the model domain and the floodplain roughness is handled by the MIKE 21 component.

Figure 3 shows a plot of the peak water levels simulated with the MIKE FLOOD model and a comparison to the final stand-alone MIKE 11 calibration for the New Years storm. In the figure, each observed high water mark is displayed at the location of the nearest water level computation point as used by the model. Water level computation points are defined in MIKE 11 at all cross section locations. The figure indicates that the two models yield a similar water surface profile for most of the modeled reach. The MIKE FLOOD calibration represents a slight improvement over the stand-alone MIKE 11 calibration, and the mean absolute value of the differences between the observed HWMs and the simulated water levels (residuals) decreased from 0.25 to 0.23 m.

Two areas exist where the modeled peak falls noticeably short of the observed high water marks for both the MIKE FLOOD and stand-alone MIKE 11 calibrations, one just downstream of the Biale Bridge (chainage 680-750 m) and one extending several hundred meters downstream of the Garfield Bridge. Although there is considerable uncertainty associated with such observations, we investigated each of these areas more closely. For the Biale area, there are two high water marks that are noticeably off the line, but review of their actual physical locations shows that the representation in the figure is misleading; one is actually at some distance from the model computation point where it is plotted in the figure, so that the residual is effectively exaggerated, and the other represents the average of two observations differing in elevation by 0.32 m, an amount comparable to the residual at this point. There is no systematic pattern here which would call the calibration into question; if anything, the uncertainty of the observed data is emphasized.

In the Garfield Bridge area, however, review of the actual locations of high water marks does indicate a pattern. The models appear to underpredict peak water levels consistently between Garfield Bridge and the Zerba Bridge. The MIKE FLOOD model represents an improvement at the HWM nearest to the Zerba Bridge but still underpredicts the peak water levels at the HWMs upstream of this. Flooding of a structure above the finished floor was reported at one residence in this reach, an older structure that is significantly lower than the surrounding properties, and high water marks indicate that the Zerba Bridge was submerged. The stand-alone MIKE 11 model did not show either effect, but the MIKE FLOOD model does show the Zerba Bridge being partially submerged, which represents an improvement over the stand-alone MIKE 11 model calibration.

The discrepancies downstream of the Garfield Bridge may be explainable by debris blockages, which are difficult or impossible to model because of their unpredictable timing. Napa County Flood District staff reported discovery of an area of blockage at around chainage 2450 m, where the Summerbrook Circle bypass rejoins the channel; and there may also have been a debris blockage downstream of Zerba Bridge (Mike Forte, pers. comm.). In our previous interpretations based only on the stand-alone MIKE 11 model, we hypothesized that in the actual event a debris blockage downstream of the Zerba Bridge may have caused water to back up to the Zerba Bridge and beyond, and that this blockage in combination with the blockage observed at around chainage 2450 m may be the source of the high water which we did not capture in the model. The MIKE FLOOD model *does* show the Zerba Bridge as being partially submerged and hits the high water marks well in that vicinity, but the other discrepancies downstream of the Garfield Bridge are not improved. It may be that the observed blockage at around chainage 2450 m alone accounts for the remaining discrepancies, although it is not possible to say for certain.

Table 5. Manning's n values for Salvador Creek, by reach

Chainage	Manning's n
0 – 200	0.03
200 – 400	0.06
400 – 450	0.08

450 – 875	0.10
875 – 925	0.075
925 – 1050	0.05
1050 – 1300	0.06
1300 – 1550	0.03
1550 – 1600	0.065
1600 – 3811	0.10

C. 100-year Flood and Scenario Descriptions

The calibrated MIKE FLOOD model was used to run the 100-year flood using the initial conditions determined in section IIIB above. As one would anticipate from our earlier experience with the stand-alone MIKE 11 model, described in Part One above, the model shows considerable street flooding and some apparent flooding of structures, which will be discussed in detail in the next section.

Several scenarios were tested with the model in order to try to relieve the flood risk to structures located near the creek. Scenario 1 involved the removal of two farm bridges, the Biale Bridge and the Bridge to Nowhere (Figure 4). This scenario was chosen because these bridge removals are likely to occur in the future and the modeling of this scenario with the stand-alone MIKE 11 model suggested that removal may lower the peak water surface profile locally without a significant downstream increase in peak water levels.

Scenario 2 retained the removal of the two farm bridges implemented under Scenario 1 and additionally created a short floodwall ~0.6 m high along the south bank of the creek in the vicinity of Lassen Street. The floodwall extends for ~25 m upstream and ~25 m downstream of Lassen Street (Figure 4). This scenario was designed to try to block the passage of water down Lassen Street and into the surrounding neighborhood. The flood wall was implemented by adjusting the geometry of the cross sections in the vicinity of Lassen Street as well as by raising the topography of a single row of cells in the MIKE 21 domain by 0.6 m in this area.

The idea for the flood wall came from several trial runs using the stand-alone MIKE 11 model, together with the observation that the main avenue for street flooding from the creek during the New Years Eve storm appeared to be a low spot on the right bank near the north end of Lassen Street.The height and length of the flood wall were chosen to remove the apparent low spot without creating extensive interior drainage issues.

Scenario 3 retained the removal of the two farm bridges and construction of the floodwall implemented under Scenario 2 and additionally created a flood terrace on the north side of the creek within the Biale vineyard. The terrace was implemented by uniformly lowering the portion of the MIKE 21 domain representing the terrace area by ~0.8 m, to a level that would be wetted when creek flow exceeds a value of about 12 m^3/s (about 400 ft^3/s). This level is considerably lower than the level of the two-year storm, which is often considered the optimum level for flood terraces. Additionally the elevations of the north bank of the MIKE 11 cross sections along this reach were lowered to match the new adjacent terrace levels. The terrace is ~25 m wide and ~265 m long (Figure 4).

The width and depth of the terrace were determined on the basis of trial runs using the stand-alone MIKE 11 model, taking into account the physical size of the parcel. The scenario work described in Part One above suggested that any terraces would have to be considerably deeper than the level of the 2-year storm, if they were to contribute significant flood conveyance. The flow level at which the terrace comes into use is approximately one third of the 2-year discharge on the creek. A wider terrace did not seem to offer significantly greater benefit and would have greatly reduced the agricultural value of the parcel, because at the new depth the terrace would be wetted too often to be farmed. An additional left bank terrace, on the next parcel downstream, was considered but did not appear to have any benefit in the Lassen Street area, while potentially worsening the situation around the Jefferson and Trower bridges.

Scenario 4 implemented a detention basin west of the highway designed to capture and retain a portion of the flow moving down both the north and south branches of Salvador Creek alongside the highway and to reduce the flow moving under the highway and entering the main channel of Salvador Creek. As with Scenario 3, this feature was modeled as a hypothetical possibility, and the detention pond was implemented by uniformly lowering the MIKE 21 topography by 1.22 m (4 ft) within the footprint of the detention pond (Figure 4). Two culverts were added to the MIKE 11 model to connect the north and south branches with the detention pond and allow flow under Solano Avenue, the roadway bordering the highway. The culverts were implemented by adding two short branches with closed cross sections to the MIKE 11 model and were located with the tops of the cross sections ~0.8 m below the existing top of bank in each channel. The cross sections were closed rectangular sections 1.83 m (6 ft) wide by 0.91 m (3 ft) tall.

Both Scenarios 3 and 4 were modeled purely in the spirit of exploring possibilities; we assume that any actual terrace or detention basin work would happen only with landowner agreement. In fact, the land owners on the north side of the channel have expressed reluctance to consider giving up any land.

D. Results and Discussion

Figure 5 shows the modeled maximum inundation extent from the December 31, 2005 flood event as well as the simulated hydrograph at the Big Ranch Road gaging station. It is important to note that these inundation maps represent direct model output from the MIKE 21 simulation and no external flood mapping routine is required as is needed when producing flood inundation maps from a stand-alone MIKE 11 simulation. Use of direct model output from MIKE 21 greatly reduces the uncertainty associated with the inundation maps, because it eliminates the uncertainty that arises when a 1-dimensional model solution is displayed on a 2-dimensional map via a flood mapping routine.

Also important to note is that the maps are produced by taking the maximum water depth during the simulation for each model grid cell and compiling them together. Thus, at no single point in the simulation did the inundation pattern look like these maps, rather this shows the “worst-case” inundation during the simulation for each grid cell. This is the most commonly used approach to generating inundation maps from transient model results and is the closest approximation to the more simplified approach commonly used by FEMA where a steady-state solution is used. A steady-state solution does not take into account the variability in timing of inundation throughout the model domain, and thus the resulting inundation maps reflect the largest inundation depths expected at any point during the event, because the peak discharge in a steady-state solution is essentially occurring throughout the model area at the same time.

The modeled maximum water level inundation map for the December 31, 2005 event was compared to the finished floor elevation survey data described in section IV, in order to determine which structures were predicted to be flooded above the finished floor in the simulation. This comparison is shown in Figures 6a and 6b, where the grid file of maximum simulated water level was compared to a grid produced by assigning each surveyed entrance elevation to its respective parcel extent. This comparison showed 14 occurrences of predicted structure flooding, including one parcel west of the highway, two parcels in the neighborhood east of Lassen Street and south of the creek, and two parcels along the creek in the downstream portion of the model, as well as 9 buildings at the high school. The model unambiguously predicts flooding of the high school buildings, but interpreting whether it is predicting flooding above the finished floors of structures on the other five parcels depends on the exact location of the structures on the parcels. Relatively few structures were reported as flooded in the actual storm; in the modeled area, only one building at the high school and one residence in the downstream area were reported as flooded. Thus the model results compare fairly well with the reports of flooding, while overpredicting the extent of flooding to a certain degree.

It should be mentioned that the model underpredicts structural flooding in one place, the area downstream of Garfield Bridge. Flooding of one finished floor was reported here, but the model does not reproduce it; this is consistent with the failure of the model to match the observed high water in this area as discussed in section VB above.

Figure 7 shows the predicted maximum inundation extent during the 100-year flood event as well as the simulated hydrograph at the Big Ranch Road gaging station, and Figure 8 shows the peak water levels along the creek and a comparison with the New Years Eve peak water levels (both the simulated New Years Eve levels and the observed high water marks). The comparison between the predicted peak 100-year water levels and the surveyed finished floor elevations is shown in Figures 9a and 9b. The inundated areas were compared with the building footprints as shown in the aerial photo and if all or a portion of the inundated area intersected the building footprint, the parcel was coded as flooded. The figures indicate that during the 100-year flood, a total of 14 structures outside the high school campus are predicted to experience flooding above the finished floor elevation, as well as an additional 9 structures at the high school. It is important to note that not all of the structures potentially at risk of flooding were included in the jamb survey, so additional structures besides those depicted here may be at risk during this event.

Figure 10 shows the predicted maximum inundation depths under Scenario 1 (bridge removal) during the 100-year flood event. Figure 11 shows a comparison of the predicted maximum inundation extent under Scenario 1 with the baseline inundation extent for the upstream portion of the model domain. The areas shown in light blue are those where the inundation extent has decreased as a result of the scenario modifications. This comparison indicates that removal of the two farm bridges results in a small decrease in inundation extent north of the creek in the vicinity of Jefferson Street. No significant changes in inundation extent occurred in the downstream portion of the model domain. Figure 12 shows the peak water surface profile for Scenario 1 compared with the baseline condition in the vicinity of the bridges that were removed. For the most part, the differences are very small (< 0.05 m) except in the vicinity of the Bridge to Nowhere, where the water surface was lowered by ~0.25 m. Comparison of the peak water levels with the surveyed finished floor elevations indicates that the scenario resulted in the same structures becoming flooded as were shown as flooded under the baseline condition (Figures 9a and 9b).

