Palo Alto Flood Basin Hydrology

Corrected Final Report Prepared for

Santa Clara Valley Water District

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Introduction

Purpose

This report documents a re-evaluation of watershed hydrology and hydraulic performance for the Palo Alto Flood Basin (PAFB). Flood basin hydrology was most recently studied in detail for the Santa Clara Valley Water District's Matadero and Barron Creeks Long-Term Remediation Project in 2002.

The District is interested in potentially repairing, modifying, or replacing the existing tide gate structure to improve the functionality of the tidal flood barrier system. This update will focus on a planning-level assessment of the tide basin as described in more detail herein.

Eventually the District will evaluate potential projects that could reduce flood risks within the lower reaches of Adobe, Barron, and Matadero Creeks; examine the environmental impacts due to submergence of salt marsh harvest mouse, California clapper rail, and black rail habitats within the flood basin; and understand the impact of potential sea level rise scenarios on flood protection during a 100-year fluvial flood event. This updated PAFB analysis forms the basis to examine the impact of sea level rise on flood basin performance and the efficacy of potential tide gate modifications with respect to the aforementioned District objectives.

A recently discovered model input error that affected results published in the Final Report dated July 2014 is corrected herein.

Background

The Palo Alto Flood Basin (PAFB) was created in 1956 with the construction of levees surrounding a 600-acre portion of the Palo Alto Baylands. It is maintained by the Santa Clara Valley Water District (SCVWD). Figure 1 shows a recent aerial image of the flood basin. The PAFB extends east-northeast from Highway 101 and receives inflow from Matadero Creek, Adobe Creek, Barron Creek, and the Coast Casey Storm Water Pumping Station, with a total tributary drainage area of roughly 31.5 square miles exclusive of the 585 acres of the flood basin itself. Inflow is stored in the PAFB and released to Mayfield Slough through a reinforced concrete tide gate structure whenever the water level in the PAFB is higher than the tide.

The tide gate structure consists of 8 box culverts, each with two 5-feet by 5-foot flap gates on the downstream face. The flap gates open when the water elevation in the PAFB is higher than the San Francisco Bay tide elevation. The gates close when San Francisco Bay tides rise above the elevation of stored water in the PAFB to prevent Bay waters from entering the PAFB, thereby maintaining available volume for holding creek runoff during high flow events. During the summer months the City of Palo Alto opens one of the tide gates to allow circulation of brackish Bay water within the PAFB. The tide gates have an invert elevation of –5.1 feet NGVD.

This study updates watershed hydrology and the modeling of flood basin operation to reflect the following:

1. Updated rainfall statistics compared to those used for the 2002 analysis.
2. An additional 12 years of peak annual stream flow records.
3. Additional annual maximum tide records for San Francisco Bay.
4. Changes in the Palo Alto Landfill, which drains directly to the PAFB.
5. The completion of the San Francisquito Creek Pump Station in 2009.

Figure 1. Palo Alto Flood Basin

Previous Studies

Several studies of the Palo Alto Flood Basin have been conducted over the years, in addition to the referenced 2002 Engineer's Report. These are briefly summarized for general background information.

1974 Santa Clara Valley Water District

A Report on the Storage Capability of the Palo Alto Flood Basin was completed by the Santa Clara Valley Water District in March 1974. The final construction phase of the PAFB, scheduled to begin in 1974, involved additional excavation on 440 acres in the Basin and filling approximately 100 acres along Highway 101. However in April 1973 the City of Palo Alto designated the PAFB as a Wetland Preserve in the Open Space Element of the Palo Alto General Plan. This prompted the SCVWD to determine whether the existing levees and floodwalls provided adequate flood protection, thus eliminating the need for additional excavation and filling. The results of this study recommended that the levees surrounding the PAFB be raised to an elevation of 7.0 feet (presumably NGVD), with no additional excavation or filling necessary.

1975 City of Palo Alto

The *Mathematical Model Study of the Palo Alto Flood Basin and Yacht Harbor* was completed by Water Resources Engineers for the City of Palo Alto in March 1975. This study used computer models to examine whether reintroducing a tidal marsh environment to the PAFB would affect the PAFB's ability to store 100-year flood flows. The study also modeled the addition of tide gates facing the Palo Alto Yacht Harbor to improve circulation and release sediment from the PAFB.

1984 City of Palo Alto

In the early 1980s the District again proposed to raise the levees surrounding the Palo Alto Flood Basin. This proposal was made in anticipation of improvements to the channels of the three creeks upstream of Highway 101 and into the foothills. Such improvements could increase peak flows downstream. The City of Palo Alto challenged this proposal and authorized an independent study of the situation. The *Hydrologic Analysis of the Palo Alto Flood Basin* report was then prepared by Linsley, Kraeger Associates for the City for Palo Alto in April 1984. This analysis determined that the existing levees in the PAFB provided adequate flood protection.

Summary of Work

Our basic scope of services and the work undertaken to complete this planning level study of the Palo Alto Flood Basin are summarized herein.

Information Gathering and Site Visit

The District provided record plans of the existing tide gate structures. A basin survey conducted in September 1999 is used as a basis of analysis at District direction. That survey, supplemented by 2007 County LiDAR topographic information, is used to evaluate storage-elevation relationships and the top of levee elevations. Pump station capacity data has been verified for Matadero Pump Staton (Palo Alto) and Coast Casey Pump Station (Mountain View).

An initial kickoff meeting to discuss project objectives was held on March 26, 2014 at District headquarters in San Jose. A site visit to the PAFB and surrounding areas for visual observation of general conditions and photo-documentation was made on April 2, 2014 during a period of low tide.

Updated Flood Basin Hydrology and Hydraulics

This study updates watershed hydrology and the modeling of flood basin operation to reflect the use of the District's preferred rainfall statistics, verification of model calibration with an additional 12 years of peak annual stream flow records, the evaluation of additional annual maximum tide records for San Francisco Bay, and incorporating changes in the Palo Alto Landfill, which drains directly to the PAFB.

Design Storm

The 2002 analysis was based upon the U.S. Army Corps of Engineers 72-hour storm pattern for Northern California, balanced to the District's 100-year "global regional equation" statistics for ungaged basins. For this study, the Corps' 72-hour storm pattern has been rebalanced using the 2013 return period-duration-specific equation (TDS) rainfall statistics provided by the District. It is assumed that mean annual precipitation has not substantially changed over the past eleven years.

Palo Alto Flood Basin Watershed Model

The watershed model built for the 2002 analysis is used for this work, largely without change, but antecedent moisture conditions have been verified against the flood-frequency curves for the USGS stream flow gaging stations at Matadero Creek and the San Francisquito Creek stream flow gage, which has an additional 12 points of data since the 2002 analysis was completed using gage data through 2000. The watershed model has been updated to incorporate the rebalanced design storm and converted to the HEC-HMS platform. Model parameters such as tributary areas, unit hydrographs, land uses, soil losses, and stream routing are assumed to be unchanged.

Updated Tidal Boundary Condition

Palo Alto Flood Basin performance during extreme runoff events is heavily predicated upon the elevation of low tides. The 2002 report concluded that there is a correlation between episodes of heavy stream runoff, storm surge, and significantly higher tides than those predicted astronomically. The coincident tide cycle previously used to analyze the flood basin has been updated to include the addition of recorded San Francisquito Creek peak annual discharges and coincident tides that have occurred since the original analysis was completed.

Flood Basin Performance

Updated inflow hydrographs and tidal boundary conditions have been used to reanalyze flood basin performance for the 100-year combined fluvial/tidal event. Levee containment elevations and storage-elevation data for the flood basin based on a detailed aerial survey completed by Towill in April 1999 are assumed to remain valid for this planning level study. Surveys show that there was a minimal decrease in basin capacity between 1972 and 1999, and this trend is assumed to remain true. The original analysis was based on the UNET model platform, which is outdated, so the re-analysis has been converted to unsteady HEC-RAS. The completed HEC-RAS model can be used to establish maximum one-percent flood basin elevations based on current conditions. The model has also been used to assess the relative risk of flooding due to the random nature of timing between rainfall and high tides.

Flood Basin Modeling

PAFB operation was modeled in 2002 using UNET, a one-dimensional unsteady flow model for open channels and storage areas. UNET has since been fully supplanted by the unsteady mode of HEC-RAS, so a new PAFB model has been created using HEC-RAS.

Figure 2 illustrates the HEC-RAS model that has been created and provided digitally as Appendix D. Model elements include the basin itself, labeled "PAFB", which is represented by a storage-elevation relationship. The PAFB is connected to Mayfield Slough, labeled "Slough", through a storage area to storage area connection with a series of 16 gates modeled after the tide gate structure, labeled "Gates". A set of rules written into the HEC-RAS input file prevents water from moving from the Slough into the PAFB, simulating the flap gates. Mayfield Slough is modeled with cross sectional data to open water, labeled "Tide 1", and a secondary branch, labeled "Tide 2" also connects the slough to open water on the east side of the mudflat island that is visible in Figure 2.

Modeling is completed by assigning boundary conditions, which include inflow to the PAFB from Matadero Creek ("Matadero"), Adobe Creek ("Adobe", which also includes Barron Creek discharges) and the Coast Casey Pump Station ("Coast Casey"); and the San Francisco Bay tide cycle. Interior runoff to the PAFB from the adjacent Palo Alto Landfill and direct rainfall are also incorporated into the HEC-RAS model.

Figure 2. HEC-RAS Model of PAFB

Palo Alto Flood Basin

The Palo Alto Flood Basin extends east-northeast from Highway 101 and receives inflow from Matadero Creek, Adobe Creek, Barron Creek, and the Coast Casey Storm Water Pumping Station. Inflow is stored in the PAFB and released to San Francisco Bay through a tide gate structure when the water level, or stage, in the PAFB is higher than the San Francisco Bay tides.

In the HEC-RAS model, the PAFB is represented by a storage-elevation curve that defines the volume of water that is stored at any given elevation. Towill, Inc. and MacKay & Somps conducted an aerial topographic survey of the PAFB in April 1999. From this topographic survey, a storage-elevation curve was developed. This curve is plotted against the elevation-storage curve that had been prepared by SCVWD in their 1974 PAFB analysis. Both curves are shown in Figure 3. They indicate that there was minimal change in PAFB storage capacity between 1974 and 1999. Since no additional basin topography has been gathered in subsequent years, this study assumes that the storage-elevation curve developed from 1999 data remains valid.

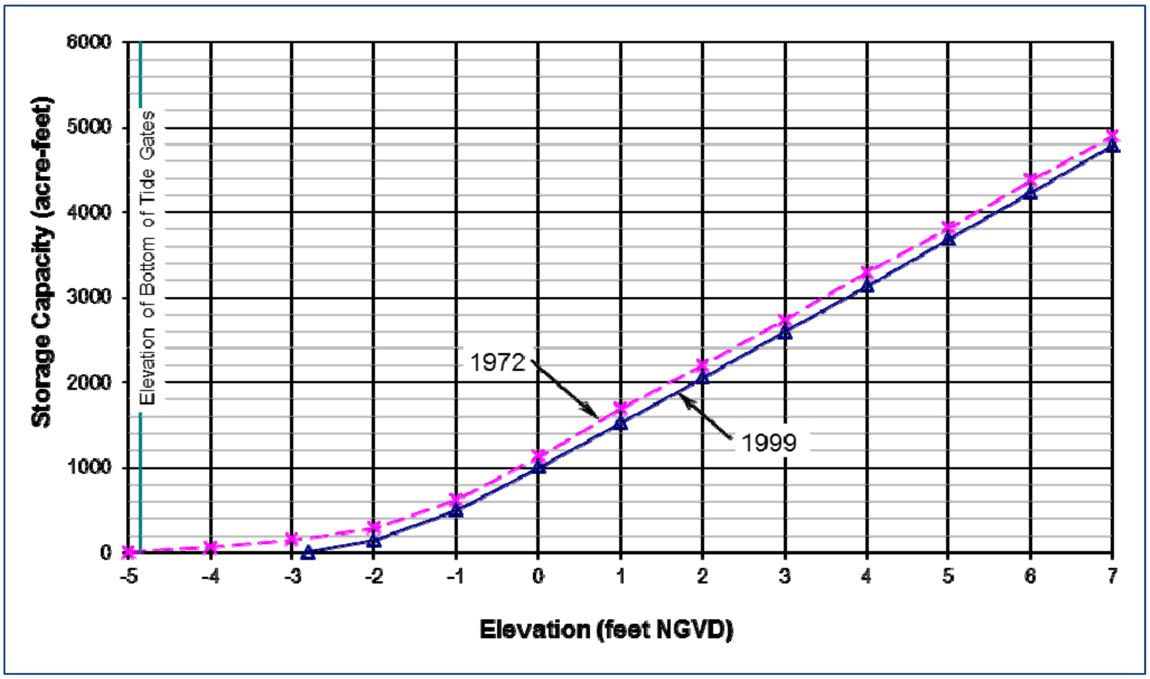

Figure 3. PAFB Storage-Elevation Curve

Mayfield Slough

The PAFB does not discharge directly to San Francisco Bay. Rather, Mayfield Slough – a smooth, relatively narrow channel that begins at the downstream face of the PAFB tide gates – conveys discharges from the tide gate structure to open water near the Dumbarton Bridge. Mayfield Slough channel geometry has been coded into the geometry file using data taken from the San Francisco Bay and Sacramento-San Joaquin Delta Digital Elevation Model (DEM) created for the California Department of Water Resources in 2012.[1] Figure 4 is clipped from the DEM and shows the general bathymetry near the tide gate structure. Elevations have been converted to the NGVD datum for the HEC-RAS cross sections by subtracting 2.684 feet from the DEM.

[1]Rueen-Fang Wang and Eli Ateljevich, San Francisco Bay and Sacramento-San Joaquin Delta DEM, November 2012. http://baydeltaoffice.water.ca.gov/modeling/deltamodeling/modelingdata/DEM.cfm

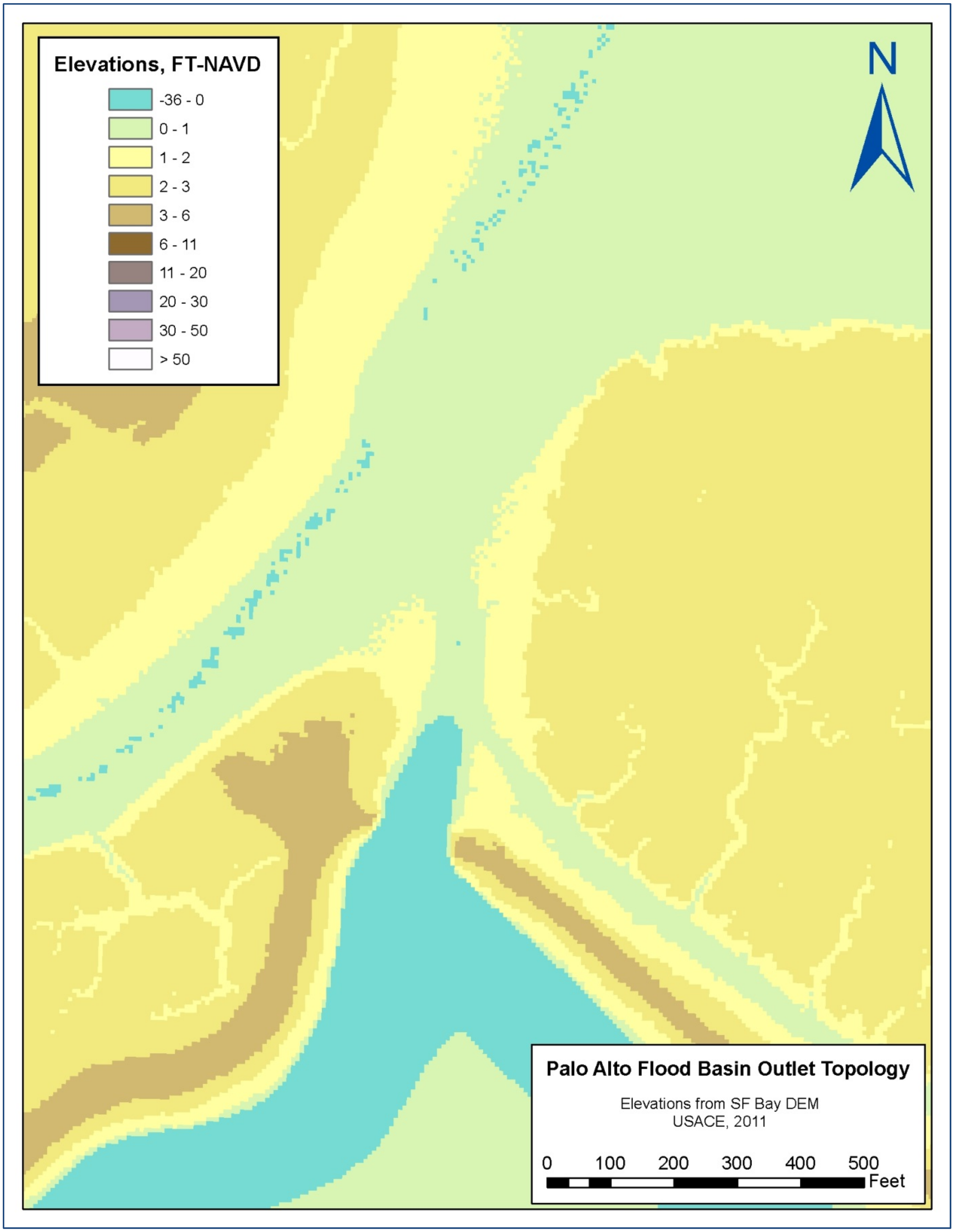

Figure 4. Bathymetry at PAFB Outlet

A channel roughness factor (Manning's "n") of 0.02 is assigned to the Mayfield Slough reach as well as the secondary slough between the PAFB levees and higher mudflats to the immediate north.

Tide Gate

Inflow is stored in the PAFB and released to Mayfield Slough through a tide gate structure when the water level, or stage, in the PAFB is higher than the San Francisco Bay tides. This tide gate structure consists of eight box culverts, each with two 5-foot by 5-foot cast iron flap gates on its downstream face. These flap gates open when the stage in the PAFB is higher than the water surface elevation in Mayfield Slough, which is predominantly controlled by San Francisco Bay tide elevations. The gates close when San Francisco Bay tides rise to prevent Bay waters from entering the PAFB, thereby maintaining available volume for holding creek runoff during high-flow events. During the summer months the City of Palo Alto opens some of the tide gates using a sluice gate feature to allow circulation of brackish bay water into the PAFB. The tide gates have an invert elevation of –5.1 feet NGVD. A plan an elevation of the tide gate structure from record drawings and a photograph taken during the referenced site visit are provided as Figure 5.

In HEC-RAS the tide gate structure is modeled as a connection between the PAFB and Mayfield Slough. Each box culvert has two gates (radial gates mimic manufacturers' head-discharge curves the best in the model), and the gates are coded so as to only allow flow from the PAFB to Mayfield Slough. Each flap gate is assumed to open upon a minimal differential head (0.2 foot) to mimic gate manufacturers' literature.

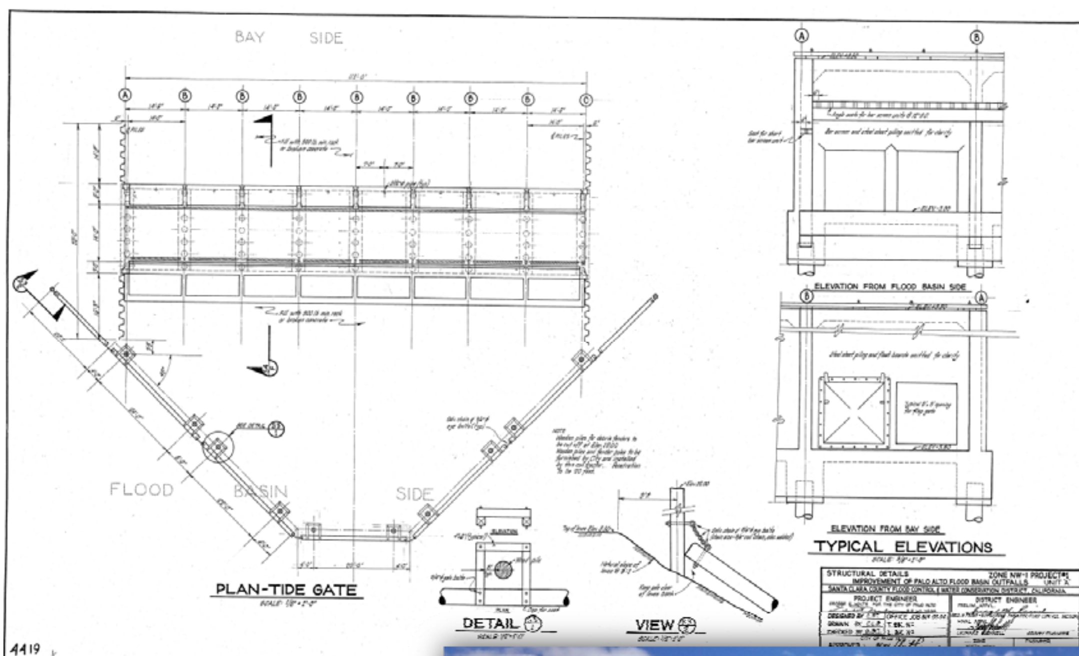

Figure 5. PAFB Tide Gate Structure

Inflow to the PAFB

With the Palo Alto Flood Basin's stage-storage relationship, tide gate structure, and discharge connection to San Francisco Bay modeled, boundary conditions are needed to complete the evaluation of basin performance. The total inflow into the PAFB is one driving boundary condition. Estimates of inflow to the PAFB from a design 100-year, 72-hour precipitation event are based on rainfall-runoff models that have been calibrated to flood-frequency analyses of local stream flow data as described herein. Analytic methods remain largely unchanged from the 2002 Final Engineer's Report, with the exception of the design storm and calibration of antecedent moisture conditions to that storm.

The volume of storm water runoff produced from a given precipitation event depends on a number of factors, most prominently precipitation, watershed losses, and the convolution of unit hydrographs. The rainfall-runoff model for the PAFB watershed completed using HEC-1 in 2002 has been converted to the HEC-HMS platform and provided digitally as Appendix D. Comparing summary results for each model platform, it is clear that simply moving the HEC-1 model to HEC-HMS does not significantly change the watershed model or its numeric results.

Tributary Watershed

Areas tributary to the Palo Alto Flood Basin generally include the areas and tributaries draining to Matadero Creek, Barron Creek, and Adobe Creek; areas that drain to the Coast Casey Pump Station and forebay in Mountain View; a portion of the Palo Alto Landfill; and the PAFB itself. Figure 6 provides the delineated watershed boundaries superimposed over an aerial photograph.

HEC-HMS is used to generate inflow hydrographs (except for direct rainfall on the PAFB itself as explained subsequently) and the watershed is broken into tributary sub-watersheds as shown in Figure 7. Sub-watersheds and their designations are taken directly from the 2002 Engineer's Report. Sub-basin and design point label names were originally designated by the District and have remained unchanged.

Appendix A provides summary tables of the tributary watershed parameters used in the HEC-HMS model (Figures 8 and 9) and described in this section, including identification, basin area, mean annual precipitation at centroid, basin length, length to centroid, basin slope, curve number, percent impervious cover, storm drain system routing using the District's unitized storage curves, and stream routing parameters.

Figure 8 illustrates the HEC-1 model schematic used to complete the 2002 Engineer's Report. Figure 9 shows the conversion of that schematic to the HEC-HMS platform, and the addition of a sub-basin for the Palo Alto Landfill.

Adobe Creek, with sub-basins labeled with an "A" prefix, drains to its junction with Barron Creek (Junction 1), which has a "B" prefix for its sub-basins. The Barron Creek sediment basin and diversion structure, which are located behind Gunn High School off Arastradero Road at Design Point "E", are labeled "BSED" and "BDIV" respectively in Figures 7 through 9. The Barron Creek sediment basin and diversion structure are passively operated (but with the potential for active operation) to limit discharge into the downstream reaches of Barron Creek by diverting flow in an underground culvert along the Bol Park bike path. Where the bike path crosses Matadero Creek (Design Point "C"), another diversion structure ("MDIV") adds additional flow that is diverted from the natural creek at this location. The combined discharge continues in an underground structure known as the Matadero Bypass until it reaches a confluence with the natural Matadero Creek at El Camino Real, also collecting runoff from the Stanford Channel along the way.

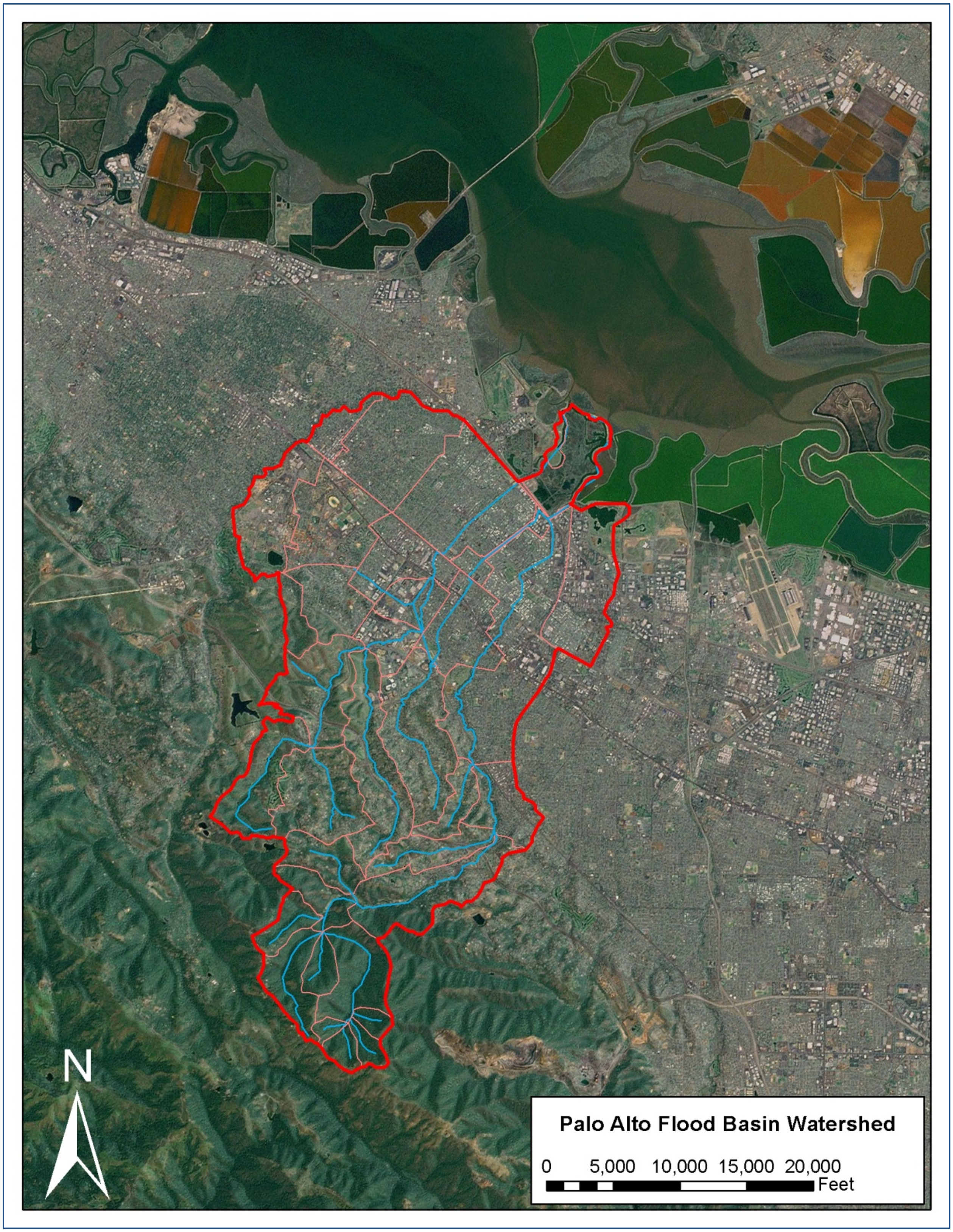

Figure 6. Palo Alto Flood Basin Watershed

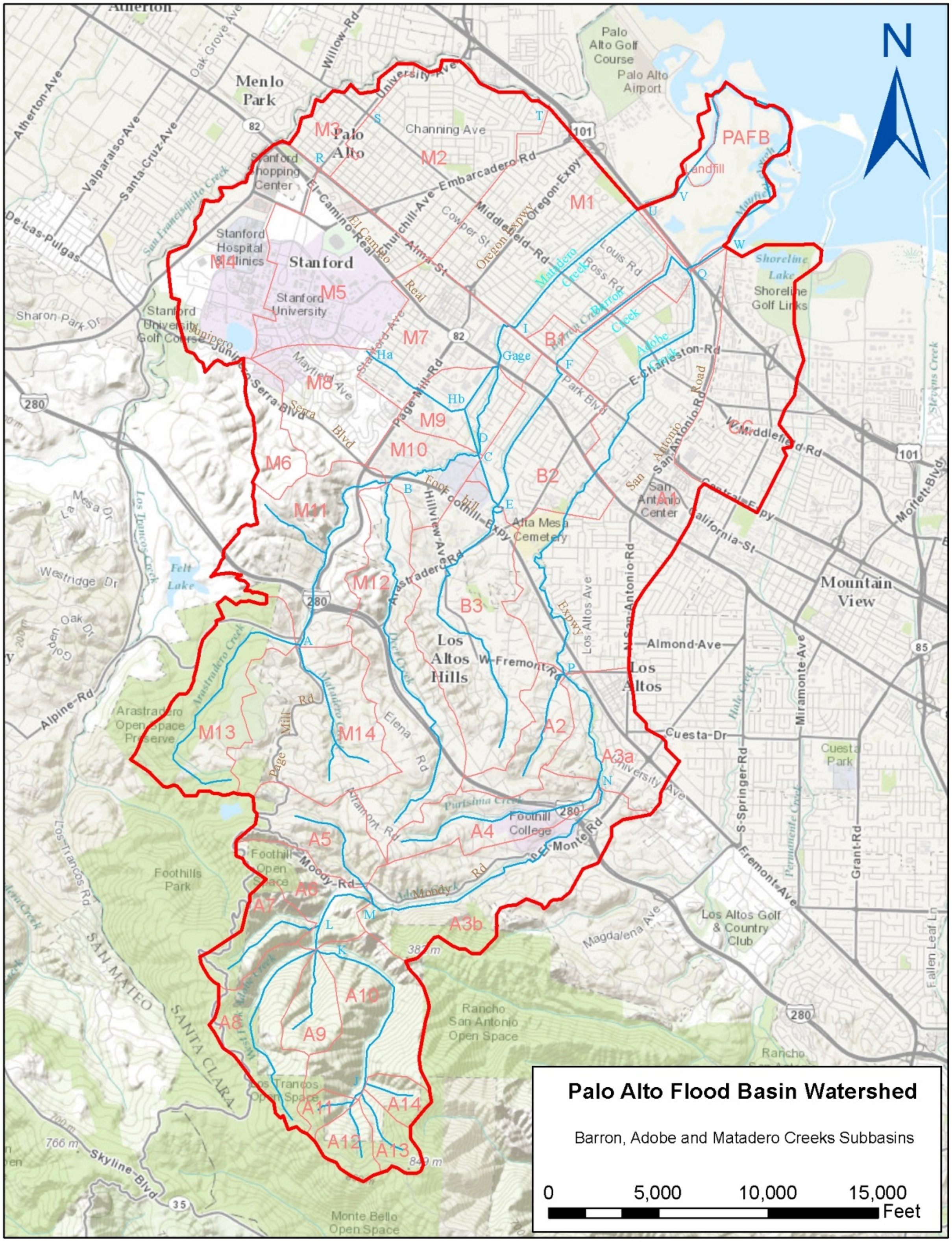

Figure 7. PAFB Sub-basins for HEC-HMS

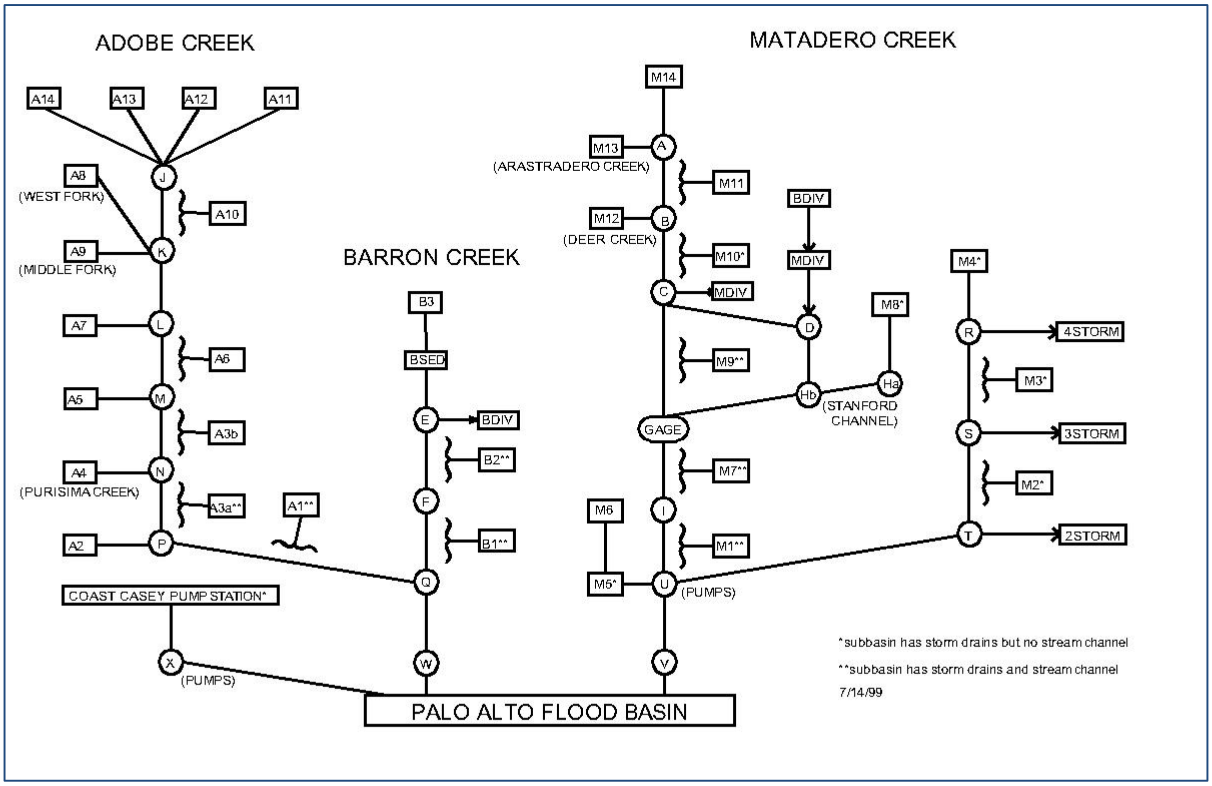

Figure 8. HEC-1 Model Schematic (2002 Engineer's Report)