Figure 13 shows the predicted maximum inundation depths under Scenario 2 (bridge removal plus floodwall) during the 100-year flood event. Figure 14 shows a comparison of the predicted maximum inundation extent under Scenario 2 with the baseline inundation extent for the upstream portion of the model domain. This comparison indicates that removal of the two farm bridges in combination with the floodwall construction results in a slightly greater decrease in inundation extent north of the creek in the vicinity of Jefferson Street than was observed under Scenario 1. A slight decrease in inundation extent south of the creek in the vicinity of Jefferson Street was also observed. No significant changes in inundation extent occurred in the downstream portion of the model domain. The maximum water surface profile is very similar to the profile for Scenario 1 where for the most part, the differences relative to the baseline are very small (< 0.05 m) except in the vicinity of the Bridge to Nowhere, where the water surface was lowered by ~0.25 m (Figure 12). Comparison of the peak water levels with the surveyed finished floor elevations indicated that the scenario resulted in the same structures predicted to flood as were shown under the baseline condition (Figures 9a and 9b).

Figure 15 shows the predicted maximum inundation depths under Scenario 3 (bridge removal plus floodwall plus terrace) during the 100-year flood event. Figures 16a and 16b show a comparison of the predicted maximum inundation extent under Scenario 3 with the baseline inundation extent for the upstream and downstream portions of the model domain. This comparison indicates that removal of the two farm bridges in combination with the floodwall construction and the terracing results in a decrease in inundation extent throughout the upstream portions of the model domain east of the highway as well as in the downstream portions of the model domain to the south of the creek. The maximum water surface profile decreased by ~0.1 m in the vicinity of the terrace and by ~0.3 m in the vicinity of the Bridge to Nowhere relative to the baseline condition. Additionally, the water surface decreased by ~0.05 m throughout the downstream portion of the model (chainage 1750 to the confluence with the Napa River). Everywhere else, the differences were very small (< 0.05 m) (Figures 17a – 17d). Comparison of the peak water levels with the surveyed finished floor elevations indicates that the scenario resulted in 5 fewer structures predicted to flood relative to the baseline, 4 within the Lassen Street neighborhood and 1 at the high school (Figure 18). The downstream reach is not shown because the predicted inundated structures along this reach are the same structures shown as inundated under the baseline condition in Figure 9b.

Figure 19 shows a comparison of the hydrographs just downstream of the highway crossing for Scenario 4 (detention west of the highway) and the baseline condition. The figure indicates that the simulated detention pond was successful at reducing the peak flow that enters the main channel of Salvador Creek by ~5 m^3/s (~175 ft^3/s). The peak is slightly delayed relative to the baseline condition and the recession occurs somewhat more slowly as some of the water that enters the detention pond is fed back into the channel following the peak. Figure 20 shows the predicted maximum inundation depths under Scenario 4 (detention west of the highway) during the 100-year flood event. Figures 21a and 21b show a comparison of the predicted maximum inundation extent under Scenario 4 with the baseline inundation extent for the upstream and downstream portions of the model domain. This comparison indicates that inclusion of a detention pond west of the highway would result in a decrease in inundation extent throughout the upstream and downstream portions of the model domain, which is similar in magnitude to that observed under Scenario 3. The only location where the inundation extent was predicted to increase was in the detention pond footprint itself. The maximum water surface profile decreased by ~0.05 to 0.15 m throughout the modeled reach (Figures 17a – 17d). Comparison of the peak water levels with the surveyed finished floor elevations indicates that the scenario resulted in 4 fewer structures predicted to flood relative to the baseline, 2 within the Lassen Street neighborhood, 1 west of the highway, and 1 at the high school (Figure 22). The downstream reach is not shown because the predicted inundated structures along this reach are the same structures shown as inundated under the baseline condition in Figure 9b.

VI. Conclusion

A. Modeling Conclusions

The MIKE FLOOD model represents a significant improvement to the stand-alone MIKE 11 model and provides a more detailed picture of the flow paths and extent of inundation associated with the 100-year flood event along Salvador Creek. This more detailed picture, represented in Figure 7, indicates more extensive areas of flooding concern than those identified in Part One above, and examination of the finished floor elevations obtained for some properties in the basin in the jamb survey indicates that some flooding of structures exceeding the finished floor elevation may be experienced in the 100-year storm.

The modeled scenarios provide some valuable insight into potential means of reducing the flood risk to structures within the basin. Simulation of the removal of the two farm bridges suggests that bridge removal does reduce peak water levels locally upstream of the bridges without a significant increase in water levels downstream although the reduced water levels do not result in a significant decrease in the extent of inundation. The tentative conclusion from Part One, namely that removal of the bridges might increase flooding downstream, is not borne out by the MIKE FLOOD model. The conclusion that can be drawn from this analysis is that removing the bridges is warranted if this is desirable for reasons other than reducing flood risk, but that it is unlikely to significantly reduce flood risk in the basin. Of course, removal of bridges does have the desirable feature that it removes a possible location for debris blockages.

Construction of a floodwall in the vicinity of Lassen Street does not appear to be worthwhile. Testing it with the improved 2-dimensional floodplain model of MIKE Flood, we are not able to corroborate our earlier impression that a modest barrier at this low point might be an effective flood defense. Simulation of this scenario in conjunction with the bridge removal suggests that floodwall construction reduces peak water levels slightly with little impact to downstream water levels, but that any improvement in inundation extent or depth is minimal.

Implementation of a large terrace along the north bank of the channel in the vicinity of the Biale vineyard does appear to result in a significant reduction in peak water levels and the extent and depths of floodplain inundation. The reductions in peak water levels and inundation occur not just in the vicinity of the terrace but throughout the modeled reach of the creek and no increased peak water levels were observed. The terrace scenario was modeled in conjunction with the bridge removal and floodwall construction so differentiating between the effects of these various activities in somewhat difficult. That being said, the greater part of the reductions in peak water levels, other than the local effects near the Bridge to Nowhere, are likely attributable to the terrace construction. Comparison of the peak inundation levels to the finished floor elevations suggests that the water level decreases simulated under the terrace scenario are significant and would decrease the number of flooded structures during the 100-year flood event.

Construction of a detention pond west of the highway also appears to result in a significant reduction in peak water levels and in the extent and depth of floodplain inundation. The reductions occur throughout the modeled reach of the creek. Comparison of the peak inundation levels to the finished floor elevations suggests that the water level decreases simulated under this scenario are significant and would decrease the number of flooded structures during the 100-year flood event. Interestingly, some of the structures where the flood risk was reduced under the detention pond scenario are different from those where the flood risk was reduced under the terracing scenario. Although this situation was not simulated, implementation of the terracing in addition to the detention pond would likely be the most effective means of reducing the flood risk along Salvador Creek among the options considered in this analysis.

B. Other Concerns on Salvador Creek

At the end of Part One, we noted a number of features of the hydrology of Salvador Creek that reflect the unique character of this urban stream, which remain important and are restated here for convenience:

- Much of the watershed is urban, and the extensive roof and pavement surfaces make for more and quicker runoff. The most recent development has been accompanied by flood attenuation structures, but there are relatively few of these and they are sized to be effective only in the 100-year storm.
- The creek channel between the highway and Jefferson was relocated in the 1960’s and the public easement has limited width.

- The creek experiences significant attenuation of flood peaks (especially design floods) beginning relatively high in the watershed, at the beginning of the main channel just west of the highway. This means that the modeled 100-year peak discharge actually decreases as one moves downstream from the highway. This smoothing of flood peaks, which is primarily caused by bridge constrictions, contradicts the common assumption that peak discharge increases steadily in the downstream direction.
- The level of the two-year flood is surprisingly high. Residents report that the creek is up around the top of bank in every big storm, and the model bears this out. To the extent that this is due to high discharge, it means that shear stress at the toe of bank is often high and that bank instability will likely be a problem.
- According to the calibrated model, roughness values are very high in some reaches of the creek.

Because the two-year flood is so high and the riparian width available for Flood District maintenance activities is so narrow, the creek can be expected to continue to have bank stability problems. For this reason, managing the creek to reduce the elevated roughness we found in some reaches of the creek should be done with great care, since reducing roughness has the potential to increase shear stress.

C. Recommendations

We recommend that the Flood District and/or the City, as appropriate, carry out the actions which follow. They appear in an order corresponding to the exposition above, which is not intended to suggest any priority.

1. Consider requesting the Federal Emergency Management Agency (FEMA) to create a flood map of Salvador Creek. If this step is not taken, extend the jamb survey carried out under this project to include all properties within the revised area of inundation as modeled with MIKE FLOOD, in order to get a more complete picture of the structures with 100-year flood risk.

2. Explore the availability of land and other costs associated with the detention pond and flood terrace scenarios.

3. Continue to maintain creek banks with bioengineering methods. Manage the roughness in the creek so as to maintain but not increase the existing roughness where bank stability is a problem. Elsewhere, consider managing the creek to reduce high roughness values.

4. Study the feasibility of modifying detention structures. If appropriate, retrofit attenuation structures installed with recent development, so that the two- to ten-year floods are attenuated as well as higher-return-interval events.

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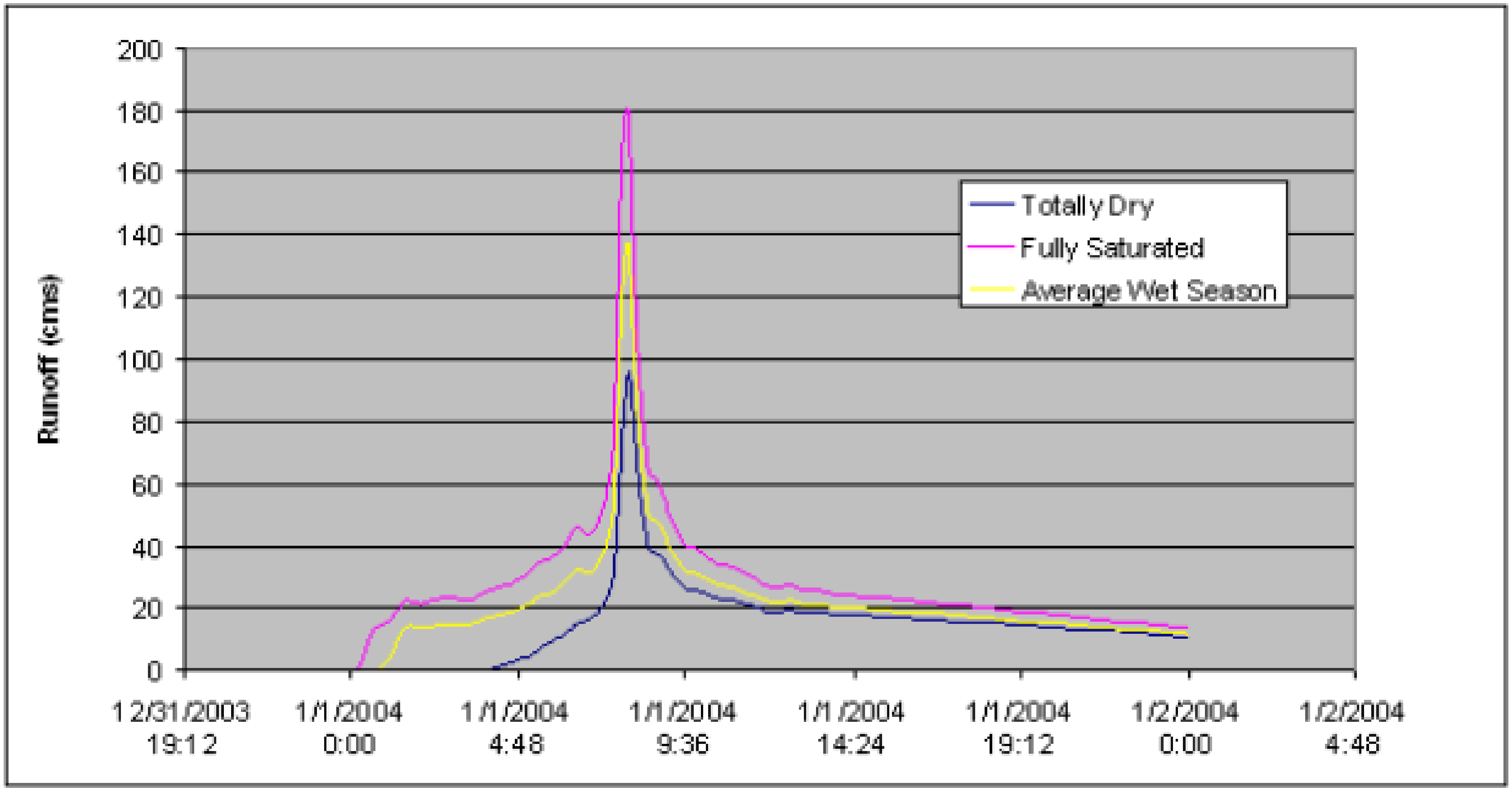