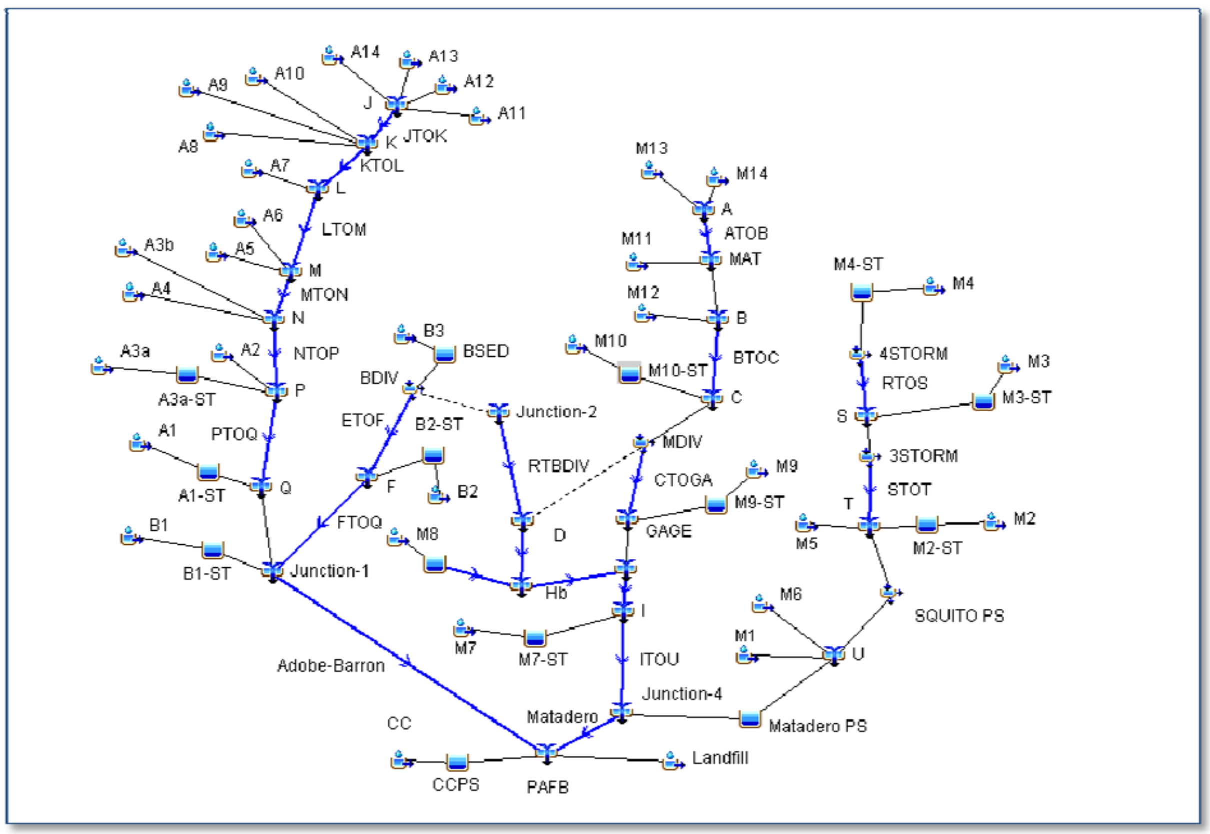

Figure 9. Updated HEC-HMS Watershed Model Schematic

The USGS stream flow gage is located downstream from El Camino Real adjacent to Boulware Park and measures the combined discharge in Matadero Creek. Between the park and Highway 101, Matadero Creek is contained within an engineered channel that is fully concrete-lined between El Camino Real and Greer Road. Matadero Creek is earthen with concrete flood walls between Greer Road and Highway 101. Downstream of the freeway, Matadero Creek bifurcates with substantial discharge carried in a bypass around the City of Palo Alto's Municipal Services Center into the PAFB.

The Matadero Pump Station, owned and operated by the City of Palo Alto, discharges to Matadero Creek nearly half-way between West Bayshore Road (frontage to Highway 101) and Greer Road. In addition to inflow from the pump station's tributary local storm drain system, runoff that exceeds the capacities of storm drain systems tributary to San Francisquito Creek (typically equal to the 10-year return period) naturally flows downhill toward Matadero Creek and to the extent of available storm drain system and pump station capacity (266 cfs), is discharged into Matadero Creek at Design Point "U". The flow of runoff out of the Matadero system and into San Francisquito Creek is marked in the model schematics as "SQUITO PS", "3STORM", and "4STORM". The San Francisquito Creek Pump Station, owned and operated by the City of Palo Alto, has four axial flow pumps with a total pumping capacity of 300 cfs. The sub-basin tributary to Mountain View's Coast Casey Pump Station ("CC") is modeled as is the storage-discharge relationship provided by the Coast Casey Forebay and its 150 cfs pump station. The Coast Casey Pump Station discharges directly into the PAFB through three steel discharge pipes with flap gates at their outfall as shown in Figure 10.

Figure 10. Coast Casey Pump Station Outfall to PAFB

Precipitation

The volume of runoff (Q) depends primarily on the volume of precipitation (P). "Design storm" is a term used to describe the total rainfall volume measured as depth, which is determined from the combination of a return period and storm duration. By definition, the base flood elevation has a 100-year return period, which means that a storm of such magnitude (as measured by total rainfall depth) has a one percent annual chance of being equaled or exceeded in any given year.

The selection of storm duration is rendered irrelevant to the prediction of peak discharge by balancing the design rainfall pattern to replicate local depth-duration-frequency statistics, and by calibrating soil loss parameters to match flood frequency analyses of local stream flow data.

The precipitation pattern used in this analysis is based upon a three-day December 1955 rainfall event compiled by the U.S. Army Corps of Engineers; an event that is still considered to be the storm of record for northern California. This pattern is adjusted to preserve local rainfall statistics using the work of the Santa Clara Valley Water District from 2013.

Rainfall Depth

The Santa Clara Valley Water District's 2013 Return Period-Duration-Specific (TDS) Regional Equation is used to establish a relationship between precipitation depth and mean annual precipitation for various storm frequencies (return periods). The mean annual precipitation at each sub-basin's centroid is based on a mean annual precipitation (M.A.P.) map published by the Santa Clara Valley Water District in 1989. Once the mean annual precipitation for a given location is determined, rainfall depths are calculated using the TDS Regional Equation:

$$x_{T,D} = A_{T,D} + \left(B_{T,D}\ MAP\right)$$

Where $X_{T,D}$ is precipitation depth for a specific return period and storm duration (inches);

T is return period (years);

D is storm duration (hours); and

A, B are coefficients determined from Table 1, which also provides the rainfall depth and percent total for a mean annual precipitation of 17.5 inches.

Table 1. 1% Rainfall Coefficients and Depths for TDS Equation with 17.5" MAP

Duration (hours)	A	B	Depth (inches)	% Total
0.25	0.4618	0.0023	0.50	8.6%
0.5	0.4901	0.0077	0.62	10.7%
1	0.5074	0.0190	0.84	14.4%
2	0.5317	0.0389	1.21	20.8%
3	0.4980	0.0579	1.51	25.9%
6	0.3228	0.1082	2.22	38.0%
12	0.2588	0.1613	3.08	52.9%
24	0.1102	0.2170	3.91	67.0%
48	0.3239	0.2751	5.14	88.1%
72	-0.0876	0.3382	5.83	100.0%

Statistically Balanced Rainfall Patterns

For this study the USACE Christmas 1955 precipitation pattern (Figure 11) has been adjusted to preserve local rainfall statistics compiled by the Santa Clara Valley Water District (SCVWD) for three mean annual precipitation values. That is the peak 15-minute, 30-minute, 1-hour, 2-hour, 3-hour, 6-hour, 12-hour, 24-hour and 48-hour rainfall depths that are embedded within the 72-hour patterns all conform to the statistics provided in Table 1. The statistically balanced rainfall pattern for a mean annual precipitation of 17.5 inches is shown in Figure 12. Statistical balancing has been performed using the hydrograph transformation function (HB card) available in HEC-1, since that function is not incorporated into HEC-HMS.

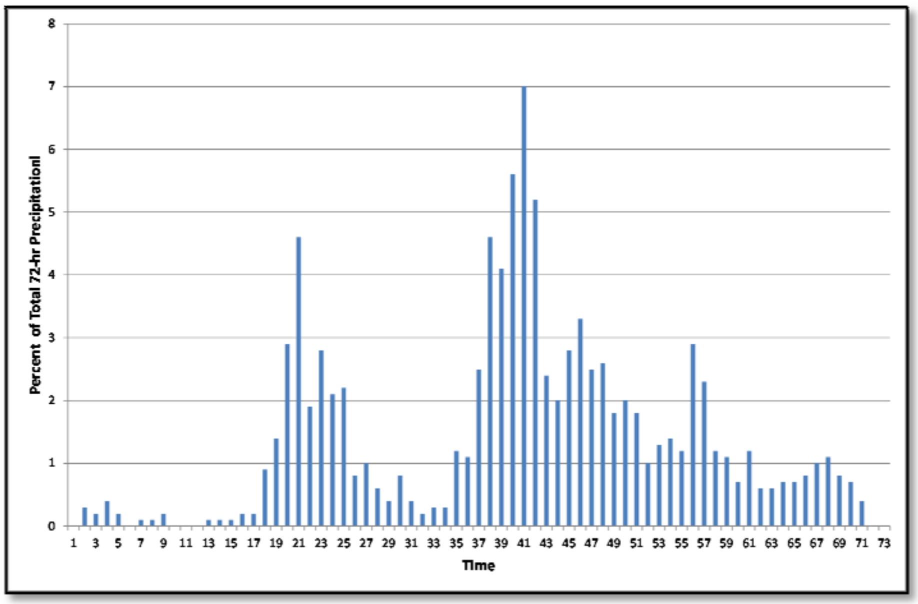

Figure 11. 72-hour USACE Rainfall Pattern (Christmas 1955)

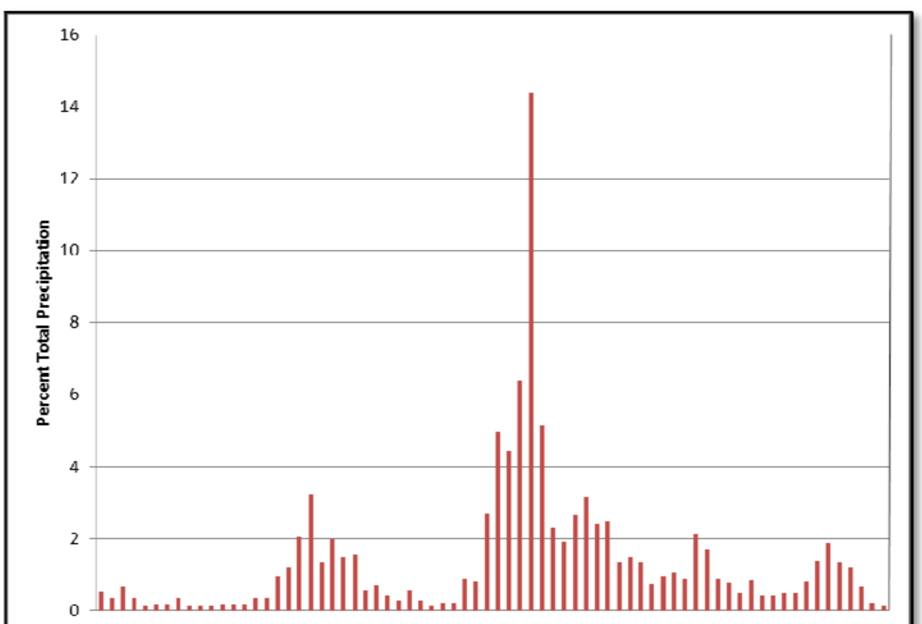

Figure 12. Balanced 15-min, 72-hr Rainfall Pattern (MAP = 17.5")

This approach, together with the soil loss parameter calibration procedure subsequently described, ensures that flood frequency estimates do not depend upon the selection of a storm pattern or duration. Furthermore, since the depth-duration relationships depend only upon mean annual precipitation (MAP) at any particular location, the statistically balanced rainfall pattern may be applied to different watersheds simply by changing the total 72-hour rainfall depth as a function of MAP.

Specific rainfall patterns do depend on the mean annual precipitation, which ranges from 13.5 inches at San Francisco Bay to 37.6 inches at the headwaters of Adobe Creek (Sub-basin A-12). Three distinct rainfall patterns are used ("gages" in HEC-HMS) to account for the range in mean annual precipitation within the PAFB watershed. Table 2 maps the three specified balanced hyetographs (rainfall patterns) that are used in the meteorological model to mean annual precipitation ranges.

Table 2. Hyetographs ("Gages") Used in HEC-HMS

Gage Name	Low Range Mean Annual Precipitation (inches)	High Range Mean Annual Precipitation (inches)
MAP17.5Pattern	13.5	21.5
MAP25.5Pattern	21.5	25.5
MAP33.5Pattern	25.5	37.6

Runoff Curve Numbers

The Soil Conservation Service (SCS, now the National Resources Conservation Service) Curve Number methodology is used to estimate direct runoff by subtracting soil infiltration and other losses from the rate of rainfall. The Curve Number (CN) method is an empirical methodology wherein the CN reflects potential loss for a given soil and cover (land use) complex. After satisfying an initial abstraction – rainfall absorbed by tree cover, depressions, and soil at the beginning of a storm – the soil becomes saturated at a certain rate so that a higher percentage of the accumulated rainfall is converted to runoff. The initial abstraction is set to 0.2S where S = (1000/CN) – 10.

Estimates of the CN are made based on the soil types and cover within a drainage basin. The number varies from 0 to 100, and represents the relative runoff potential for a given soil-cover complex for given AMC. Appendix A contains tables showing the development of Curve Numbers for each sub-basin.

Curve numbers for the Palo Alto landfill and PAFB wetlands are based on literature research. The landfill and PAFB are underlain by Bay Mud and the assumed hydrologic soil group is Type "D", which represents the least permeable soil. For a municipal landfill the Curve Number for HSG "D" is 93;[2] and for a wetland complex, the Curve Number for HSG "D" is 98.[3]

[2] Palos Verdes Landfill Remediation Investigation Report, Appendix E.13 "Hydrologic Evaluation of Landfill Performance (HELP Model), undated.

[3] St. John's River Water Management District Department of Water Resources (Palatka, FL), "A Guide to SCS Runoff Procedures" (Technical Publication No. 85-5), July 1985.

Calibration of Antecedent Moisture Condition and Base Flow

Curve Numbers are adjusted to reflect the antecedent moisture condition (AMC), which is a measure of soil saturation at the beginning of the storm period. AMC is characterized by the SCS as:

AMC I	soils are dry
AMC II	average conditions
AMC III	heavy rainfall, or light rainfall with low temperatures; saturated soil

Rather than select AMC arbitrarily or *a priori*, antecedent moisture conditions are calibrated for the statistically balanced storm patterns used in this study. The following procedure is used to calibrate the PAFB watershed models using flood frequency analyses of recorded stream flow gage data for Matadero Creek and nearby San Francisquito Creek.

1. Perform statistical analyses of stream flow data at the USGS gages on Matadero Creek in Palo Alto and San Francisquito Creek at Stanford. Confirm statistical correlation between gage data.
2. Prepare a rainfall-runoff model for the watershed tributary to the San Francisquito Creek gage, which is adjacent and hydrologically similar to the PAFB watershed.
3. Using the design 100-year rainfall pattern, adjusted for the mean annual precipitation at the centroid of the San Francisquito Creek watershed, calibrate the San Francisquito Creek model by adjusting AMC to replicate 100-year flood frequencies for peak discharge and runoff volume.
4. Use the calibrated AMC to adjust Curve Numbers within the PAFB watershed model.
5. Compare the modeled 100-year discharge at the location of the Matadero Creek gage to the flood-frequency analysis at that gage for a measure of model verification.

Statistical Analysis of Matadero Creek Stream Flow Data

The United States Geologic Survey (USGS) has operated a stream gage (No. 11166000) on Matadero Creek since 1953, with no record in 1992 during construction of channel improvements. Ideally the data set used for statistical analyses of stream flow will provide a representative sample of random and homogeneous natural events, so that annual peak flow data define an unbiased estimation of future flood risk.

Within the Matadero Creek watershed as measured at its gage (Figure 7), however, events have occurred over the years that may introduce bias into the frequency analysis. These events include some increase in basin urbanization since the early 1950s (the basin is now roughly twenty percent impervious), and flow diversions from the Barron Creek began in September 1996 (Water Year 1997). Cumulative urbanization can increase the lesser annual flow peaks relative to what they would have been without urbanization, which can reduce the standard deviation of the data set and thereby the estimates for the magnitude of extreme runoff events. The Barron Creek diversion regulates measured stream flow at the Matadero Creek gage for some annual peaks, is a significant non-homogeneity in the record, and therefore must be accounted for if those peak discharge values are to be included with the systematic record.

Flow diversions from Barron Creek into Matadero Creek were recorded during the peak discharge events in Water Years 1998 and 2000, but not in 1997 or 1999. While it is possible that there were no actual diversions during those years, a continuous record that could verify this does not exist, so data from 1997 and 1999 are excluded from the frequency analysis. Detailed flow diversion records are not available for water years beyond 2000, so the data set remains unchanged from the data set used in the 2002 Engineer's Report and is represented herein for the record.

Recorded annual maximum discharges on February 2-3, 1998 and February 13, 2000 are adjusted to eliminate regulated diversions from the Barron Creek watershed. Based on a physical model study of the diversion structure (CH2M-Hill, 1991), average flow velocity within the Matadero Creek bypass channel while it carries the design discharge is 16 feet per second. Since the total distance from the Barron Creek Sediment Basin to the gage location is 7,500 feet, the travel time is:

$$\frac{7{,}500\ feet}{16\ ft/s} = 470\ s = 7.8\ minutes$$

Stage in the Barron Creek diversion basin was recorded every 30 minutes during the two flood events. Since the travel time to the gage is about one-quarter of that recording interval, it is assumed that the stage recorded at the diversion basin is roughly coincident with USGS stream flow measurements at the gage. Correcting the stream gage record to reflect undiverted flows involves subtracting Barron Creek diversions based on recorded stage at the diversion basin, using critical depth for unpressurized flow and the orifice equation when stage reaches the bottom of the steel plate at the diversion gate. Stage-discharge relationships for the flow data adjustment are:

When Stage < 83.7 feet	Diverted Flow = 0	
When 83.7 feet < Stage < 87.7 feet	Critical Depth Control	$Q_{diverted} = b\sqrt{y_c^3\ g}$
Where b = net width of open diversion gate(s) = (No. of Gates Open)(9.79 feet)		
$Y_c = (2/3)\ E_c$		
E_c = Stage – 83.7 feet		
When Stage > 87.7 feet	Orifice Control	$Q_{diverted} = C\ A\ \sqrt{2\ g\ \Delta h}$
Where C = 0.53 (ref. CH2M-Hill)		
Δh = Stage – 85.7 feet (centerline of gate)		

Tables 3 and 4 provide the calculations of diversion adjustment for the peak discharges in Water Year 1998 and Water Year 2000, respectively. Flood-frequency analysis procedures outlined in USGS Bulletin #17B are used with the Matadero Creek stream flow data set, adjusted for known diversions to obtain a flood-frequency plot. Following Bulletin 17B procedures for the systematic record of 1953 through 2000 (excluding 1992, 1997, and 1999), the low outlier is 23 cfs. If 1954 (26 cfs) and 1957 (28 cfs) are eliminated, the low outlier is 40 cfs. If 1961 (45 cfs) is eliminated, the low outlier is 50 cfs. If 1976 (81 cfs) is eliminated, the low outlier is 58 cfs, indicating that 1976 belongs in the data set. Table 5 summarizes the statistical results for Matadero Creek with a low outlier test criterion of 50 cfs, and is unchanged from the 2002 Engineer's Report.

Table 3. Matadero Creek Diversion Adjustment February 2-3, 1998

Time	Recorded Flow at Gage (cfs)	Recorded Stage at Basin (feet)	Diversion (cfs)	Adjusted Flow at Gage (cfs)	Remarks
21:30	1156	87.4	430	726	2 gates open
22:00	1320	87.9	494	826	
22:30	1350	86.7	314	1036	
23:00	1380	86.5	283	1097	
23:30	1410	87.0	363	1047	
24:00	2557	88.2	527	2030	
0:30	2541	88.5	279	**2262**	1 gate closed
1:00	2259	88.6	284	1975	
1:30	1796	88.5	279	1517	
2:00	1778	88.2	264	1514	
2:30	1566	88.0	253	1313	

Table 4. Matadero Creek Diversion Adjustment February 13, 2000

Time	Recorded Flow at Gage (cfs)	Recorded Stage at Basin (feet)	Diversion (cfs)	Adjusted Flow at Gage (cfs)	Remarks
16:58	1065	85.3	61	1004	1 gate open
17:11	1105	85.7	86	1019	
17:14	1146	85.7	86	1060	
17:53	1189	85.8	92	1097	
18:08	1271	86.1	112	1159	
18:14	1316	86.1	112	**1204**	
18:16	1271	86.1	112	1159	
18:41	1316	86.3	127	1189	
20:00	1321	86.2	120	1201	
20:10	1232	86.2	120	1112	
20:15	1189	85.5	73	1116	
21:00	1026	84.9	40	986	

Table 5. Flood-Frequency Statistics for Matadero Creek

Parameter	Value
Mean of Logarithms	2.606
Standard Deviation (S)	0.335
Station Skew (G)	-0.070
Regional Skew, SCVWD	-0.600
Weighted Skew (G_w)	-0.226
100-year Discharge ($Q_{1\%}$)	2,130 cfs

Statistical Analysis of San Francisquito Creek Stream Flow Data

Bulletin 17B suggests that comparisons between computed frequency curves for hydrologically similar regions are useful for testing the reasonableness of flood flow frequency determinations. The centroid of San Francisquito Creek's watershed is roughly six miles from Matadero Creek's watershed centroid, so this is a natural comparison to make. The San Francisquito Creek gage began recording stream flows in 1932, and provides 73 years of record through 2012 with missing data from 1942 to 1950. There are no diversions within the watershed, or substantial urbanization over the period of record.

An updated flood-frequency plot for San Francisquito Creek at Stanford has been created following the same procedures outlined in USGS Bulletin #17B and as modified for low outlier testing as described for the Matadero Creek gage analysis. The final tested low-flow outlier threshold is 139 cfs. Low-flow outliers are 1939 (120 cfs), 1957 (125 cfs), 1961 (12 cfs), 1976 (82 cfs), and 1977 (82 cfs). Figure 13 shows the adjusted flood-frequency curve for San Francisquito Creek at Stanford, updated with verified annual peak discharge data through Water Year 2012. The one percent discharge at the gage location is 7,800 cfs. Table 6 provides a summary of the final synthetic statistics with low outliers removed and the conditional probability adjustment.

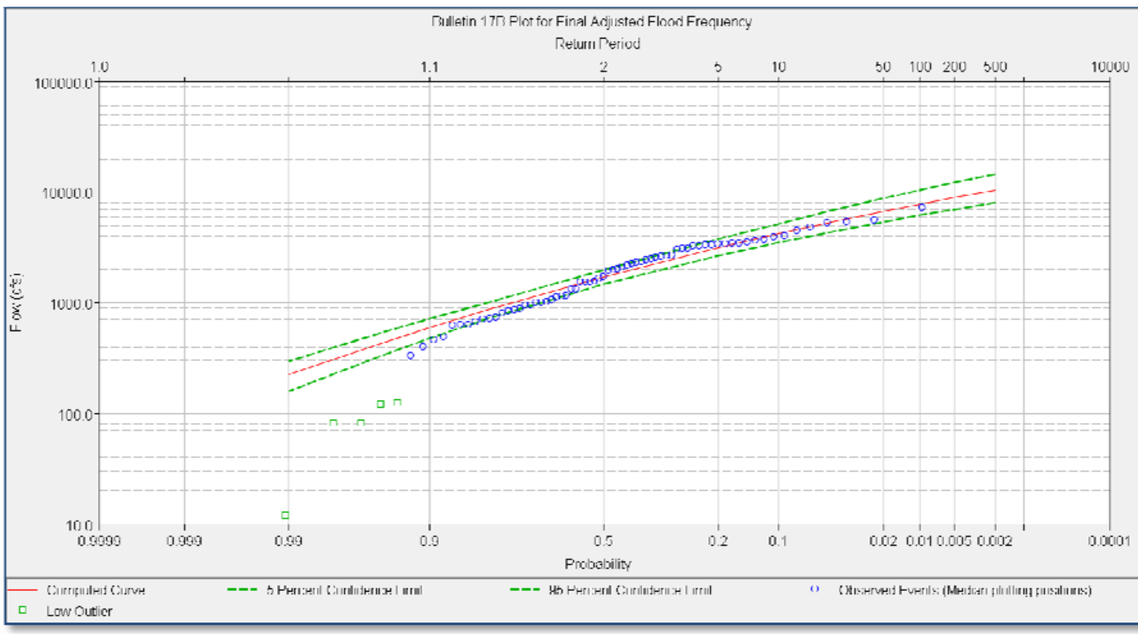

Figure 13. Flood-Frequency Plot for San Franciscquito Creek at Stanford

Table 6. Flood-Frequency Statistics for San Francisquito Creek

Parameter	Value
Mean of Logarithms	3.212
Standard Deviation (S)	0.332
Station Skew (G)	-0.309
Regional Skew, SCVWD	-0.600
Weighted Skew (G_w)	-0.376
100-year Discharge ($Q_{1\%}$)	7,810 cfs

Correlation of Matadero Creek to San Francisquito Creek

Bulletin 17B provides a procedure for adjusting a "short" record to reflect experience at a nearby long-record station. "Short" records are defined as those less than 50 years in length, so the Matadero Creek data set qualifies. With 73 years of record, the San Francisquito Creek gage qualifies as a long-record station. The first step of the procedure is to correlate observed peak flows for the short record with concurrent observed peak flows for the long record as follows:

Regression Coefficient

$$b = \frac{\sum X_1 Y_1 - \frac{\sum X_1 \sum Y_1}{N_1}}{\sum X_1^2 - \frac{(\sum X_1)^2}{N_1}}$$

Correlation Coefficient

$$r = b\,\frac{S_{X_1}}{S_{Y_1}}$$

Excluding outliers, the concurrent record includes 1953, 1955, 1956, 1959, 1960, 1962-1975, 1978-1991, 1993-1996, 1998, and 2000. Table 7 presents statistical parameters for the flood-frequency correlation.

Table 7. Statistics for Flood-Frequency Correlation

Parameter	Value
Number of years peak flow concurrently observed at the two sites (N_1)	40
Number of years peak flows observed at long record site, but not short record site (N_2)	28
Mean of logarithms of flows from long record during concurrent period (X_1)	3.241
Mean of logarithms of flows at long record site for period with no flows at short record site (X_2)	3.247
Mean of logarithms of flows for entire period at long record site (X_3)	3.243
Mean of logarithms of flows from short record during concurrent period (Y_1)	2.637
Standard deviation of logarithms of flow from long record during concurrent period (S_{X1})	0.329
Standard deviation of logarithms of flow at long record site for period with no flows at short record site (S_{X2})	0.308
Standard deviation of logarithms of flow from short record during concurrent period (S_{Y1})	0.311
Regression coefficient (b)	0.886
Correlation coefficient (r)	0.938

Since there is such a strong correlation between data sets (the correlation coefficient is 94%) improved estimates of the short record mean and standard deviation can be made:

$$\bar{Y} = \overline{Y_1} + b\,(\overline{X_3} - \overline{X_1})^2 = 2.6394$$

Adjusted variance is computed:

$$S_y^2 = \frac{1}{(N_1 + N_2 - 1)}\left[(N_1 - 1)S_{Y_1}^2 + (N_2 - 1)b^2 S_{X_2}^2 + \frac{N_2(N_1 - 4)(N_1 - 1)}{(N_1 - 3)(N_1 - 2)}(1 - r^2)S_{Y_1}^2 + \frac{N_1 N_2}{N_1 + N_2} b^2 (\overline{X_2} - \overline{X_1})^2\right]$$

The adjusted variance (S_y) is 0.3018, which represents a 19 percent reduction in short-station variance. This reduction in variance remains the same from the 2002 Engineer's Report. According to Bulletin 17B, adjustments to the short-station mean and standard deviation are justified if the reduction in variance exceeds ten percent. The adjusted short-record frequency estimate for Matadero Creek (with a station skew of -0.07) is therefore:

$$\log Q = 2.6394 + (2.2747)(0.301782) = 3.3259$$

$$Q = 10^{3.3259} = 2{,}120\ cfs$$

The equivalent number of years of record (N_e) for this adjusted estimate, which is used subsequently for model verification, is calculated as:

$$N_e = \frac{N_1}{1 - \frac{N_2}{N_1 + N_2}\left(r^2 - \frac{(1 - r^2)}{(N_1 - 3)}\right)} = 63\ years$$

Rainfall-Runoff Model for San Francisquito Creek Watershed

Schaaf & Wheeler developed curve numbers and other basin parameters for the San Francisquito Creek watershed as part of the 2002 Engineer's Report, which are summarized in Table 8. The basin time of concentration is calculated using a modified USACE lag equation, which relates the Corps' definitions of basin lag and time of concentration. The USACE lag equation was originally based on their S-graph format for unit hydrographs. Based on model simulations, using the Corps lag equation along with its S-graph for the San Francisco District generally replicates synthetic unit hydrographs produced by Clark unit hydrograph parameters in HEC-1, when the time of concentration equals the modified basin lag. The equation for time of concentration is:

$$t_c = (.862)24N\left(\frac{LL_c}{\sqrt{S}}\right)^{0.38}$$

where N = USACE watershed "roughness" factor relating to density of drainage systems

L = maximum length from watershed divide to outlet in miles

L_c = length along main drainage path from outlet to point perpendicular to basin centroid in miles

S = effective slope along L in feet per mile

Table 8. Watershed Parameters for San Francisquito Creek at Stanford

Parameter	Value	Parameter	Value
Area	37.5 mi^2	N	0.08
SCS Curve Number (AMC II)	68	L	12.08 mi
Percent Impervious	5	L_c	5.30 mi
Mean Annual Precipitation	32 in	S	84 ft/mi
100-year, 72-hour Precipitation Depth	10.73 in	t_c	3.46 hours

The use of the Clark Unit Hydrograph within the District's unit hydrograph procedure requires a second parameter, the storage coefficient R. The ratio of R to the sum of R and t_c is generally between 0.5 and 0.9 for rural areas. Since the shape of the unit hydrograph is sensitive to the selection of R, additional work was performed for the 2002 Engineer's Report to evaluate the relationship between R and t_c.

The San Francisco District S-graph was used in the 2002 Engineer's Report to establish a unit hydrograph for the San Francisquito Creek watershed at the gage. At the time Clark's unit hydrograph was manipulated to replicate results by varying R according to the basin "N" for the same curve number. As demonstrated in Table 9, the ratio of R to t_c does not vary as long as the time of concentration is allowed to vary with basin "N" using the modified Corps lag equation. Basin "N" values range from 0.100 for completely undeveloped sub-basins to 0.025 for highly urbanized sub-basins.

Table 9. Calibration of Storage Coefficient R

Basin N	t_{lag} (hours)	t_c (hours)	Q_{peak} (cfs)	R	$\frac{R}{(R+t_c)}$
0.100	5.0	4.3	6,830	5.0	0.54
0.075	3.8	3.3	8,130	3.8	0.54
0.050	2.5	2.2	10,200	2.5	0.54
0.025	1.3	1.1	12,800	1.4	0.56

A constant relationship between t_c and R is used in the PAFB watershed model:

$$\frac{R}{t_c + R} = 0.54 \; or \; R = 1.17 t_c$$

Table 10 presents a summary of the watershed model calibration for antecedent moisture conditions. With the balanced 100-year precipitation pattern shown in Figure 12 (but for a mean annual precipitation of 32 inches), using an AMC of I¾ best replicates the flood-frequency characteristics of San Francisquito Creek. (The precise calibrated Curve Number to match the gaged 100-year discharge is 63.4, which represents an interpolated AMC of 1.77; however antecedent moisture conditions are generally calibrated to the nearest one-quarter of an integer value.)

Table 10. Calibration of AMC for Watershed Modeling

Return Period	AMC	Adjusted CN	Modeled Peak Discharge (cfs)	Variance from Gage
100-year (1%)	I¾	63	7,730	1.0%

It is noted that in the 2002 Engineer's Report, AMC was calibrated at a value of 1.55 (the adjusted Curve Number to replicate the 100-year discharge was 59). Since watershed parameters have not changed, this indicates that there is less rainfall in the updated 72-hour storm relative to the 2002 Engineer's Report.

Base Flow Recession

Constants for an exponential recession curve have been estimated using the hydrograph recorded at the San Francisquito Creek gage during the February 1998 storm event and adjusted for the HEC-HMS base flow methodology.

San Francisquito Creek's representative extreme-event base flow started at about 560 cfs (equivalent to 15 cfs per square mile); the recession threshold begins at 800 cfs (or 0.11 times the peak discharge); and the exponential decay constant, which is defined by HEC-HMS as the ratio of base flow at time t to the base flow one day earlier, is measured as 0.5 from the recorded hydrograph shown in Figure 14. The base flow constants are reflected in the AMC calibration summarized by Table 10.

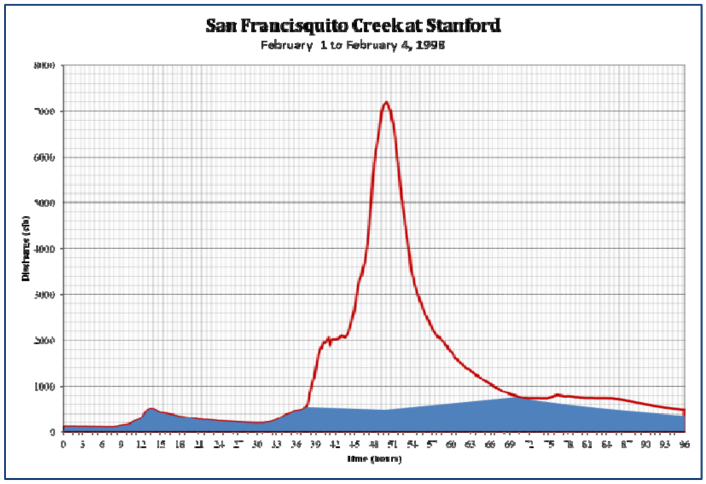

Figure 14. Base Flow Separation

Runoff Calculations

The calibrated AMC is used with the 72-hour balanced patterns for the PAFB watershed to produce 15-minute 100-year runoff hydrographs at the design points shown in Figures 7, 8, and 9 in HEC-HMS. With a 72-hour, 100-year storm more than 12,000 acre-feet flow into the Palo Alto Flood Basin over a seven day period. Table 11 provides a summary of inflow volume to the PAFB, with comparisons to previous studies.

Table 11. Inflow Volume to PAFB

Study	6-hour Volume (acre-ft)	24-hour Volume (acre-ft)	72-hour Volume (acre-ft)
1974 SCVWD	2,400	6,500	n/a
2002 Engineer's Report	2,500	5,700	9,700
2014 Study	2,400	5,800	9,900

Figure 15 shows the combined inflow hydrograph to the PAFB with the storm pattern at the PAFB superimposed. Table 12 provides peak discharges at the major design points in the watershed, comparing them to the peak discharges from the 2002 Engineer's Report at the same location. Variance ranges from zero to 7 percent, well within typical hydrologic accuracy.