Figure 1. Effect of Initial Conditions on Runoff in MIKE 11

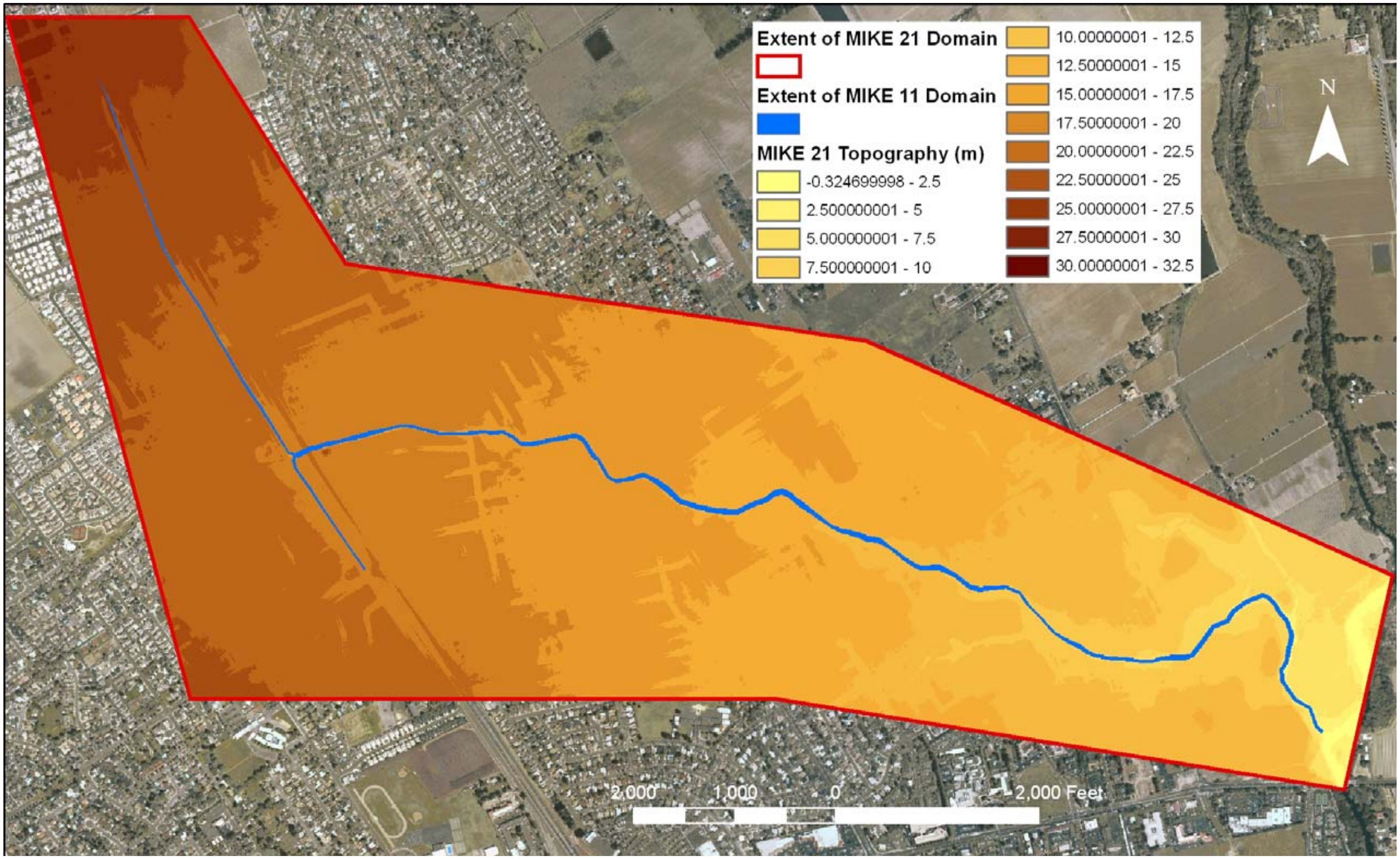

Figure 2. MIKE 21 topography and extent of the MIKE 11 and MIKE 21 domains

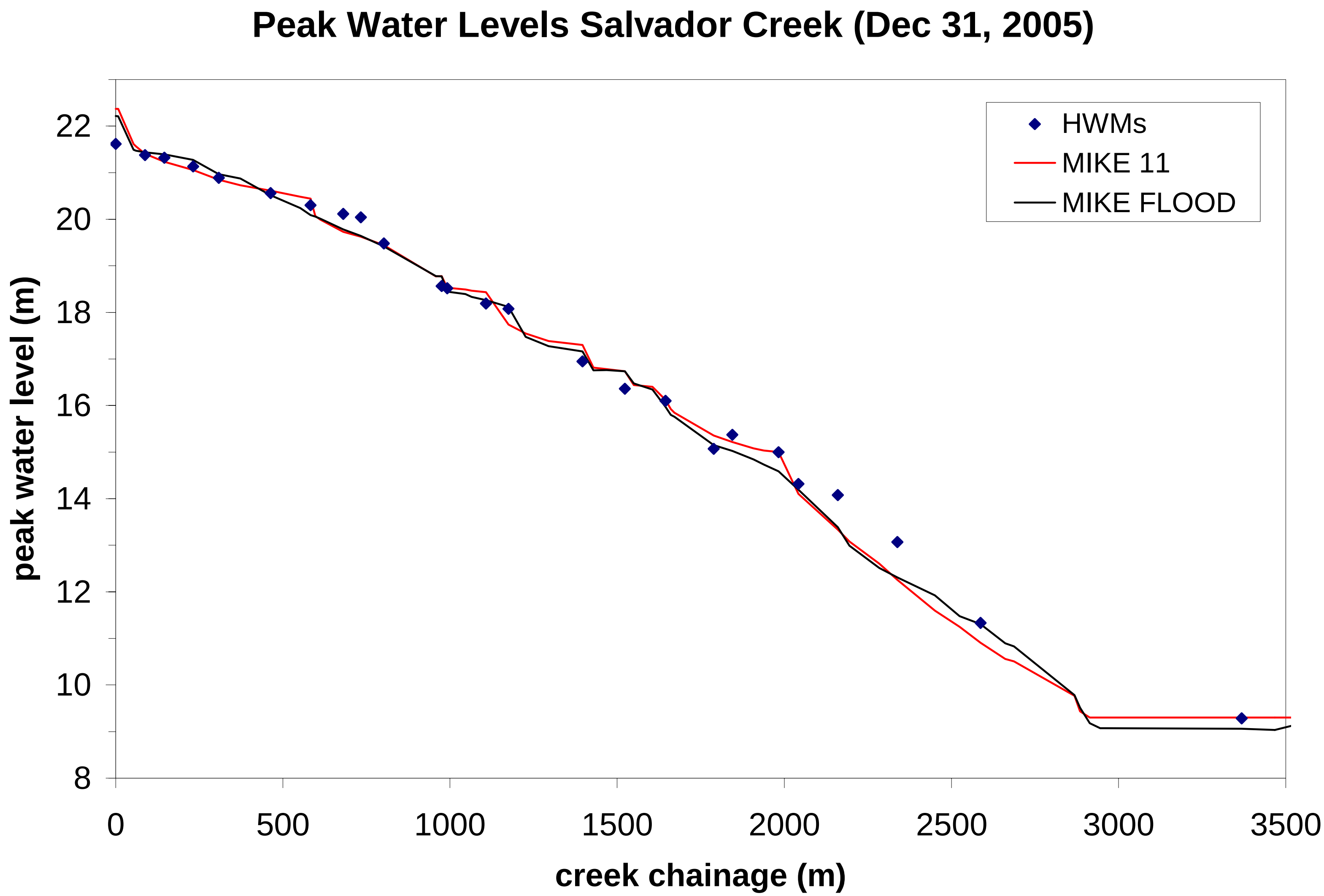

Figure 3. Peak water level comparison between measured high water marks (HWMs), stand-alone MIKE 11 simulated water levels and MIKE FLOOD simulated water levels

Figure 4. Scenario elements including bridges that were removed, the extent of the floodwall near Lassen Street, the extent of the terrace, and the location and extent of the detention pond west of the highway

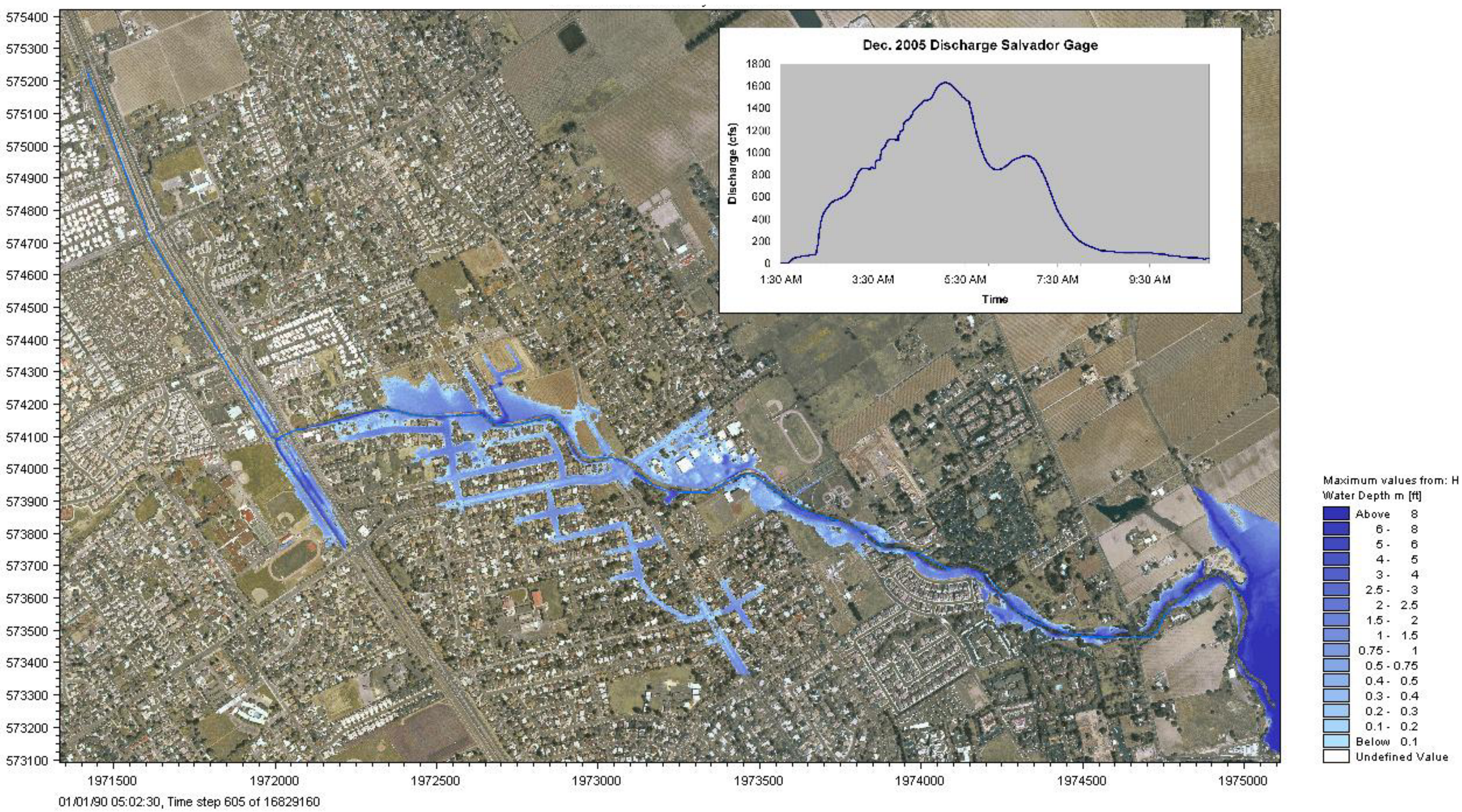

Figure 5. Maximum inundation depths (m) and hydrograph at the Big Ranch Road gage for the December 31, 2005 flood event