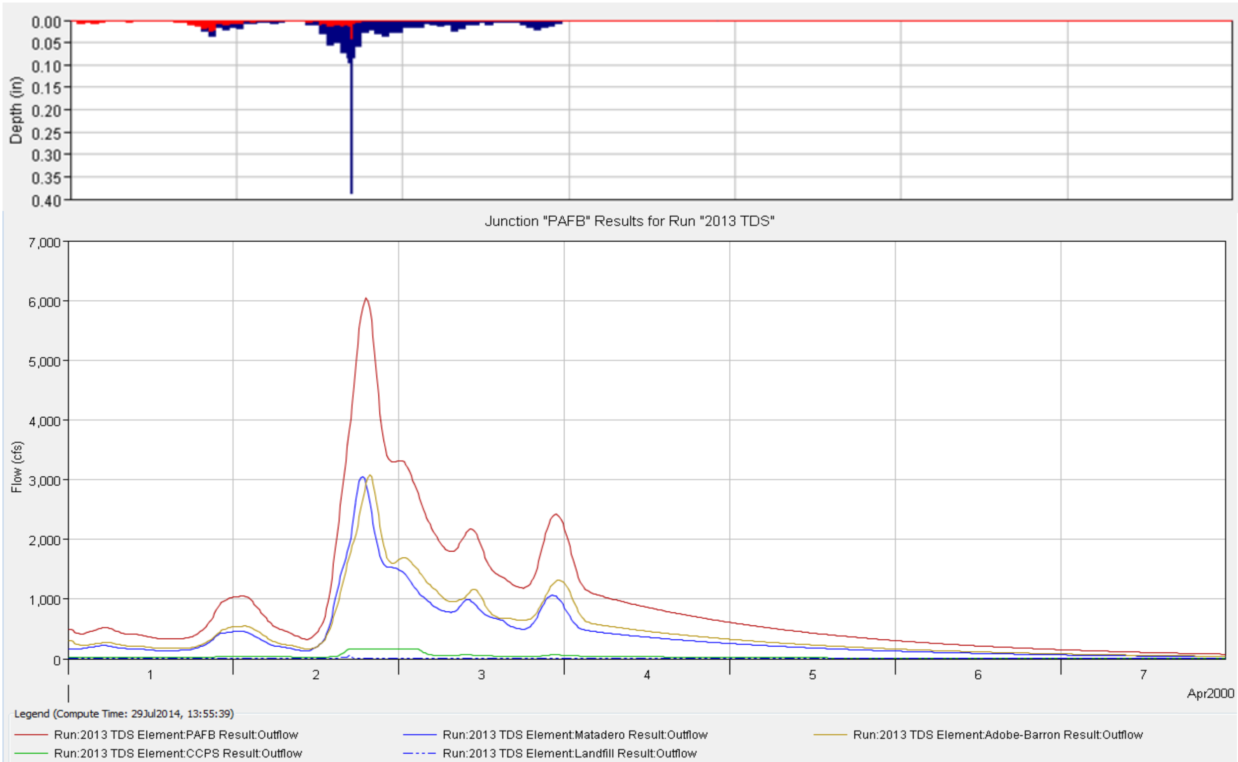

Figure 15. Combined Inflow Hydrograph to PAFB

Table 12. Summary of Watershed Discharges

Model ID	Location	100-year Discharge (cfs) 2002 Engineer's Report	Updated HEC-HMS
A	Matadero Creek at confluence with Arastradero Creek	1,145	1,070
B	Matadero Creek at confluence with Deer Creek	1,930	1,880
C	Matadero Creek at Matadero Bypass Diversion	2,030	1,970
D	Matadero Bypass	1,420	1,370
E	Barron Creek upstream from sediment basin	740	700
–	Diversion from Barron Creek to Matadero Bypass	530	500
–	Barron Creek downstream from diversion facility	160	155
F	Barron Creek at Alma Street	250	250
G	Matadero Creek at USGS gaging station (El Camino Real)	2,700	2,670
I	Matadero Creek at Railroad (Caltrain)	2,800	2,790
N	Adobe Creek at Interstate 280	2,500	2,370
P	Adobe Creek at Fremont Road	2,655	2,530
Q	Adobe Creek upstream from confluence with Barron Creek	2,910	2,800
U	Matadero Creek at Highway 101	3,060	3,050
W	Adobe –Barron Creeks at Highway 101	3,190	3,075
PAFB	Palo Alto Flood Basin combined inflow	6,040	6,040

Verification of Flow at Matadero Creek Gage

In the 2002 Engineer's Report design discharges for the Matadero Creek remediation project were based on weighting of different estimates of flow by the equivalent lengths of record used to generate the estimate. Reach discharges for Matadero Creek were based on a weighted estimate at the USGS gage with downstream additions for local storm drain runoff including the Matadero Pump Station. Discharge estimates at the gage location include the correlated flood-frequency analysis of 2,120 cfs with 63 years of equivalent record and the results of the updated HEC-HMS watershed model, which needs to be adjusted for Barron Creek diversion:

Modeled discharge at gage = 2,790 cfs – 500 cfs = 2,290 cfs

The difference in estimates is 8 percent, which is also well within typical error bounds for hydrologic analysis. Bulletin 17B proscribes the use of a ten year record length in the absence of an appraisal of estimation accuracy, and is adopted for the modeled discharge. If the estimates are re-weighted as in the 2002 Engineer's Report, the design discharge for Matadero Creek at the USGS gage location downstream of El Camino Real without Barron Creek diversions is:

$$Q = \frac{(2120)(63) + (2290)(10)}{73} = 2{,}145\ cfs$$

The discharge at El Camino Real is obtained by adding 500 cfs to 2,145 cfs, which is rounded to 2,650 cfs. This revised design discharge estimate is within 50 cfs (2 percent) of the design discharge used for the Matadero Creek long-term remediation project (2,700 cfs).

Direct Rainfall into PAFB

Appendix B contains spreadsheet calculations for the conversion of 15-minute rainfall depth over 72 hours into runoff volume using the SCS rainfall-runoff relationship. The volume of rain falling on the area bound by the PAFB levees, while relatively small, should also be accounted for in the PAFB model. For this portion of the analysis, the Soil Conservation Service (SCS, now Natural Resource Conservation Service or NRCS) rainfall-runoff relationship is used to convert the 100-year statistically balanced 72-hour storm pattern into direct runoff, all of which is assumed to flow into the PAFB instantaneously.

The volume of storm water runoff from a given precipitation event depends on a number of factors. In developing the SCS rainfall-runoff relationship, the total rainfall is separated into three components: direct runoff (Q), actual retention (F), and the initial abstraction (Ia) as shown schematically in Figure 16. The SCS equation is used to calculate the amount of direct runoff into the PAFB based on the following relationship:

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

P is precipitation

Ia is initial abstraction = 0.2S

S is the retention = 1000/CN – 10

CN is the curve number (97 for AMC I¾)

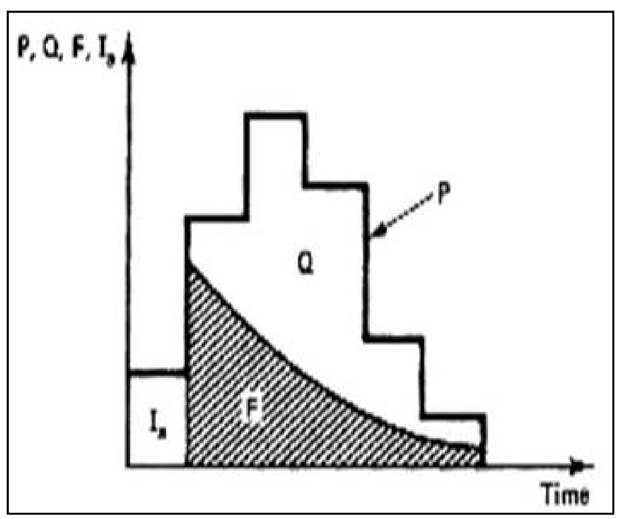

Figure 16. Separation of Rainfall (McCuen, 1989)

Tide Boundary Conditions

The Palo Alto Flood Basin stores the inflow depicted by the hydrograph in Figure 15 and discharges from storage into San Francisco Bay, which forms the downstream boundary of the Palo Alto Flood Basin model depicted in Figure 2. The tide gate structure was constructed to prevent Bay tides from filling the PAFB and to allow the discharge of stored runoff from the basin during ebb (low) tides. The elevation and timing of the tides during storm events plays a crucial role in the filling and draining of the PAFB and have a great impact on the extent and duration of peak water elevations in the basin.

Tide boundary conditions are established based on coincident probability analyses. Earlier analyses of PAFB operation relied on the assumption of average tides. As shown herein, this assumption is not based on a robust analysis of data collected over many years.

Astronomic Tides

A 19-year mean tide cycle is established for San Francisco Bay and other geographical locations on the West Coast. This cycle represents average tide heights over a specific period known as the tidal epoch, which spans the 19 years it takes for every possible combination of relative positions between the sun, moon and earth to occur. A mixed tide cycle predominates on the West Coast of the United States. This cycle consists of two high tides (one higher than the other) and two low tides (one lower than the other) each lunar day.

Based on calculations for these relative celestial positions, it is possible to predict tides for any day of the year at any time of day. *Astronomic tides,* created by the gravitational forces of the moon and sun acting on earth's oceans, are provided in tide prediction calendars. The mean tide cycle is simply the long-term average of astronomic tides. *Observed tides*, on the other hand, are actual tidal elevations recorded by National Oceanic and Atmospheric Administration (NOAA) gaging stations located throughout coastal areas. Table 14 provides the extreme points of the 19-year metonic cycle for the current tidal epoch (1983-2001) and the relevant datum conversions based on local NGS benchmark information and tide translation from the Presidio to the outlet of the PAFB.

A tide station was maintained for a number of years at the Palo Alto Yacht Harbor, but its data are not used because the harbor is not located in open water. Since the PAFB HEC-RAS model includes Mayfield Slough and the secondary slough that carry discharge between the tide gates and open water, the downstream open water boundary is represented by the adjusted tide cycle at the nearby Dumbarton Bridge.

Table 13. Mean Tide Cycle at Dumbarton Bridge

Tide[1]	19-year Mean at Presidio (MLLW)	19-year Mean at Dumbarton (MLLW)	19-year Mean at Dumbarton (NGVD)
Higher High (MHHW)	5.84	8.61	4.52
High (MHW)	5.23	8.00	3.91
Mean Sea Level (MSL)	3.12	4.68	0.59
Low (MLW)	1.13	1.26	-2.83
Lower Low (MLLW)	0.00	0.00	-4.09

1. Epoch data collected from NOAA website at http://tidesandcurrents.noaa.gov/stations.html?type=Datums

Establishing a Coincident Boundary Condition

Traditionally Mean Higher High Water (MHHW) has been used as the backwater condition where riverine (freshwater) runoff meets an estuarine (saltwater) body. However, evidence shows that mean tide elevations are not an appropriate boundary condition during storm events and tide elevations in San Francisco Bay are elevated (relative to predicted tides) during periods of heavy rainfall. Furthermore, the relationship between coincident tides and maximum annual runoff can be quantified and used in the model, providing for a more statistically correct solution than an arbitrarily selected tide condition.

Observations from the Storm of Record

The El Niño storm of February 2-3, 1998 provided an ideal event for examining potential correlations between runoff events and tide action. During that event stream runoff measured by local gages approached historic recorded levels and observed tides in San Francisco Bay were substantially higher than predicted. Figure 17 shows predicted and recorded tides in early February 1998 at NOAA's Golden Gate (San Francisco Presidio) gage. Recorded tides during the week of this runoff event were consistently higher (on the order of 2 feet) than the astronomic (predicted) tide heights due to storm surge. As a control, observed tide heights are compared to predicted tides six months later at the same station, using the same sets of data (Figure 18) during early August 1998, when there is very close agreement between the predicted and the actual tides and no rainfall. Both figures present tides on the local Mean Lower Low Water (MLLW) datum.

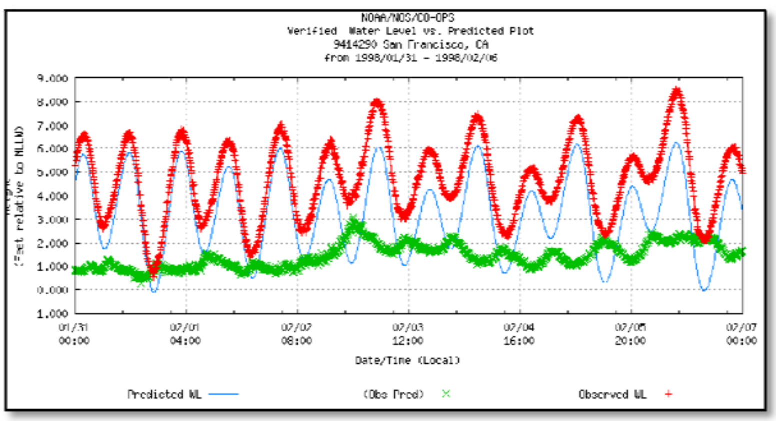

Figure 17. Impact of Storm Surge on San Francisco Bay Tide

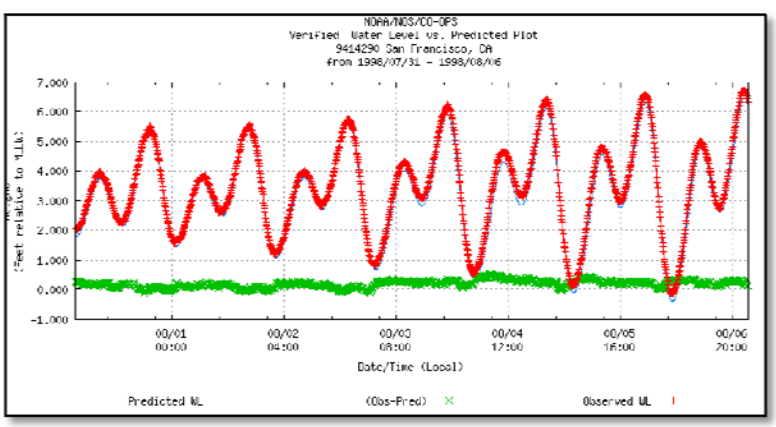

Figure 18. Lack of Storm Surge Effect during Summer Months

Historic tide records have been examined to see whether the phenomenon demonstrated in February 1998 at the Golden Gate occurred elsewhere in the Bay Area and during other heavy runoff events in the past. The observed phenomenon is not strongly dependent upon tide gage location, particularly within San Francisco Bay, and is exhibited during many historic storm events. From observed historical data, it appears that storm-related forces induce higher tides during rainfall events, and by extension, runoff events. NOAA refers to the term "inverse barometer effect", and defines it as higher tides that are caused by lower barometric pressures associated with winter storm systems. References to "storm surges", the meteorological effects of low barometric pressures and/or strong southerly winds, are also found in the literature.

Assessing the Conditional Probability of Coincident High Tide

To model an appropriate San Francisco Bay tidal cycle during a storm event of particular return period (with tides adjusted to the nearby Coyote Point Marina location), elevations for each critical point in the tide cycle are adjusted based on the one-percent conditional probability of coincident occurrence with the annual maximum discharge of San Francisquito Creek at Stanford, which represents the closest USGS stream flow gaging location with sufficient length of record for analysis; and this gage data is also used to calibrate the rainfall-runoff model. This procedure is as described by Dixon (1986), whose hypothesis was that high tide events tend to occur the same day as flood flow events using conditional probability:

$$P_{(x,y)} = P(x|y)\,P(y)$$

where P(x,y) is the probability of occurrence of x and y; P(x|y) is the probability of occurrence of x given y; P(y) is the probability of occurrence of y; x is tide elevation; and y is maximum annual peak discharge. Since we are interested only in annual maximum discharges, P(y) is one and the probability of joint occurrence, P(x,y), is equal to the probability of x given y.

Coincident Tides at Golden Gate

Tide cycle points are taken from fitted probability curves of data contained in Appendix C, using the median plotting position for every recorded tide extreme that occurred within 24 hours of the recorded maximum annual discharge. Figures 19 and 20 show the probability distributions for high and low tides, respectively; and Table 14 provides the values for each point on the tide cycle. Observed tide elevations at Golden Gate are translated to the Dumbarton Bridge by adding 2.6 feet to high tides and 0.1 foot to low tides.

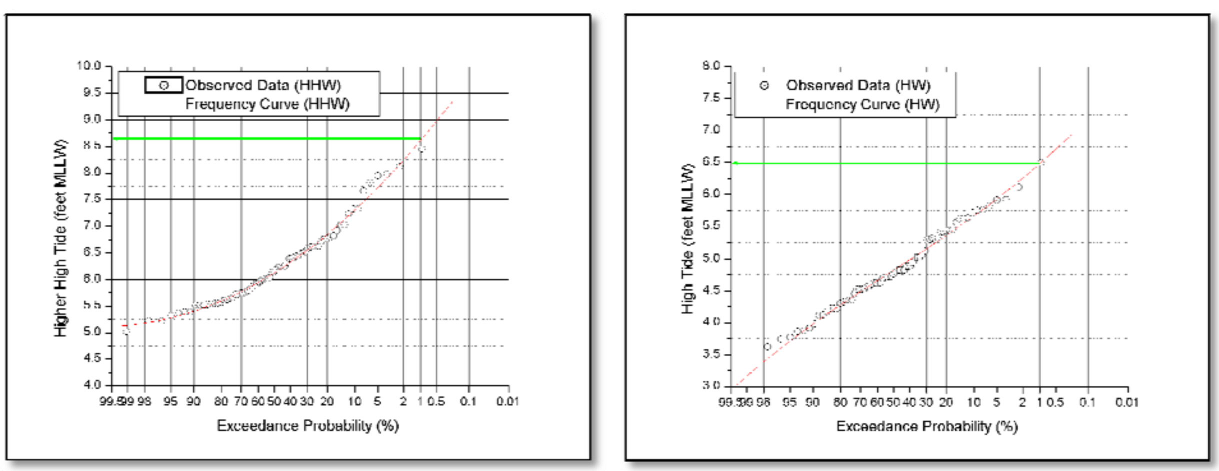

Figure 19. Conditional Probability of High Tides at Golden Gate

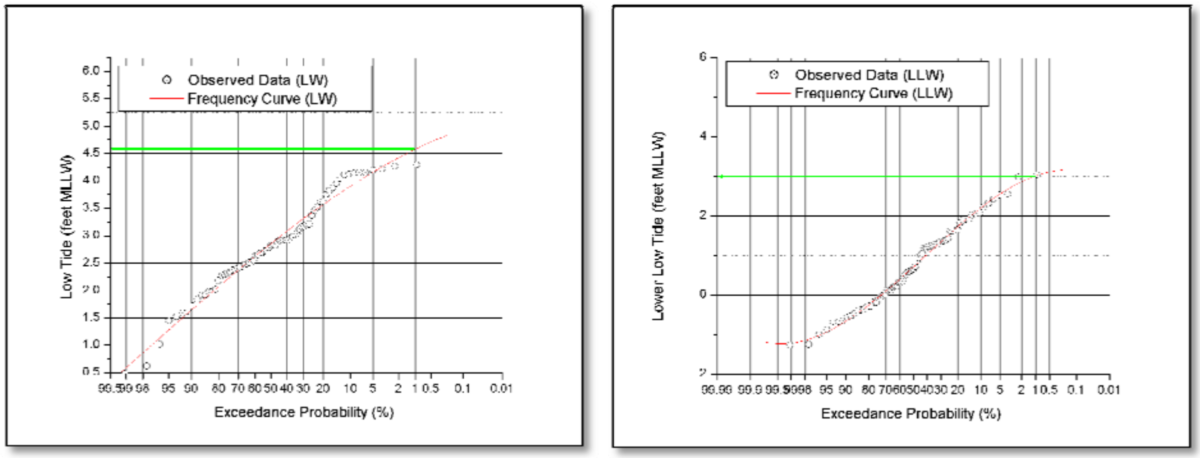

Figure 20. Conditional Probability of Low Tides at Golden Gate

Table 14. San Francisco Bay Boundary Conditions

Tide	100-year Coincident at Golden Gate (feet MLLW)	100-year Coincident at PAFB (feet MLLW)	100-year Coincident at PAFB (feet NGVD)	19-year Mean at PAFB (feet NGVD)
Higher High	8.7	11.3	7.21	4.52
High	6.5	9.1	5.01	3.91
Low	4.6	4.7	0.61	–2.83
Lower Low	3.0	3.1	–0.99	–4.09

Downstream Boundary Condition at PAFB

The coincident tide cycle points listed in Table 14 are used to produce a sinuous design tide cycle based on the timing of the USACE’s 19-year mean tide cycle for the Golden Gate Station. To translate tides from Golden Gate to the Dumbarton Bridge, high tide elevations are lagged 1.00 hour and low tide elevations are lagged 1.63 hours from the time of high tide at the Golden Gate. Figure 21 compares the observed February 1998 tide at the Golden Gate transposed to the NOAA Redwood City tide station (the closest tide station with verified observations) to the recorded tide at Redwood City, demonstrating the validity of this methodology, using the tide translation factors from USACE.

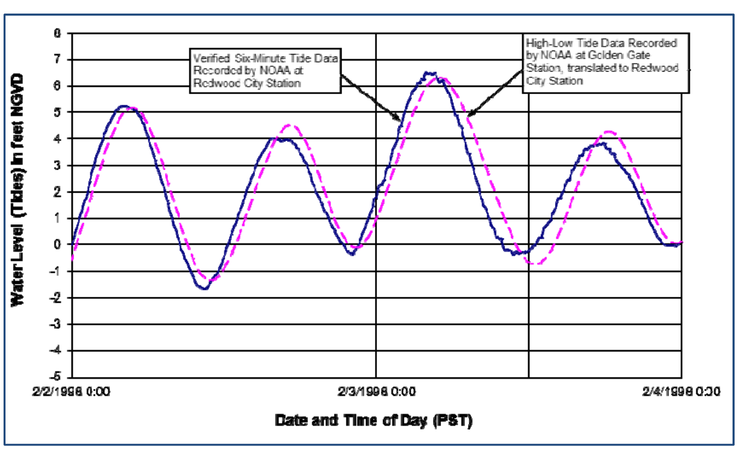

Figure 21. February 1998 Tides at NOAA Redwood City Station

Figure 22 shows the design tide cycle used as the downstream boundary condition for the HEC-RAS analyses. Observed tides at the tide gate outlet are also plotted, shifted to coincide with high and low tides. Design tide cycles published in the 2002 Engineer's Report, which are essentially validated herein with another 12 years of coincident record, are substantially different from the design tide cycles used in earlier studies, which are summarized in Table 15. Figure 23 compares the updated design tide cycle to observed tides at the flood basin outlet from February 1998 and design tides from the District's 1974 study.

With the exception of the Linsley-Kraeger study (1984), the updated design tide is similar to the 1974 and 1975 design tides, but only for the high water points of the tide cycle. A significant disparity is seen between the updated design tide cycle's low water points when compared to those used in earlier studies. As the boundary condition affects the operation of the PAFB, the most important point in the tide cycle is the ebb, or low tide, since its elevation impacts the ability of the tide gates to discharge flood flows from the PAFB.

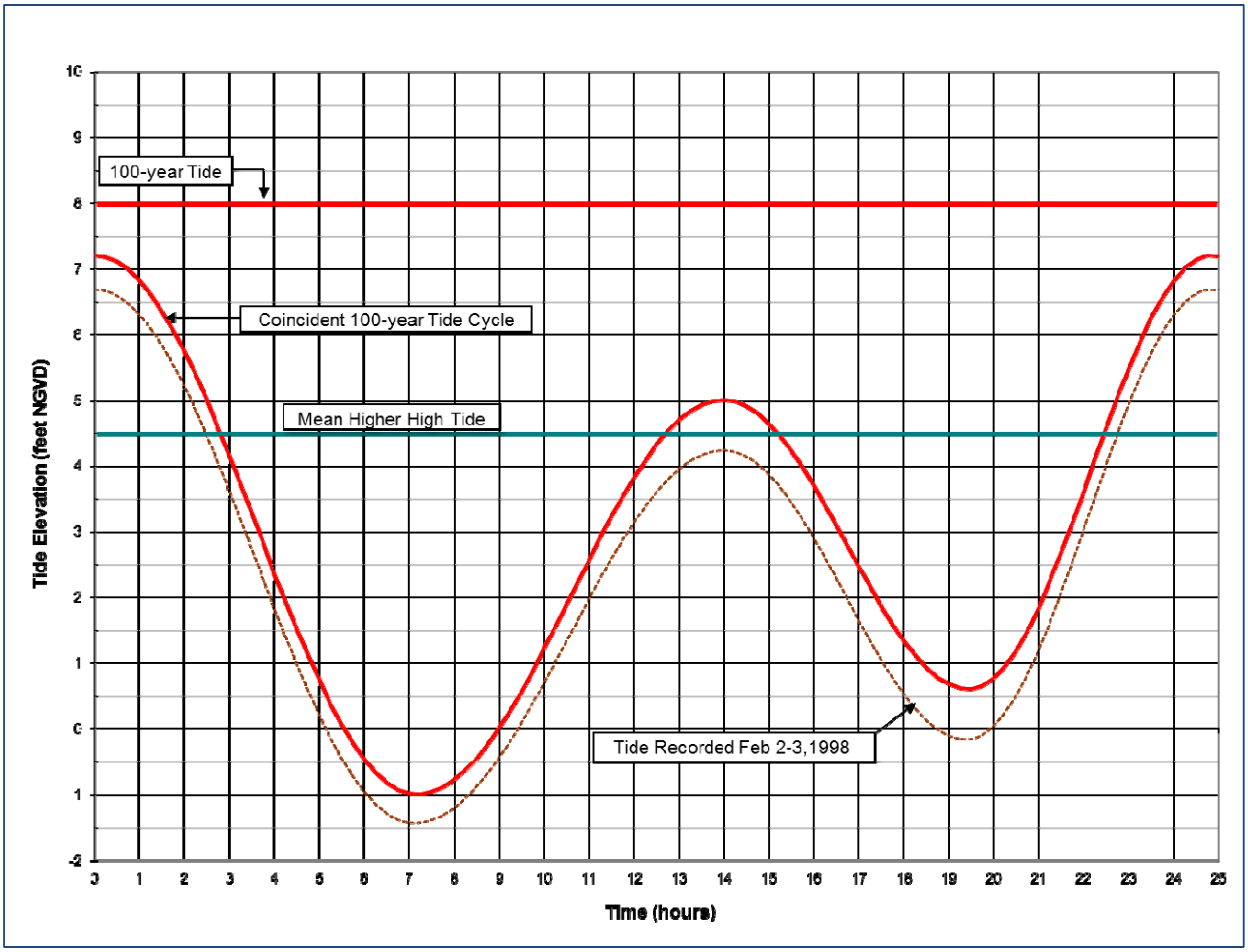

Figure 22. Design Boundary Condition at San Francisco Bay

Table 15. Comparison of Coincident 100-year Design Tide to Previous Design Tides

Tide	Updated Design Tide (feet NGVD)	2002 Engineer's Report (feet NGVD)	1974 SCVWD (feet NGVD)	1975 Water Resources Eng. (feet NGVD)	1984 Linsley Kraeger Assoc. (feet NGVD)
Higher High	7.2	7.2	6.7	6.8	3.0 to 4.0
High	5.0	5.4	4.0	4.4	1.5 to 2.3
Low	0.6	1.0	–2.0	–0.1	–1.6 to –0.7
Lower Low	–1.0	–0.5	–5.6	–4.8	–3.7 to –1.8

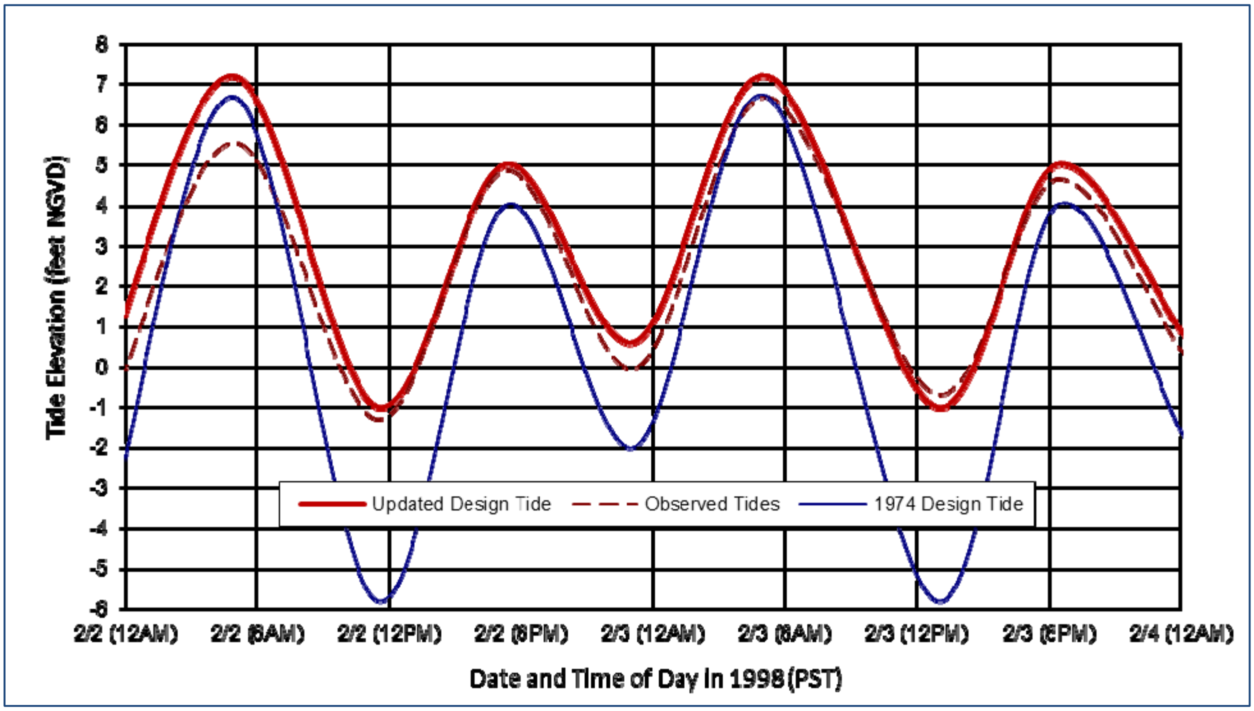

Figure 23. Tide Cycle Comparison

The timing of coincident tide elevations with the beginning of the storm event (rainfall) is a random process. Since there are not sufficient data to statistically analyze the impact of tide timing, a sensitivity analysis has been conducted to quantitatively assess the relative risk of achieving certain 100-year elevations within the Palo Alto Flood Basin. This analysis is fully documented in the next chapter.

With upstream boundaries (flow), downstream boundaries (tide), and the basin fully modeled with a storage-elevation curve connected to the slough reaches with a gated structure; PAFB operation can be evaluated.

Palo Alto Flood Basin Performance

The hydrologic and hydraulic models described in this document are used to evaluate Palo Alto Flood Basin Performance during a design 100-year storm event. With established planning objectives, the HEC-RAS basin model can also be used to assess potential mitigation measures.

Existing Condition Simulations

Using the updated data and assumptions presented herein, the performance of the PAFB under a design 100-year storm loading has been modeled. Results are presented in several formats.

Starting WSEL for Simulations

Unsteady HEC-RAS requires an initial condition of storage in the PAFB at the beginning of storm runoff. In no case would the level of water in the PAFB be below elevation −5 feet NGVD, because this is the invert elevation of the individual tide gates. At the time of aerial survey in 1999, the water elevation within the PAFB was −3 feet NGVD.

The initial water surface elevation has been set to −1.0 foot NGVD, which is equal to the lowest 100-year coincident tide. It is assumed that prior to the rainfall event, without inflow the tide gates are capable of evacuating the flood basin to the elevation of the lowest tide. Rainfall, runoff, flood basin elevation, and tide elevation observations made by the City of Palo Alto during the February 1998 event indicate that this is a reasonable assumption.

Depending upon the timing of the initiation of rainfall against the design tide cycle, the simulation can become unstable. In some instances it has been necessary to lower the starting water surface elevation as much as two feet. A sensitivity analysis indicates that the starting water surface elevation does not change the outcome of simulation (maximum stage in PAFB) when the starting water surface varies between −3.0 feet NGVD and −1.0 feet NGVD.

Tidal Timing Shift

Rather than adjust the timing of incipient precipitation against a static tide cycle, it proved easier to shift the tide cycle to perform the statistical analysis of PAFB flooding risk. After some iteration, a time shift increment of one hour is deemed as an adequate increment to assess PAFB operation.

Levee Containment Elevations

At a few locations along the PAFB containment levees, including the bike path adjacent to East Bayshore Road, the top of levee elevation is between 5 and 6 feet NGVD. Model simulations assume that calculated water surfaces elevations within the PAFB are fully contained, even when they exceed the minimum levee containment elevation, rather than let stored water spill from the flood basin when it exceeds that elevation. This assumption is made so that the relative effect of eventual alternative mitigation measures can be evaluated in full without capping maximum PAFB stage at a particular elevation based on current levee conditions.

100-year Water Surface Elevations in PAFB

Results of HEC-RAS simulations for existing conditions are presented graphically and in tabular form herein. Figure 24 shows a summary of PAFB operation for the tidal shift (29 hours) that produces the maximum 100-year water surface elevation in the basin, which is 6.0 feet NGVD. Stage is labeled on the left-hand y-axis; flow is on the right-hand y-axis. Tide elevation, PAFB elevation, total flow through the tide gate structure (eight gates), flow through each individual gate, and the total flow into the PAFB are all charted over a one-week period. Simulated gate operation does not allow reverse flow, and the minor negative flow spikes are considered a small model instability that does not affect the overall result.

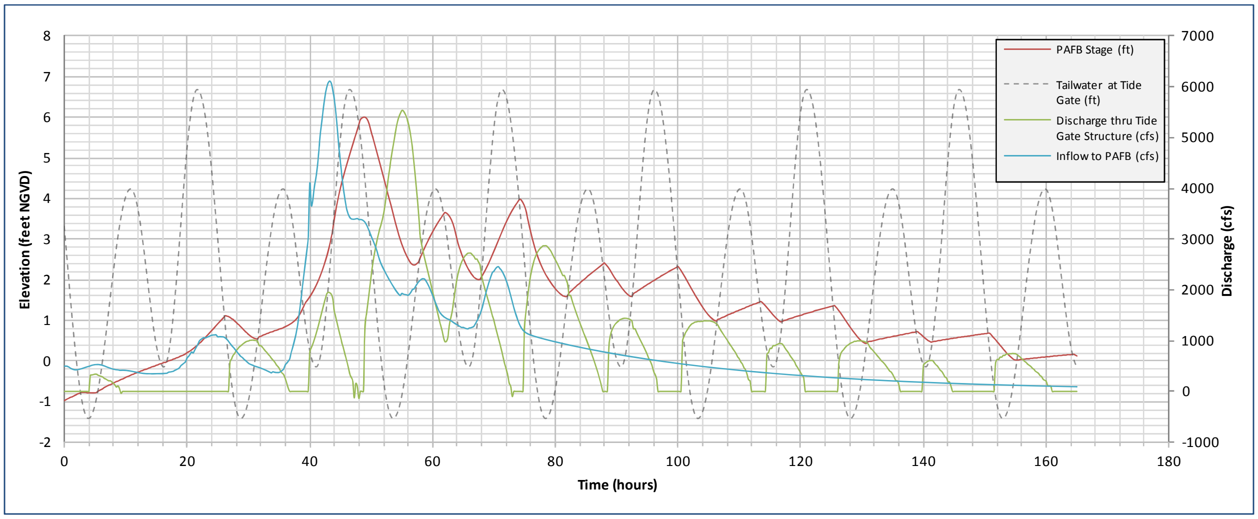

Figure 24. PAFB Operation with Maximum Stage based on Random Tidal Shift

Summary of Tidal Shift Impact

The random nature of relative timing between the beginning of the 100-year storm and the timing of the coincident 100-year tide cycle is captured by shifting the tide cycle in one hour increments until the tide cycle repeats, rerunning the PAFB simulation. Figure 25 presents the impact of this tide cycle shift on the maximum stage in the PAFB as well as the PAFB stage when inflow from Matadero Creek and Adobe-Barron Creek are at their respective peaks. Table 16 summarizes this same information numerically.