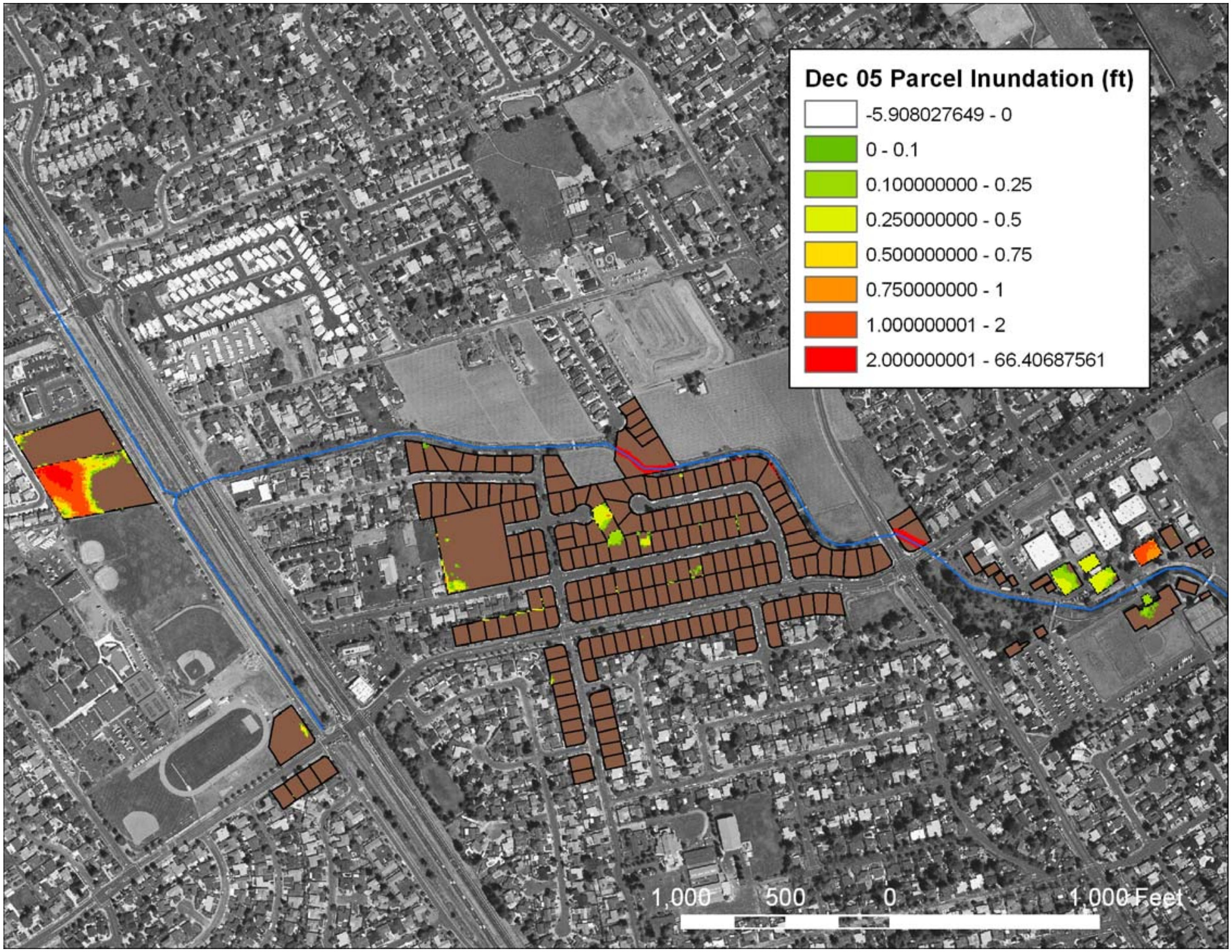

Figure 6a. Comparison of the maximum inundation depths for the December 31, 2005 flood event with the surveyed finished floor elevations showing parcel inundation depths in feet (upstream view)

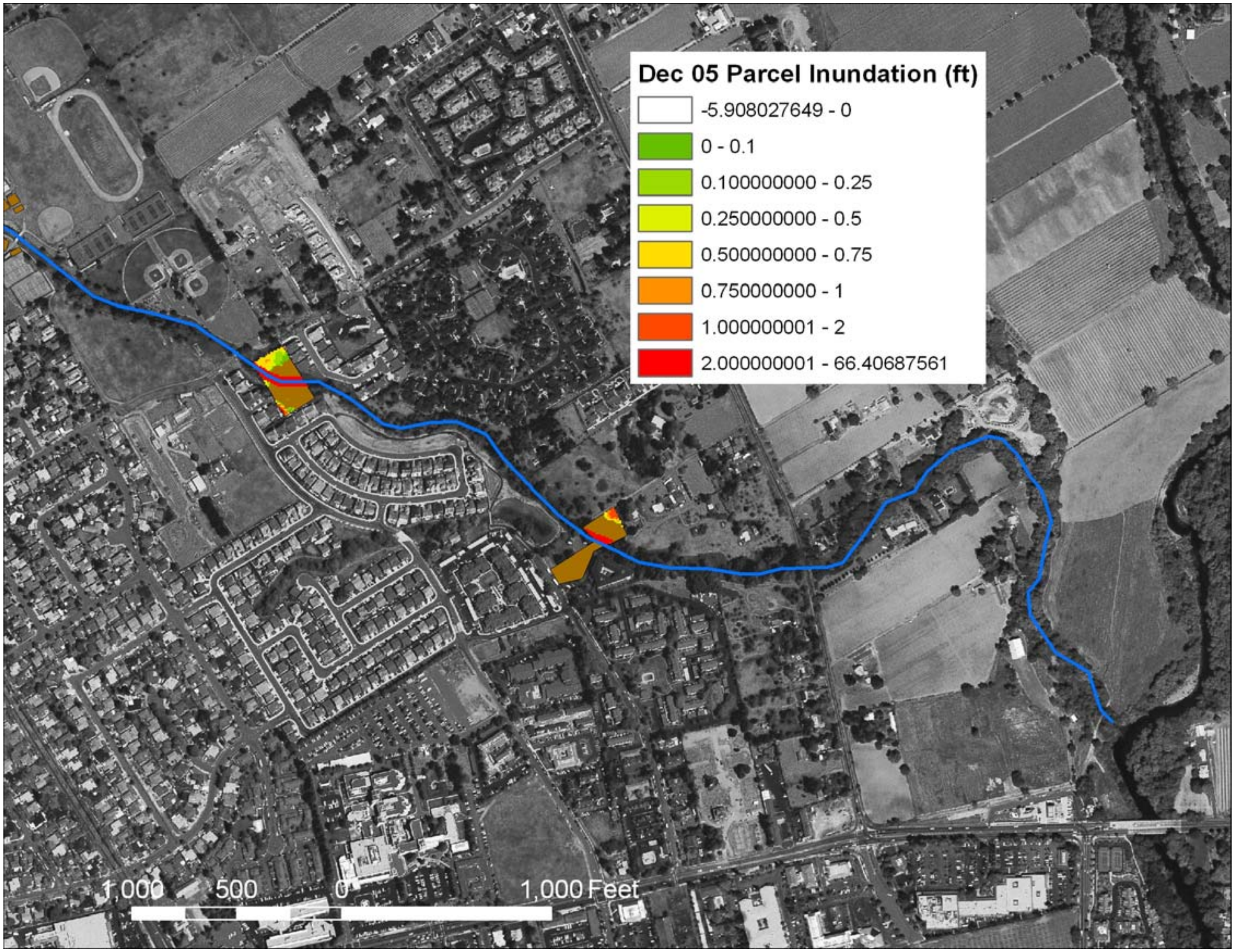

Figure 6b. Comparison of the maximum inundation depths for the December 31, 2005 flood event with the surveyed finished floor elevations showing parcel inundation depths in feet (downstream view)

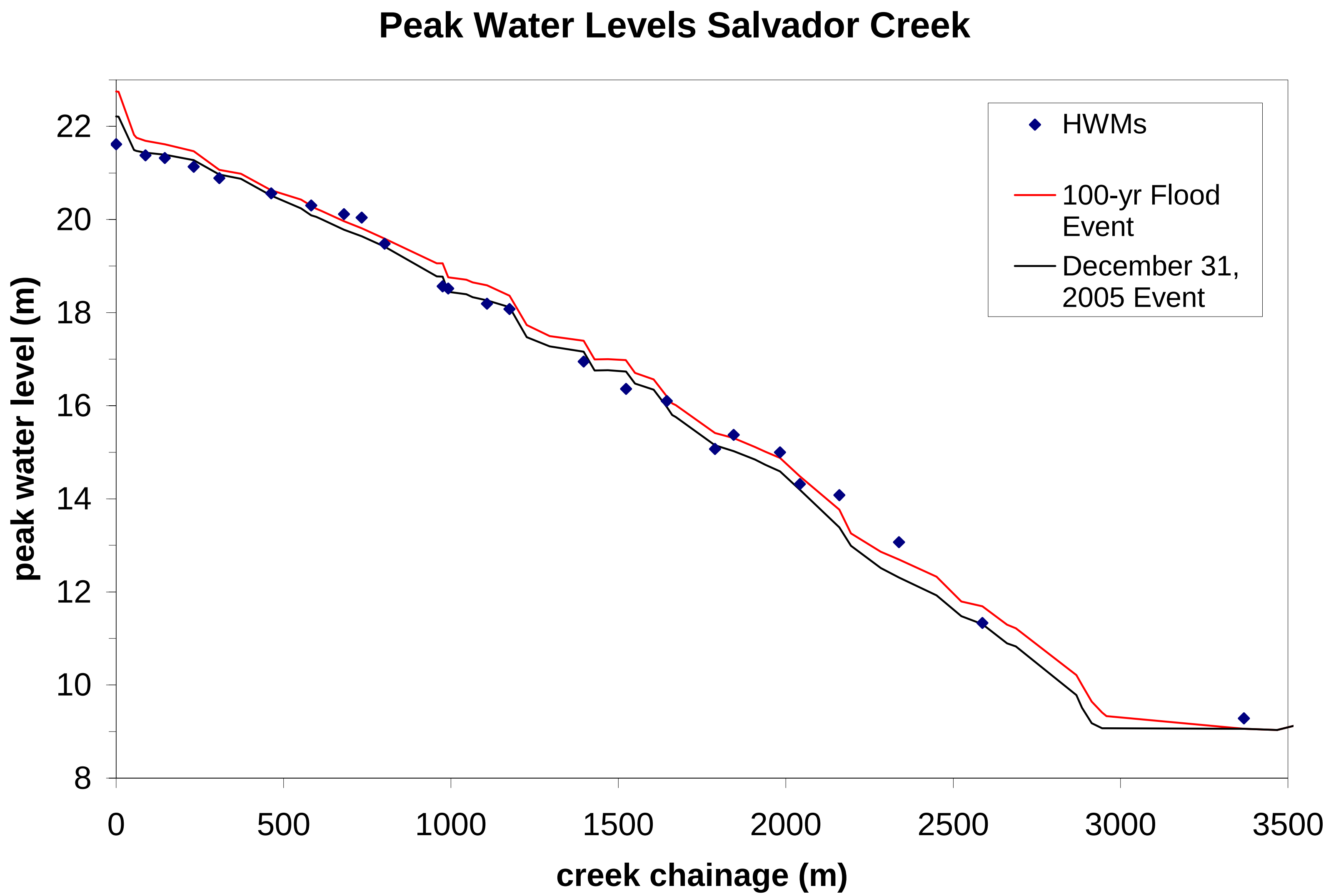

Figure 8. Peak water levels during the 100-yr flood event and comparison with peak water levels during the December 31, 2005 event