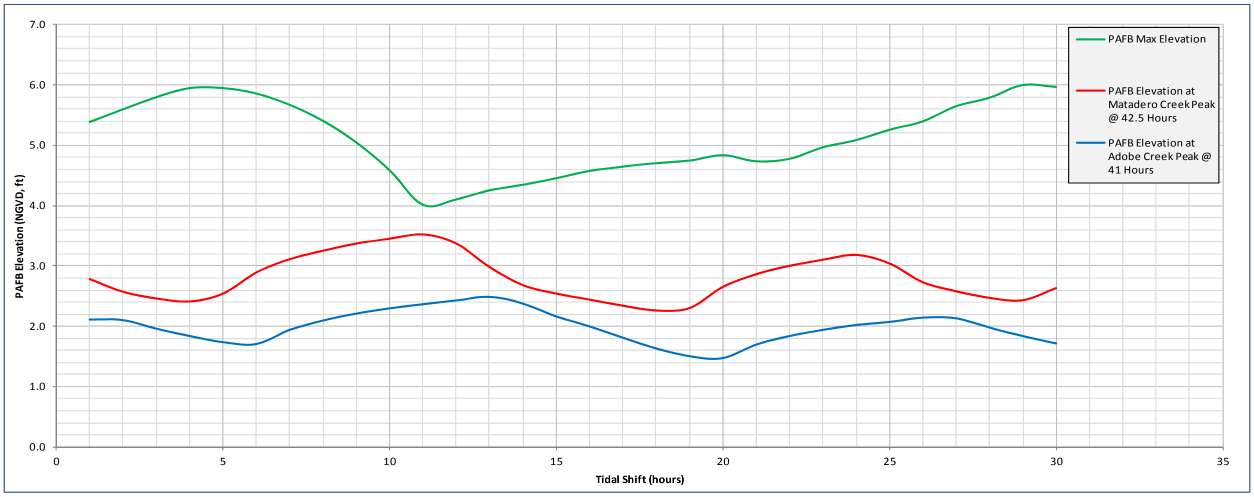

Figure 25. Tidal Shift Impact on PAFB Operation

Table 16. Summary of Tide Shift Impact on PAFB Stage

PAFB WSEL at Matadero Creek Peak (feet NGVD)	PAFB WSEL at Adobe Creek Peak (feet NGVD)	Tidal Time Shift in Hours	Maximum PAFB WSEL (feet NGVD)
2.78	2.11	1	5.39
2.57	2.10	2	5.60
2.46	1.96	3	5.80
2.41	1.84	4	5.95
2.54	1.74	5	5.95
2.89	1.71	6	5.86
3.11	1.94	7	5.67
3.25	2.09	8	5.41
3.37	2.21	9	5.05
3.45	2.29	10	4.59
3.52	2.36	11	4.02
3.37	2.42	12	4.11
2.98	**2.48**	13	4.26
2.68	2.37	14	4.35
2.54	2.16	15	4.46
2.44	2.00	16	4.58
2.34	1.81	17	4.65
2.26	1.64	18	4.71
2.30	1.51	19	4.75
2.65	1.48	20	4.84
2.86	1.70	21	4.74
3.00	1.84	22	4.78
3.10	1.94	23	4.97
3.18	2.02	24	5.09
3.04	2.07	25	5.26
2.73	2.14	26	5.40
2.58	2.13	27	5.65
2.47	1.98	28	5.79
2.43	1.84	29	**6.00**
2.63	1.72	30	5.97

Statistical Analysis of Random Tide Shift

The maximum PAFB 100-year water surface elevation of 6.0 feet NGVD depicted in Figure 24 represents a worst case combination of rainfall and tide cycle timing. As such it does not represent a “true” 100-year return period. Rather, the relative risk of various 100-year PAFB water surface elevations – which is analogous to a confidence limit – can be derived by plotting each elevation as a random occurrence on a probability scale. This is done in Figure 26 for the maximum stage, Figure 27 for the stage at peak Matadero Creek inflow, and Figure 28 for the stage at peak Adobe Creek inflow.

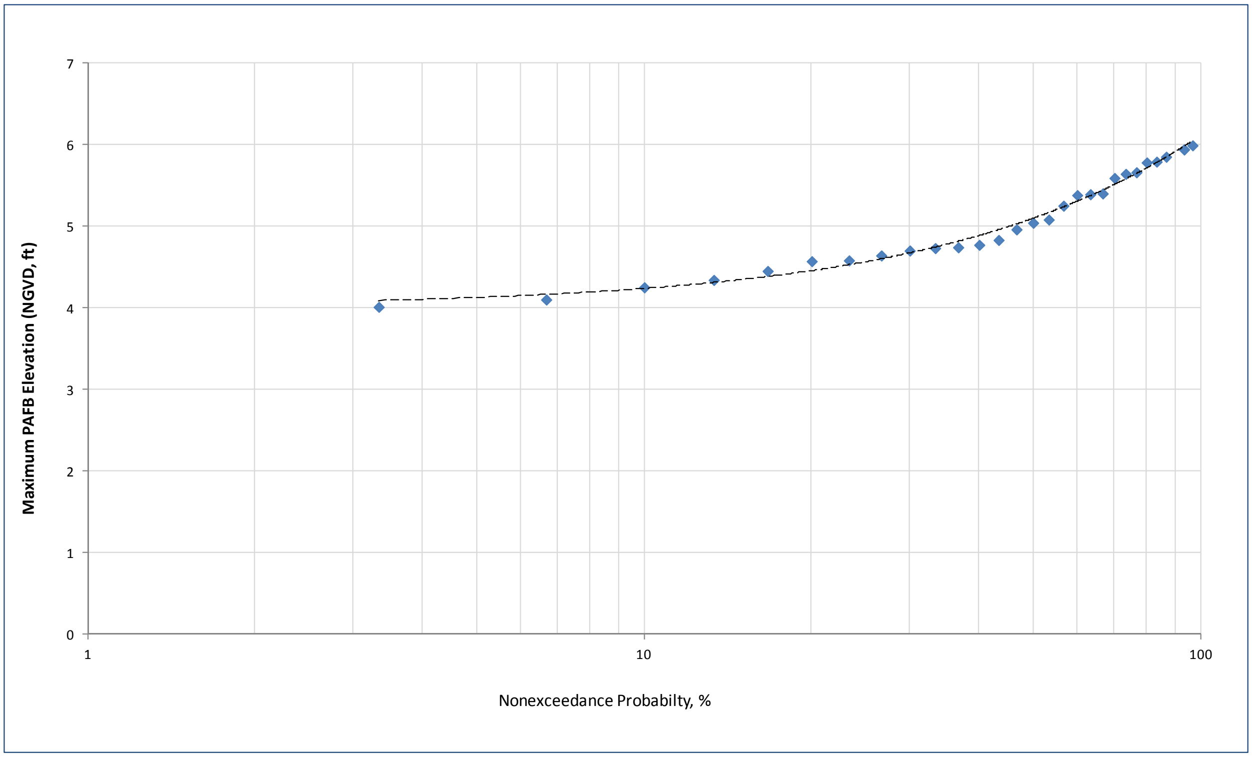

Figure 26. Non-exceedance Probability of Maximum 100-year PAFB Stage

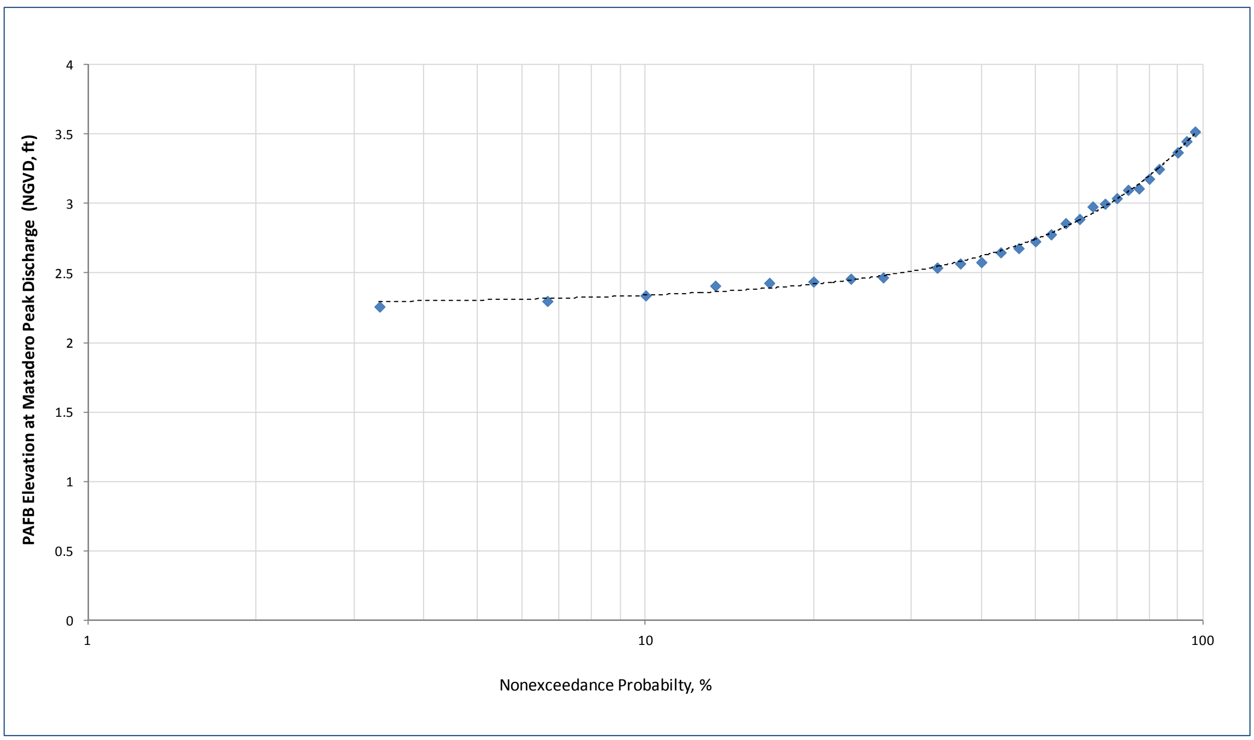

Figure 27. Non-exceedance Probability of PAFB Stage at Time of Matadero Creek Peak Discharge

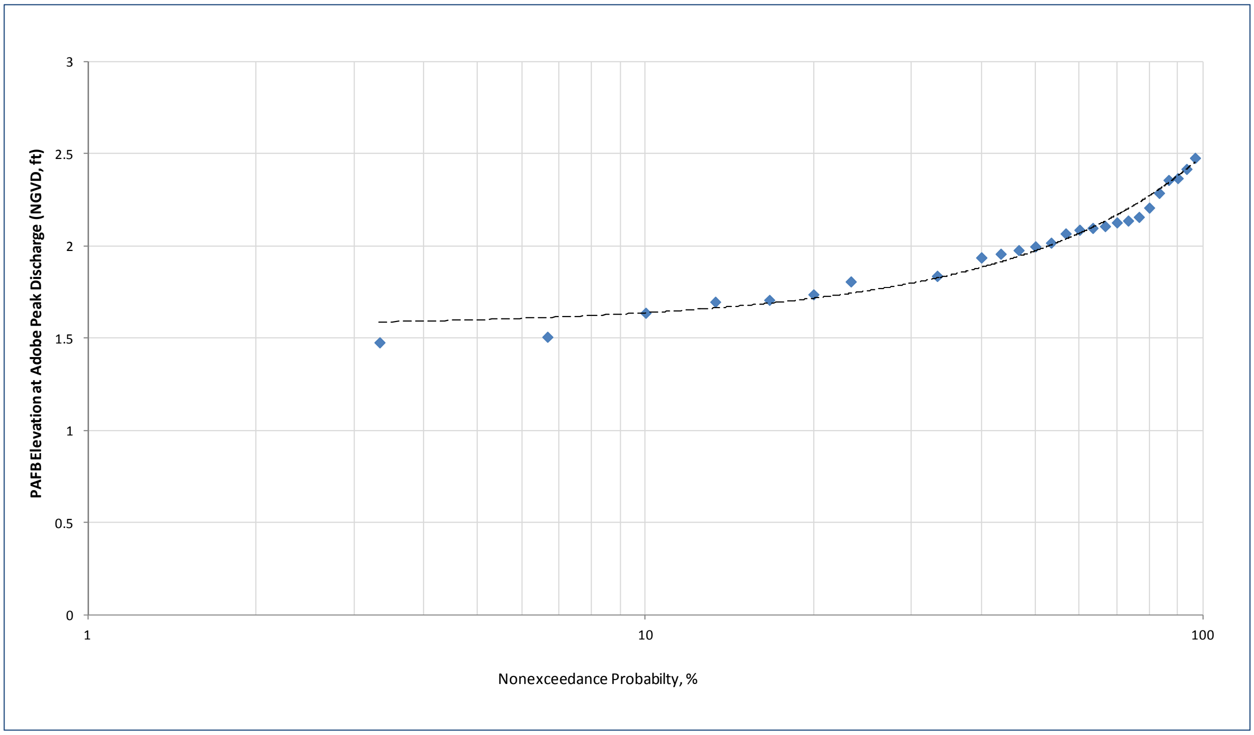

Figure 28. Non-exceedance Probability of PAFB Stage at Time of Adobe Creek Peak Discharge

Table 17 summarizes these statistics providing the 50-, 90- and 95-percent confidence limits for the 100-year PAFB water surface elevations of interest. The table provides the non-exceedance probability; that is, the confidence that a given water surface elevation will not be exceeded during the design 100-year event. A fifty percent confidence means that the given water surface elevation is just as likely to be exceeded as not.

The 90- and 95-percent confidence limits are provided since those statistical abstractions were used in the risk and uncertainty analysis of the Matadero Creek floodwalls, which assumed a maximum PAFB water surface elevation of 7.2 feet NGVD (coincident one-percent tide) and a PAFB stage of 4.6 feet when Matadero Creek inflow is at its peak, for risk-based backwater analysis. These assumptions remain conservative based on the updated statistics.

Table 17. Summary of PAFB Water Surface Elevation Uncertainty

Scenario	50% Confidence (feet NGVD)	90% Confidence (feet NGVD)	95% Confidence (feet NGVD)
Maximum WSEL in PAFB	5.1	5.9	6.0
WSEL in PAFB when Matadero Creek inflow is at its peak	2.7	3.4	3.5
WSEL in PAFB when Adobe Creek inflow is at its peak	2.0	2.4	2.5

For comparison, the coincident 100-year tide is 7.2 feet NGVD and the 100-year stillwater elevation is 8.0 feet NGVD.

Future Planning Scenarios

The Santa Clara Valley Water District has expressed that they may establish project planning objectives wherein the following information may be desirable:

- Maximum allowable water surfaces in the basin to achieve flood protection objectives for the lower reaches of Adobe, Barron, and Matadero Creek.
- Maximum allowable water surfaces in the basin as a function of return period to evaluate and protect different habitats that are found or could be created within the flood basin.
- Impacts to flooding levels caused by sea level rise scenarios, based on BCDC thresholds or other future regulation.
- Changes in PAFB water surfaces that result from various project plan scenarios such as the replacement of the tide gate structure, providing additional storage, and pumping.

With the number of variable inputs including inflow and coincident tide cycles, it is relatively difficult to directly determine the conditions that will produce a target water surface elevation. Rather, the updated PAFB model is run over a range of conditions to produce performance curves and probability distributions. For instance, the maximum water surface in the PAFB can be related to the number of identical gates installed in a rebuilt tide gate structure.

Sea Level Rise

Sea Level Rise (SLR) scenarios are adopted from recent projections from the National Research Council.[4] Sea level rise predictions are added directly to the coincident tide cycles developed herein, essentially modeling the rise in Bay tide cycle as a uniform vertical datum adjustment. Despite some evidence that the difference in the intertidal range (i.e. between MHHW and MLLW) may be widening along with the overall trend of rising seas, the California Climate Change Center "assumes that all tide datums, e.g. mean high tide and flood elevations, will increase by the same amount as mean sea level."[5]

Low and high range of the projections are both used to reflect the uncertainty bounds inherent in developing the projections and applying them to a single location. Table 18 provides a summary of the range of SLR projections contained in the 2012 NRC document.

Table 18. Summary of NRC Sea Level Rise Scenarios

Time Period	Low Range SLR (feet)	High Range SLR (feet)
2000 – 2030	0.13	0.98
2000 – 2050	0.39	2.00
2000 – 2100	1.38	5.48

[4] National Research Council, Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past Present, and Future, National Academies Press, Washington, 2012.

[5] California Climate Change Center, "The Impacts of Sea-Level Rise on the California Coast," May 2009, Page 9.

Sea Level Rise Impact with Existing Flood Basin Configuration

Maximum stage in the PAFB resulting from the SLR predictions is estimated from a worst case combination of rainfall and tide cycle timing with the existing tidal gates as performed for the existing conditions. Temporal shifts in the tide cycle on the order of one hour are manually performed to pinpoint when the highest PAFB stage could occur. Furthermore, the water surface elevations associated with random tidal shifts are plotted to evaluate the relative risk of the 100-year PAFB elevation under each SLR scenario in the form of probability distributions.

Table 19 provides a summary of the predicted maximum stage in the PAFB without levee overtopping for the predicted ranges of three SLR rise scenarios (2030, 2050 and 2100) assuming the configuration of the flood basin and tide gate structure are not changed. (Note the table lists SLR scenarios in ascending order according to the magnitude of sea level rise.) Probability plots for the range of sea level rise scenarios have been consolidated into a single graphic (Figure 29) to show how the variability in range of sea level rise projections dominates the variance in maximum PAFB stage.