Figure 9a. Comparison of the maximum inundation depths for the 100-yr flood event for the baseline condition with the surveyed finished floor elevations, inundated parcels are shown in blue (upstream view)

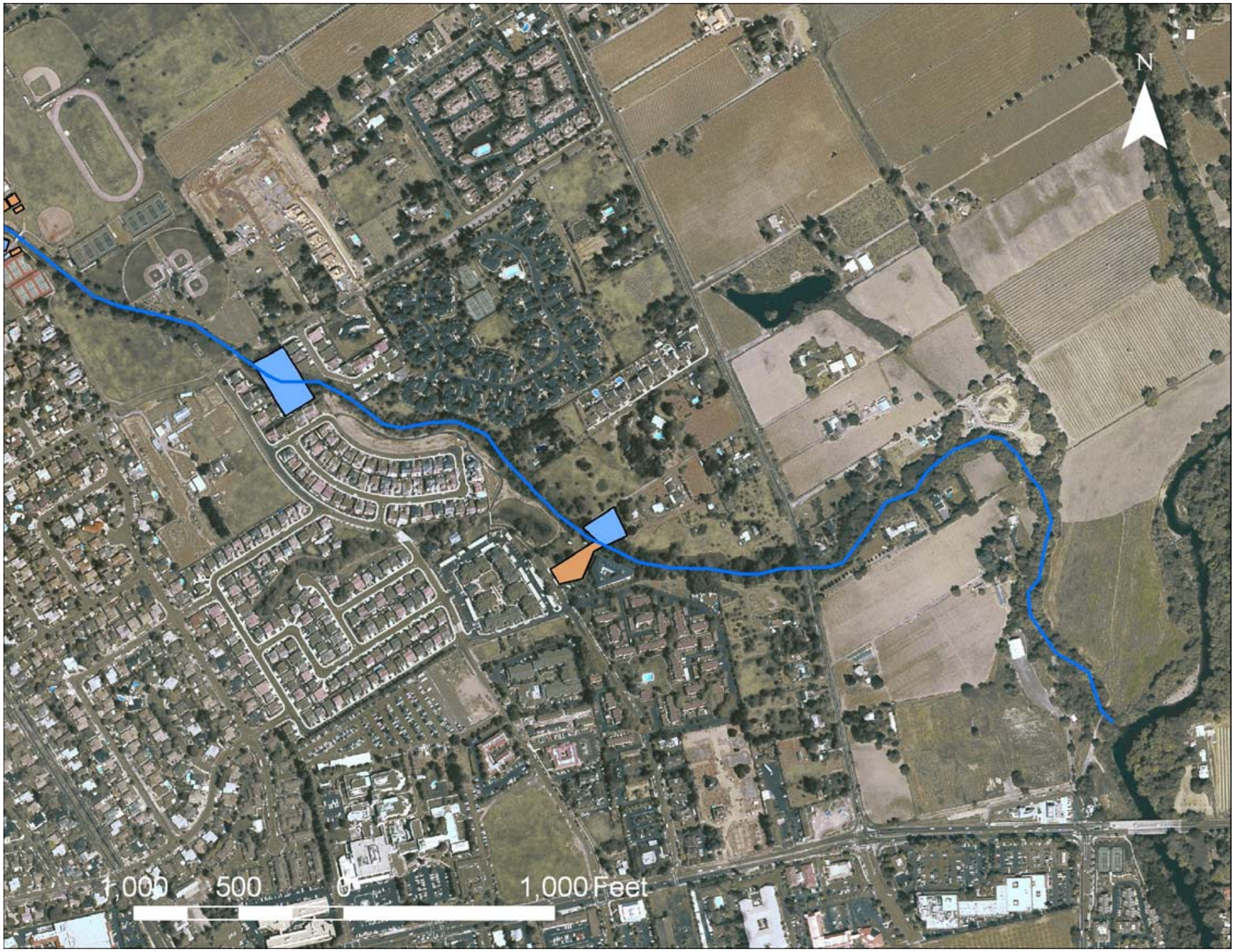

Figure 9b. Comparison of the maximum inundation depths for the 100-yr flood event for the baseline condition with the surveyed finished floor elevations, inundated parcels are shown in blue (downstream view)

Figure 11. Maximum extent of inundation for Scenario 1 (pink) compared with the baseline inundation extent (light blue) for the 100-yr flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 1

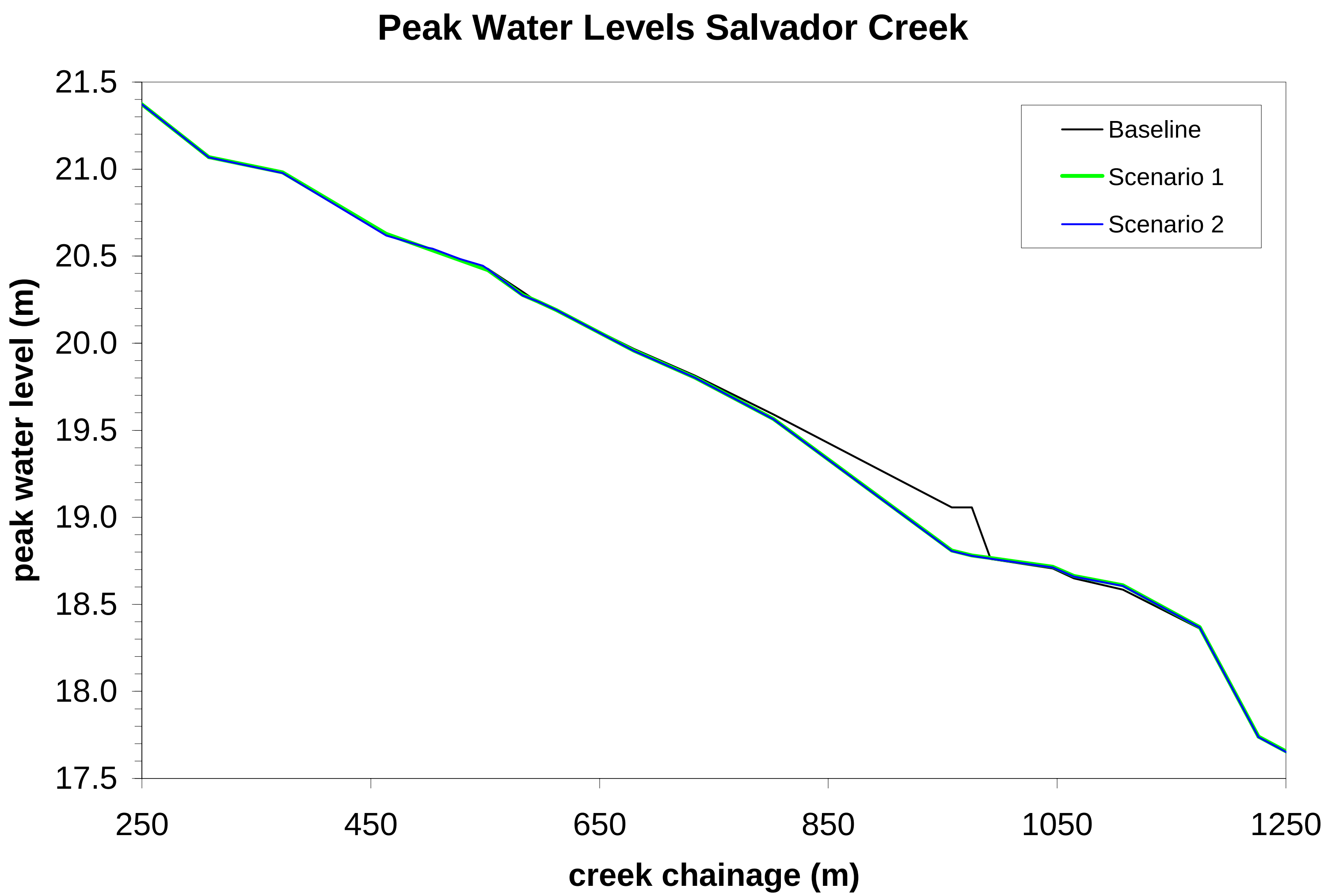

Figure 12. Peak water levels during the 100-yr flood event for the baseline condition and for Scenarios 1-2 in the vicinity of the removed bridges

Figure 14. Maximum extent of inundation for Scenario 2 (pink) compared with the baseline inundation extent (light blue) for the 100-yr flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 2

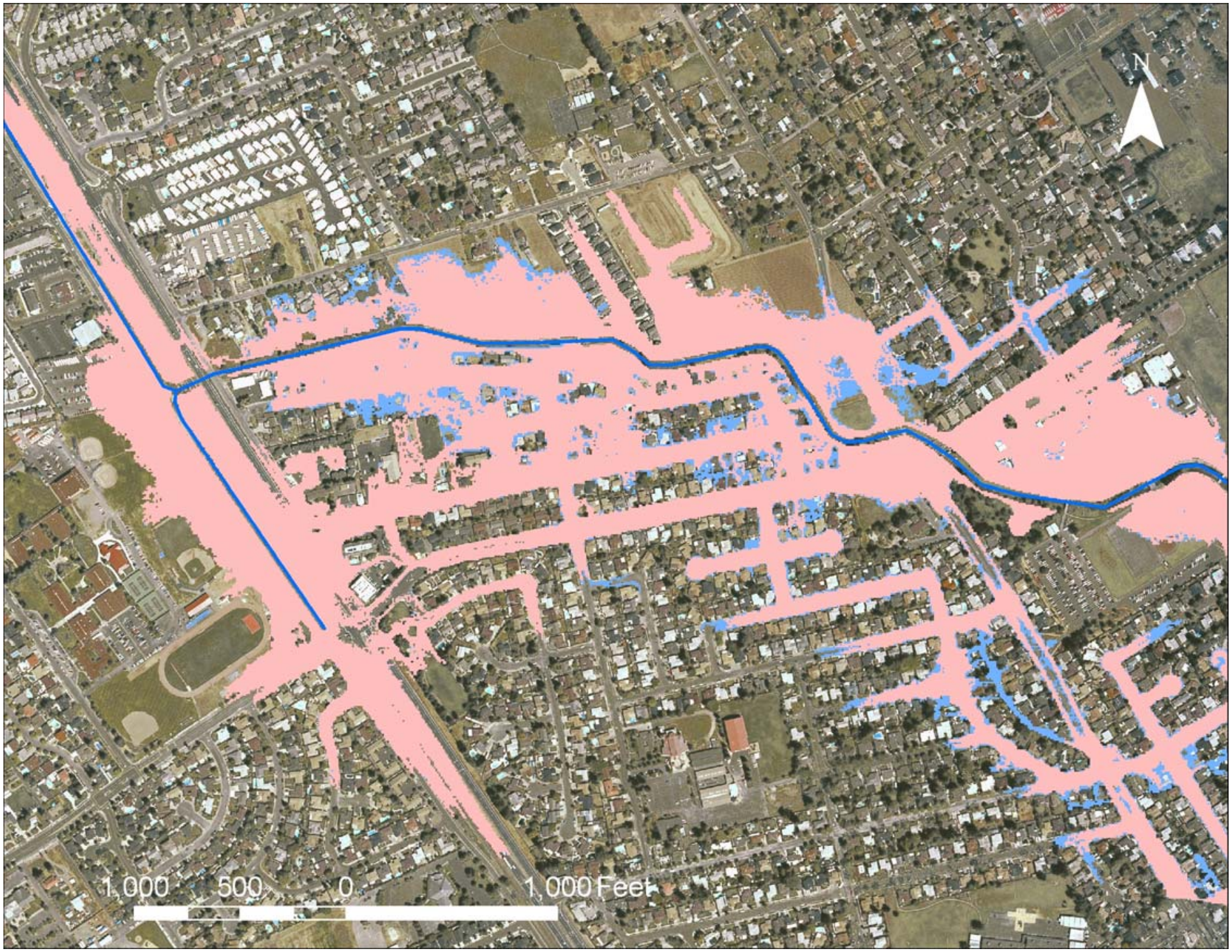

Figure 16a. Upstream view of maximum extent of inundation for Scenario 3 (pink) compared with the baseline inundation extent (light blue) for the 100-yr flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 3

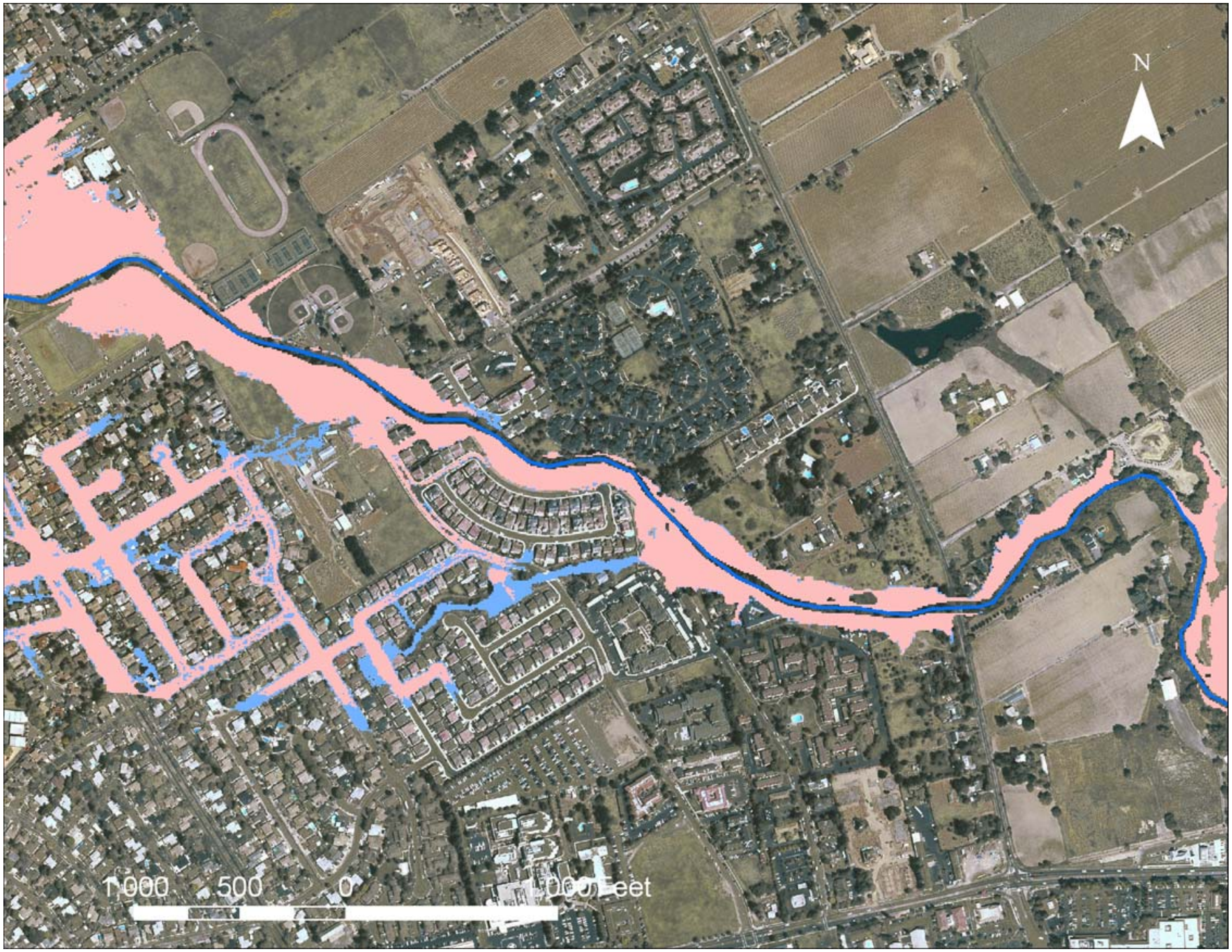

Figure 16b. Downstream view of maximum extent of inundation for Scenario 3 (pink) compared with the baseline inundation extent (light blue) for the 100-yr flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 3

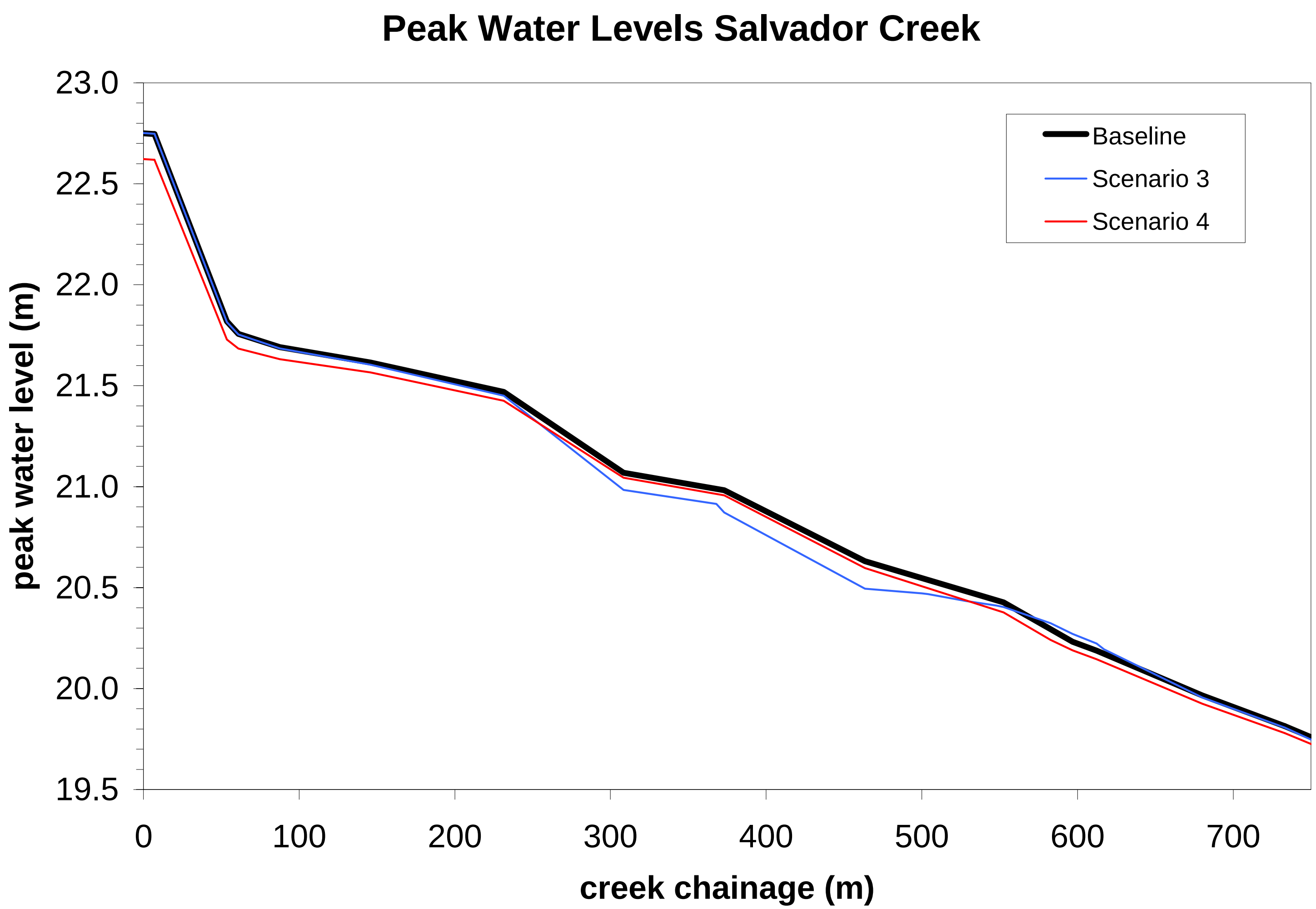

Figure 17a. Peak water levels during the 100-yr flood event for the baseline condition and for Scenarios 3-4 (chainage 0 – 750 m)

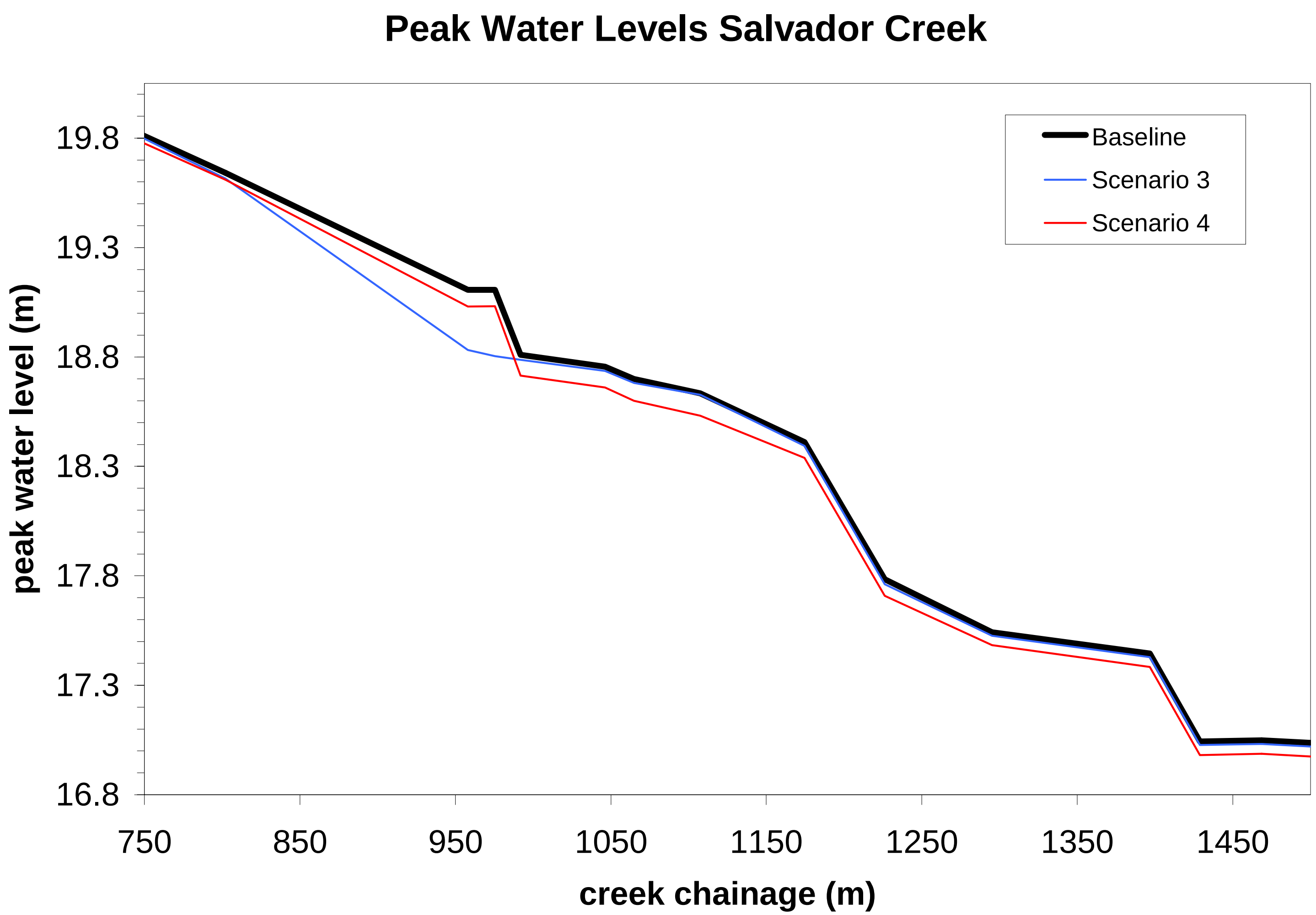

Figure 17b. Peak water levels during the 100-yr flood event for the baseline condition and for Scenarios 3-4 (chainage 750 – 1500 m)

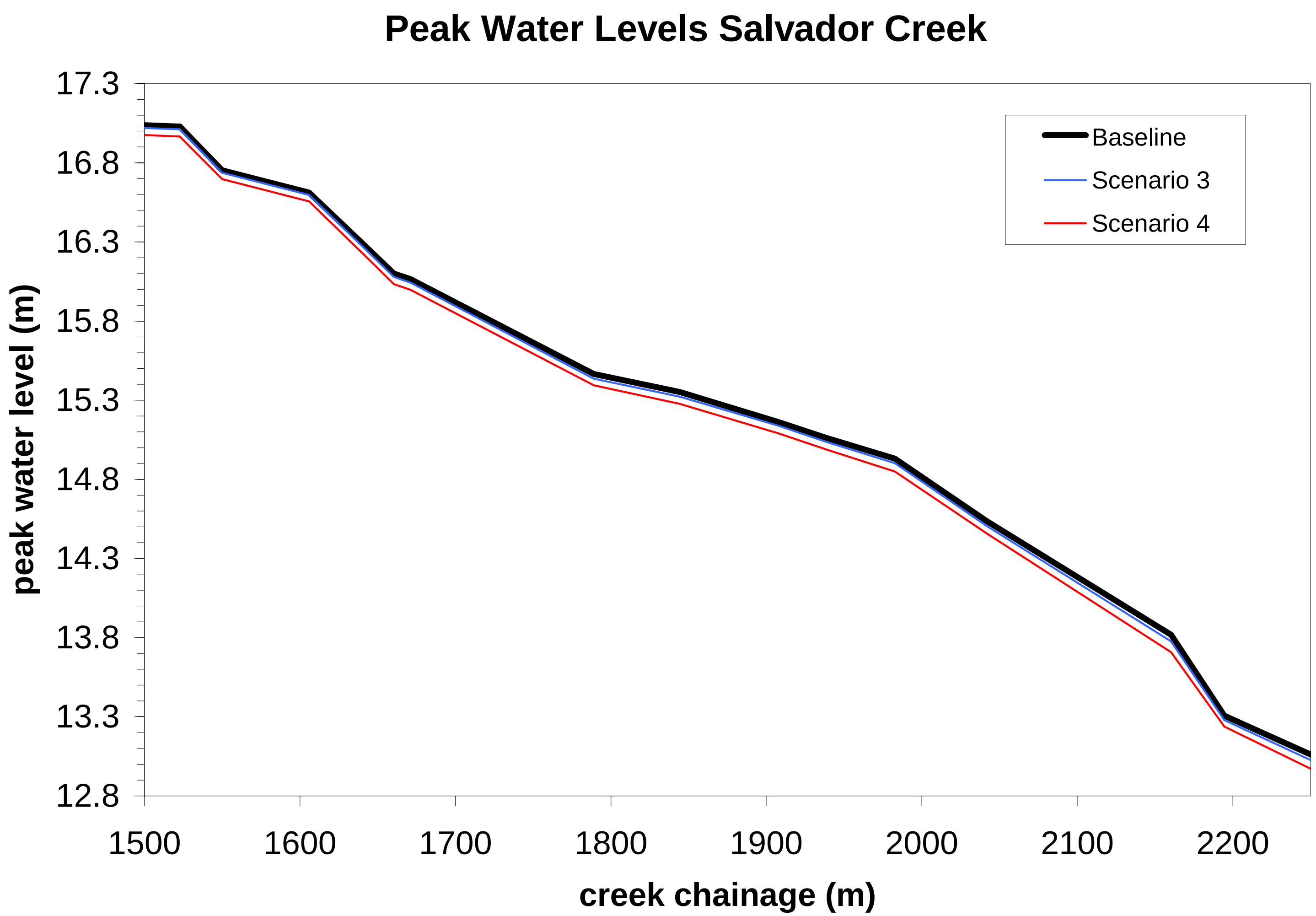

Figure 17c. Peak water levels during the 100-yr flood event for the baseline condition and for Scenarios 3-4 (chainage 1500 – 2250 m)

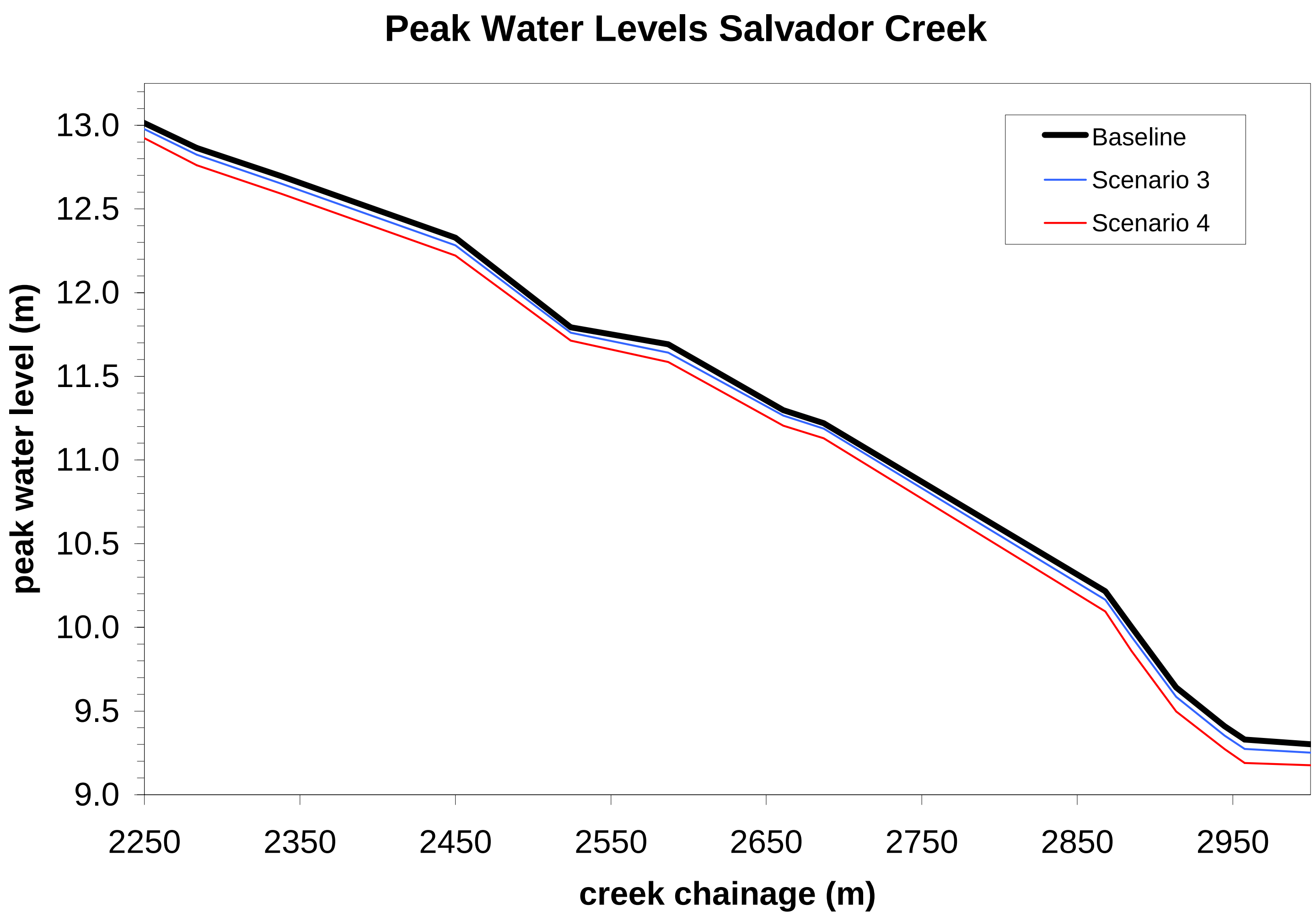

Figure 17d. Peak water levels during the 100-yr flood event for the baseline condition and for Scenarios 3-4 (chainage 2250 – 3000 m)

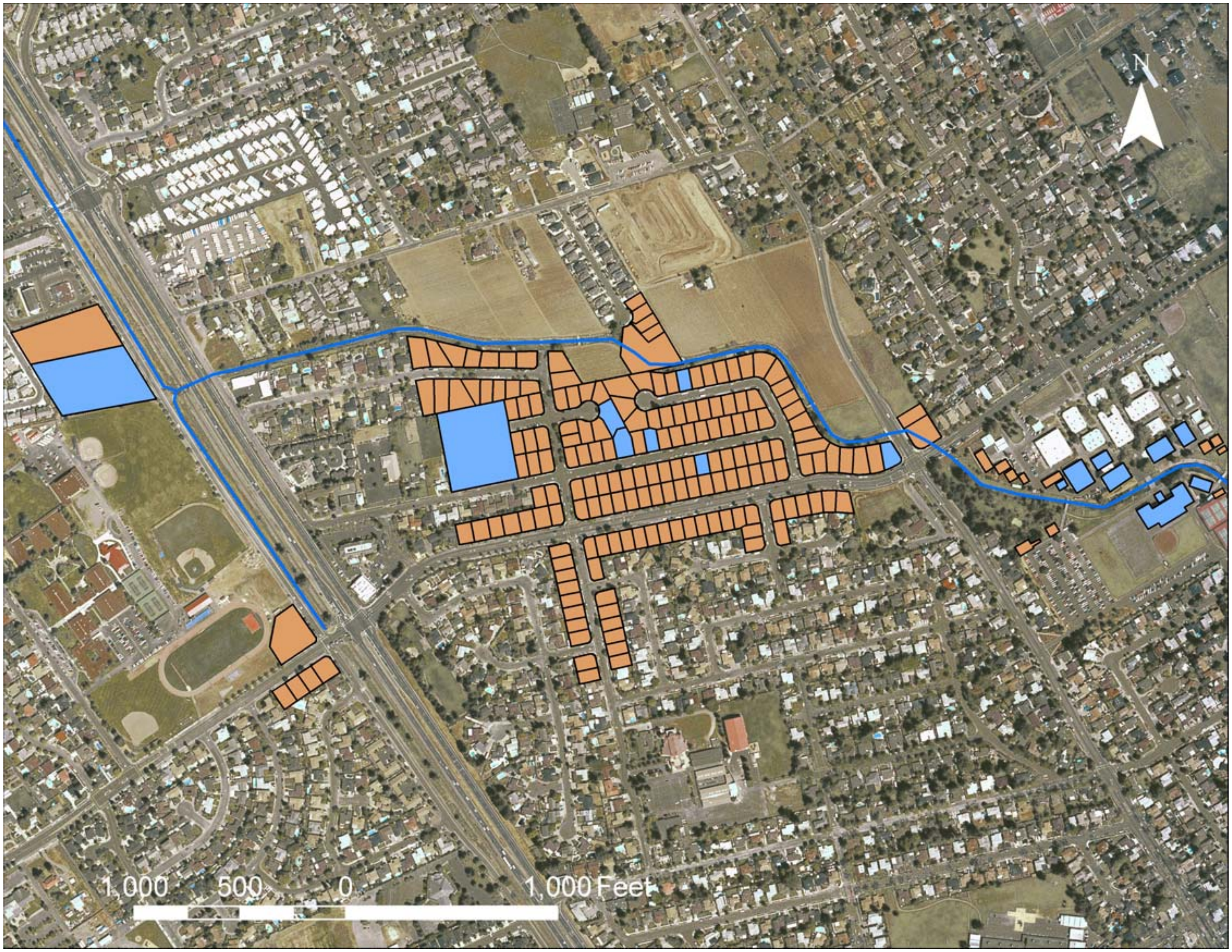

Figure 18. Comparison of the maximum inundation depths for the 100-yr flood event for Scenario 3 with the surveyed finished floor elevations, inundated parcels are shown in blue

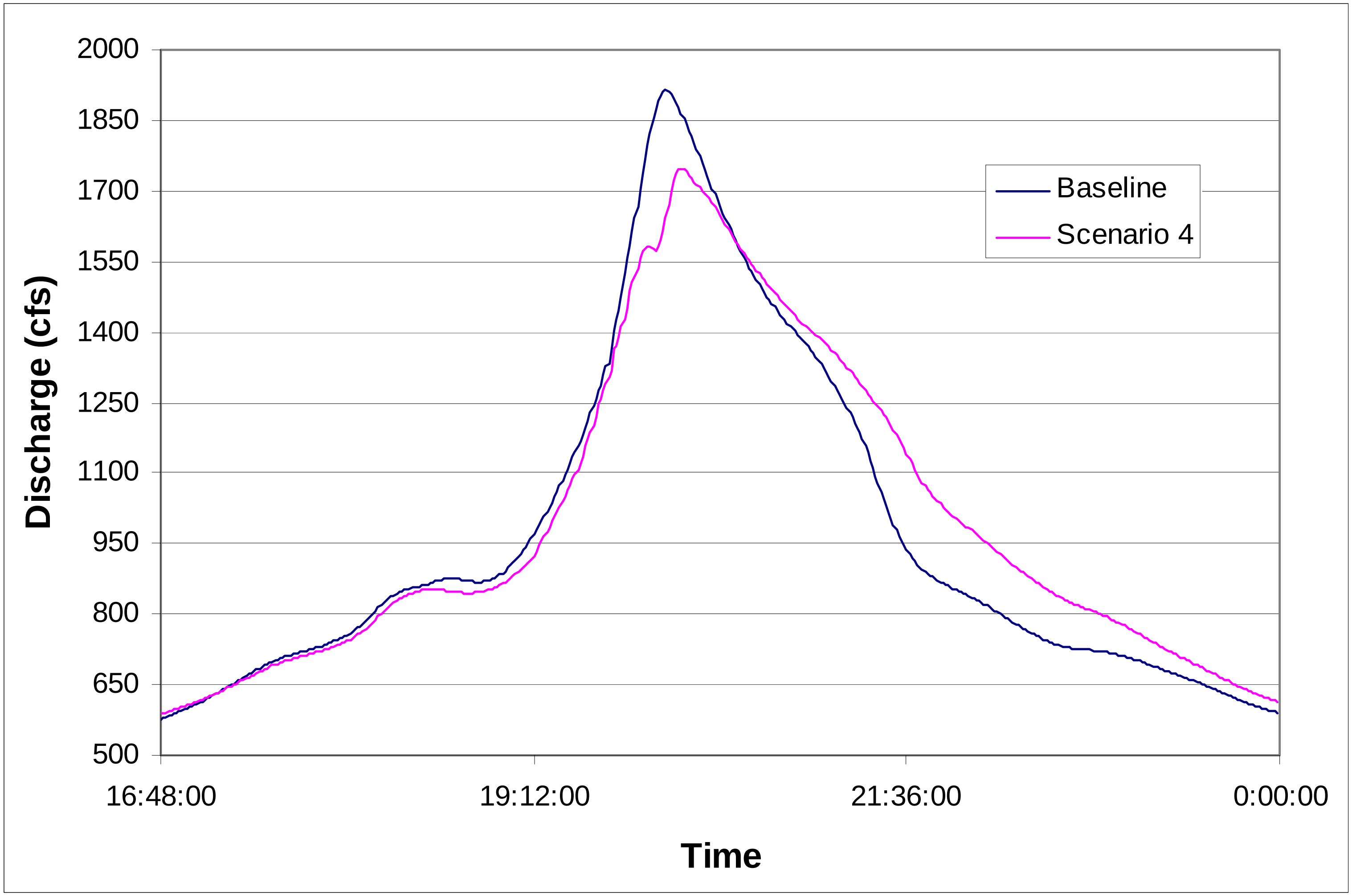

Figure 19. Comparison of the baseline and Scenario 4 hydrographs along Salvador Creek just downstream of the highway crossing

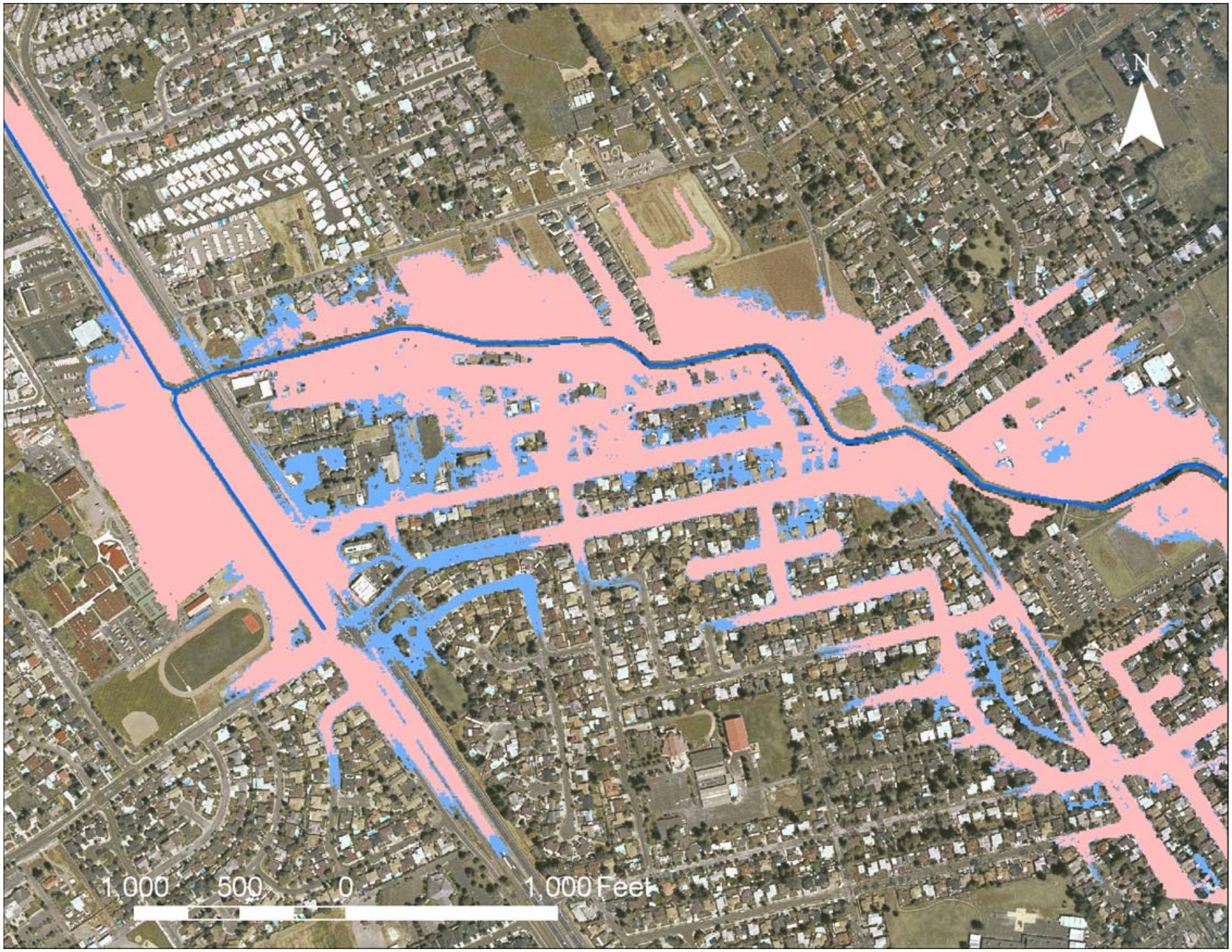

Figure 21a. Upstream view of maximum extent of inundation for Scenario 4 (pink) compared with the baseline inundation extent (light blue) for the 100-yr flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 4

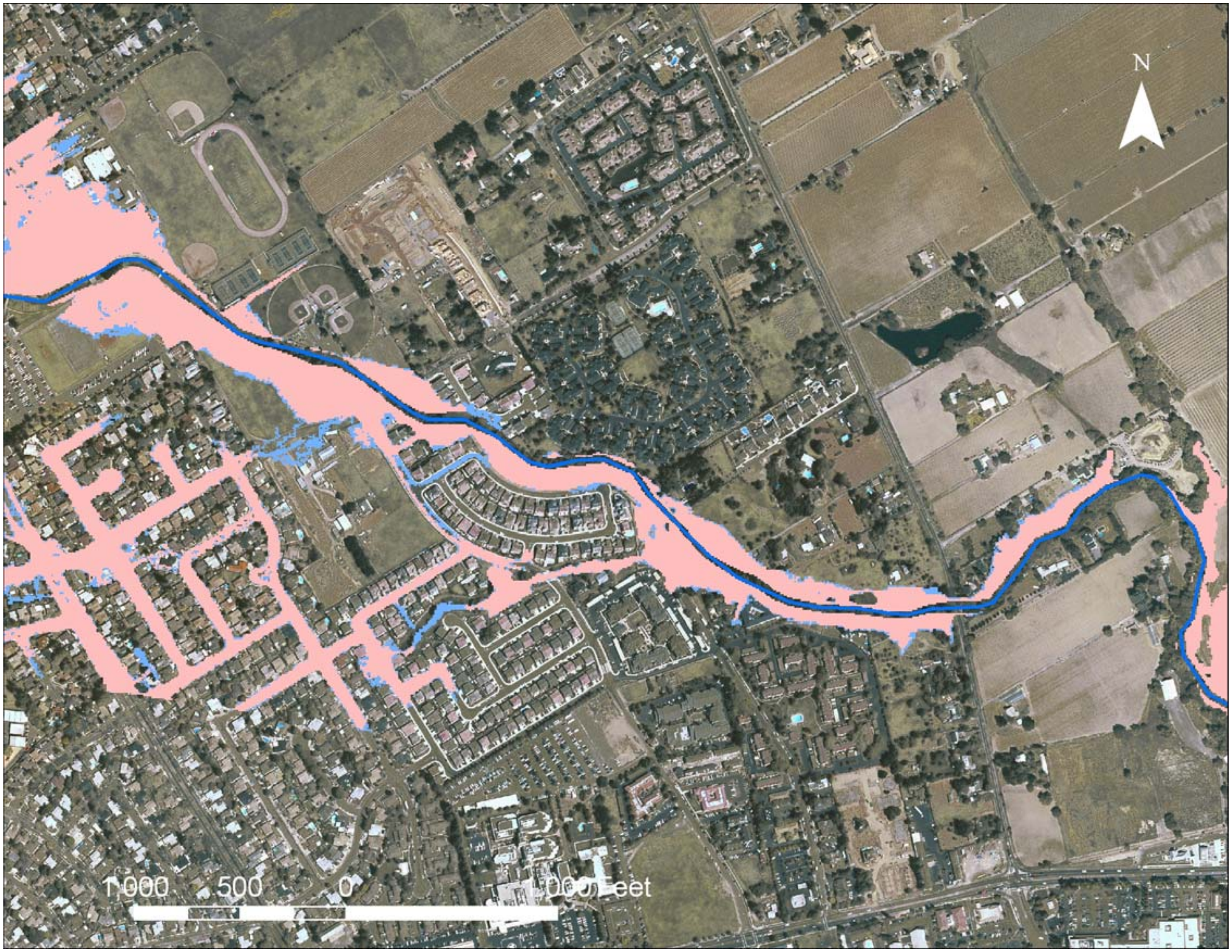

Figure 21b. Downstream view of maximum extent of inundation for Scenario 4 (pink) compared with the baseline inundation extent (light blue) for the 100-yr flood event, areas where light blue is visible indicate reduction in inundation extent under scenario 4

Figure 22. Comparison of the maximum inundation depths for the 100-yr flood event for Scenario 4 with the surveyed finished floor elevations, inundated parcels are shown in blue