Table 19. Impact of Sea Level Rise on Maximum PAFB Stage

SLR Scenario	Sea Level Rise (feet)	100-year Coincident Tide (feet NGVD)	Maximum PAFB Stage (feet NGVD)
Existing	n/a	7.20	6.00
2030 Low	0.13	7.33	6.16
2050 Low	0.39	7.59	6.56
2030 High	0.98	8.18	6.93
2100 Low	1.38	8.58	7.00
2050 High	2.00	9.20	7.67
2100 High	5.48	12.68	11.31

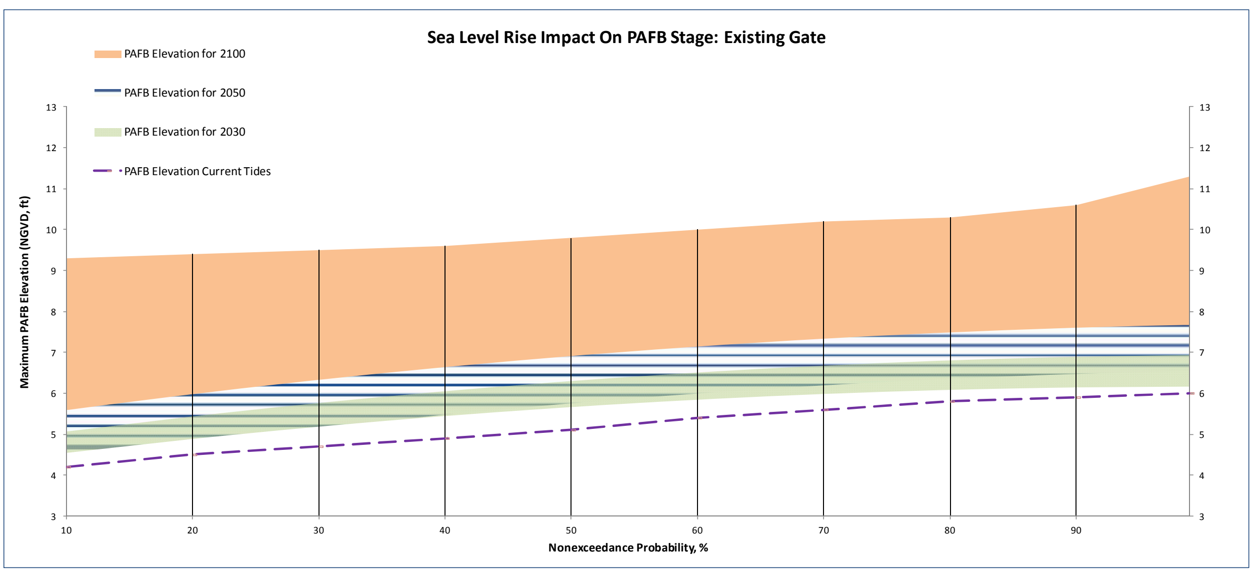

Figure 29. SLR Impact on Maximum PAFB Stage Confidence (Existing Gates)

Tide Gate Structure and Flood Basin Modifications

As depicted in Figure 29, the Palo Alto Flood Basin and lower reaches of Adobe, Barron and Matadero Creeks are vulnerable to the effects of possible future sea level rise. Since the Palo Alto Flood Basin tide gate structure is near the end of its useful life by exhibiting spalling and corrosion that has begun to compromise the concrete reinforcement (Figure 30), the feasibility of modifying the tide gate structure and/or flood basin to also meet the aforementioned planning objectives has been evaluated using the updated PAFB performance model. Potential modifications might include:

- Installing a greater number and/or larger tide gates or modifying their elevation.
- Dredging the flood basin to create additional volume.
- Connecting additional wetland areas to create additional volume.
- Pumped discharge of stored floodwaters.

Figure 30. PAFB Tide Gate Condition

Increasing the Number of Tide Gates

If and when the tide gate structure is rebuilt, there is ample space in the vicinity of the existing structure to build a new structure with additional gated openings to discharge stored flood water at higher rates for a given head differential across the tide gate than under existing conditions. Alternative rebuilt tide gate structures with 32 gates (double the total net discharge area), 48 gates (triple) and 64 gates (quadruple) have been modeled with various sea level rise scenarios.

For all modeling scenarios individual gates are assumed to be identical to the existing gates. That is, 5-foot by 5-foot rectangular openings with an invert elevation of -5.1 feet NGVD and a flap gate to prevent backflow. Modeling results are presented in the summary tables and figures.

Creating Additional Storage

Increasing the volume of storage is another means of reducing PAFB stage, whether alone or in combination with a rebuilt tide gate structure with additional discharge capacity. Without commenting on the feasibility of such an alternative, Figure 31 shows the Renzel Marsh directly connected to the PAFB across Matadero Creek. (Details of an overflow structure are not considered at this planning level.) Figure 32 presents the change in storage and Figure 33 shows the modified HEC-RAS model for this scenario. The mitigation area sandwiched between Renzel Marsh, Matadero Creek, and the Palo Alto Landfill is not included in the additional storage area to avoid mitigating the mitigation. Neither the potential environmental impacts to Renzel Marsh nor the regulatory hurdles that would need to be overcome are considered in this evaluation. Modeling results are presented in the summary tables and figures.

Figure 31. Overflow to Renzel Marsh to Create Additional Storage

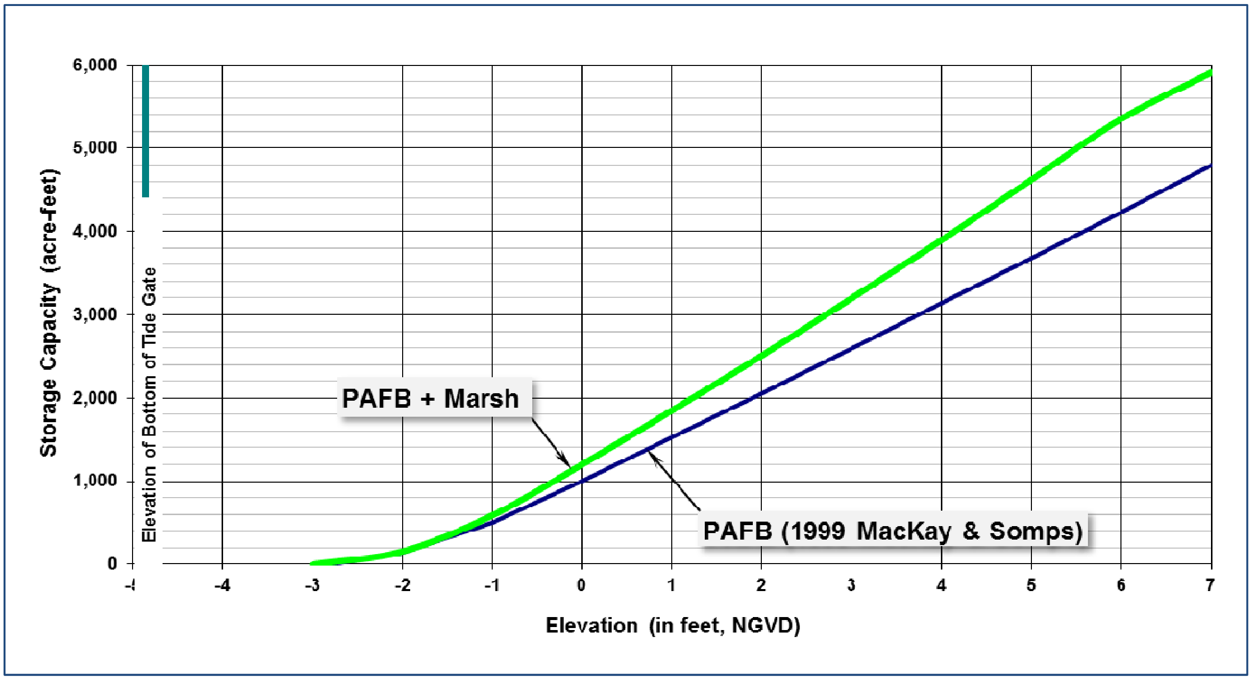

Figure 32. Change in Storage with Addition of Renzel Marsh

Figure 33. HEC-RAS Model with Renzel Marsh Storage Added

Gravity Remediation Alternatives

Storage volume can be increased in combination with additional tide gate discharge. Table 20 provides a comparative summary of maximum predicted PAFB stages for various rehabilitation alternatives and SLR scenarios. Figure 34 provides a side-by-side graphic comparison for this same information. Clearly none of the identified gravity options (additional gates or additional storage) can overcome high-range sea level rise projections, and increasing tide levels mute the benefits of any remedial alternative. Providing additional discharge through a rebuilt tide gate appears to be more effective than providing additional storage, and dimensioning returns are demonstrated when the number of gates is increased by a factor of more than two.

Assuming that doubling the number of gates would be considered for future tide gate modifications and increasing storage is not effective, confidence limits for maximum PAFB stage with 32 gates for the range of SLR scenarios are shown in Figure 35.

Table 20. Sea Level Rise Mitigation Using Gravity Remediation Alternatives

		Maximum Stage in Palo Alto Flood Basin (feet NGVD)						
		Gate Modification Only				Add Renzel Marsh Volume		
SLR Scenario	SLR (ft)	Existing Gate Structure	Double No. of Gates	Triple No. of Gates	Quadruple No. of Gates	Existing Gate Structure	Double No. of Gates	Triple No. of Gates
Existing	n/a	6.00	5.46	5.24	5.20	5.25	4.54	4.15
2030 Low	0.13	6.16	5.55	5.43	5.31	5.79	4.67	4.38
2050 Low	0.39	6.56	5.77	5.65	5.56	5.51	4.90	4.64
2030 High	0.98	6.93	6.27	6.10	6.05	5.92	5.49	5.53
2100 Low	1.38	7.00	6.61	6.50	6.41	6.26	5.84	5.68
2050 High	2.00	7.67	7.09	7.03	6.87	6.82	6.56	6.48
2100 High	5.48	11.31	11.32	11.20	11.20	11.22	11.11	11.0

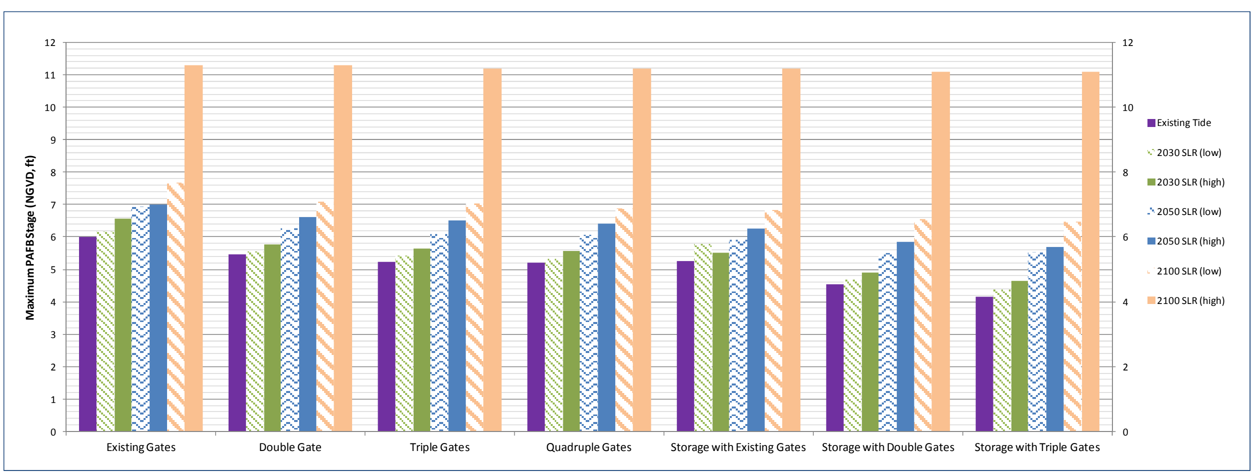

Figure 34. Graphic Representation of SLR Mitigation with Gravity Remediation Alternatives

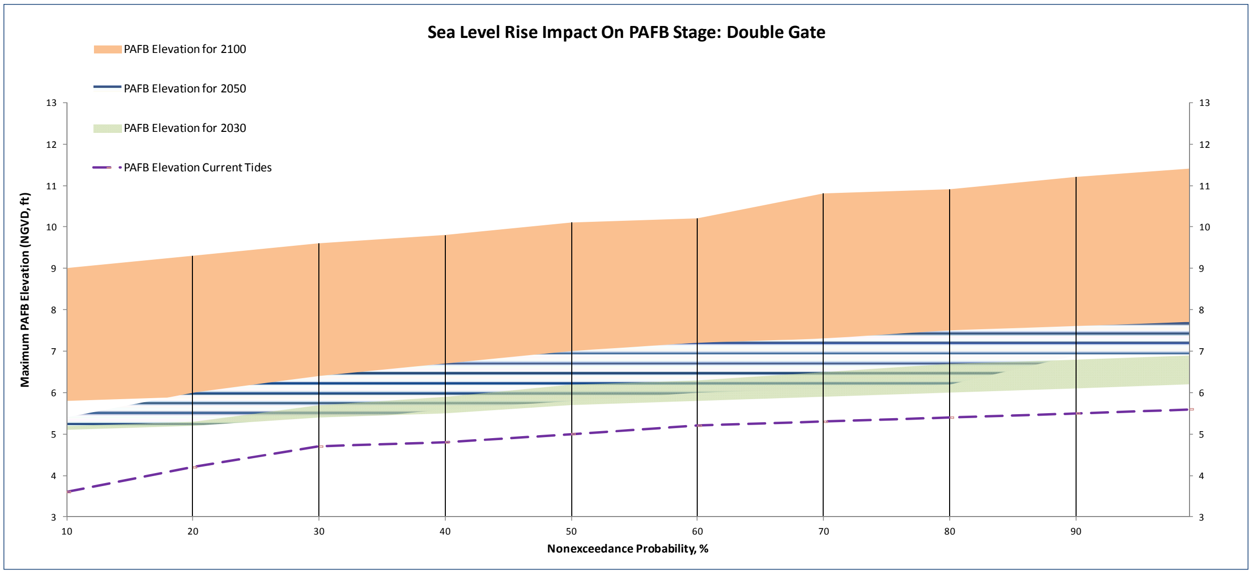

Figure 35. SLR Impact on Maximum PAFB Stage Confidence with Increase to 32 Gates

As an illustrative example of how to use this information, assume that the 2050 high-range SLR estimate of a 2 feet increase in tidal elevations is adopted as a planning horizon. If the PAFB configuration is not changed, SLR will cause rises in maximum PAFB water surface elevation from 6.0 feet NGVD to 7.7 feet NGVD, with the 90% confidence-level WSEL increasing from 5.9 feet NGVD to 7.6 feet NGVD (Figure 29). These elevations are problematic for the existing flood basin containment levees and interior areas of Palo Alto south of Highway 101.

Table 20 and Figure 34 can be used to ascertain the relative benefit achieved by various combinations of gravity remediation alternatives. For the same 2050 high-range SLR scenario, doubling the number of tide gates reduces the maximum PAFB WSEL by 0.58 foot from 7.67 feet NGVD to 7.09 feet NGVD. An additional 0.06 foot reduction could be achieved by tripling the number of gates, but that extra expense does not seem warranted. Even under existing tide conditions, tripling the number of gates reduces the maximum flood basin stage by less than 3 inches more when compared to doubling the number of gates.

Adding Renzel Marsh volume to the PAFB appears to provide more efficient reductions in stage than increasing the number of tide gates. With additional flood basin storage, rising sea levels would not create truly problematic maximum PAFB stages until the 2100 high SLR scenario.

If the goal is to maintain existing 100-year flood basin performance, but against rising tides due to climate change, doubling the number of tide gates would stem the deleterious predicted high-range tide increase until about 2030. Additional gravity remediation alternatives such as adding even more tide gates or adding Renzel Marsh storage would not substantially change this prediction. Any further increase in tidal stage at the downstream boundary of this system would require mechanical pumping to keep 100-year PAFB flood stage from exceeding its current maximum.

Pumping

At some point in the future gravity alternatives to decrease PAFB stage will no longer be sufficient to mitigate sea level rise. The only remaining alternative is to pump the stored flood water against high tide. The effects of adding pumping capacity to the existing system on the maximum PAFB stage (with 90 percent confidence) for the existing tide gate structure configuration and a rebuilt structure with twice as many tide gates are presented in Figures 36 and 37.

Ultimate required pumping capacity to meet various target maximum water surface elevations can be somewhat reduced if a new tide gate structure with 32 gates is constructed, but as is the case with gravity remediation alternatives there is a decreasing benefit with increased sea level rise. For example, to hold maximum flood stage to 5.9 feet NGVD (the existing 90% confidence limit) with the 2030 high-range SLR scenario the 400 cfs pumping plant required with the existing tide gate configuration could only be reduced to 350 cfs if the number of tide gates is doubled. Other scenarios can be examined to evaluate the efficacy of adding tide gates in addition to pumping. The 2100 high-range SLR curves are not shown, but a 2,200 cfs pump station would be required to maintain existing PAFB stage. This represents a pumping rate equivalent to more than 40 percent of the combined peak inflow. At this level of sea level rise pumping is essential to provide interior flood protection during extreme runoff events.

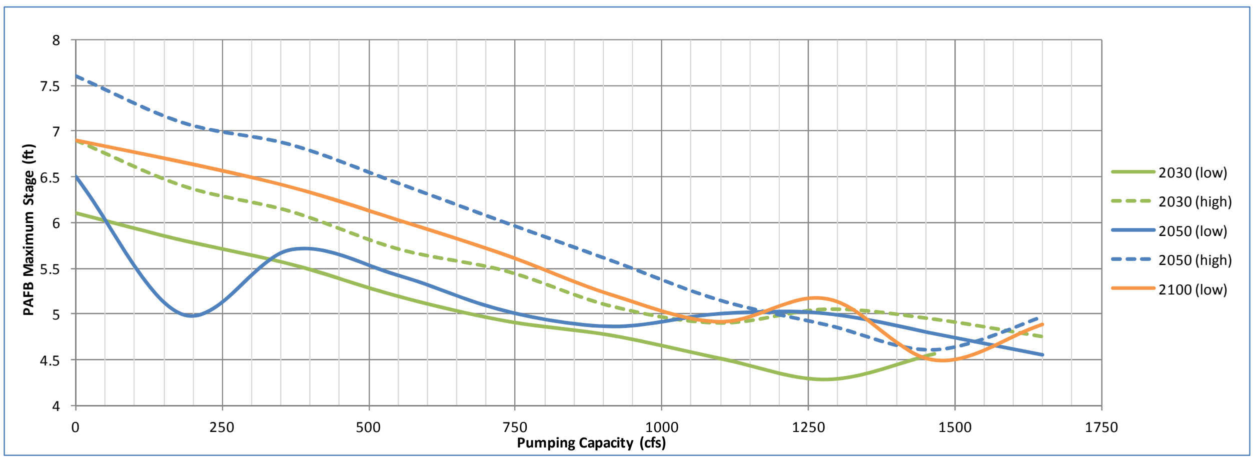

Figure 36. Pumping Capacity Required to Meet Target WSELs in PAFB w/ 16 Tide Gates

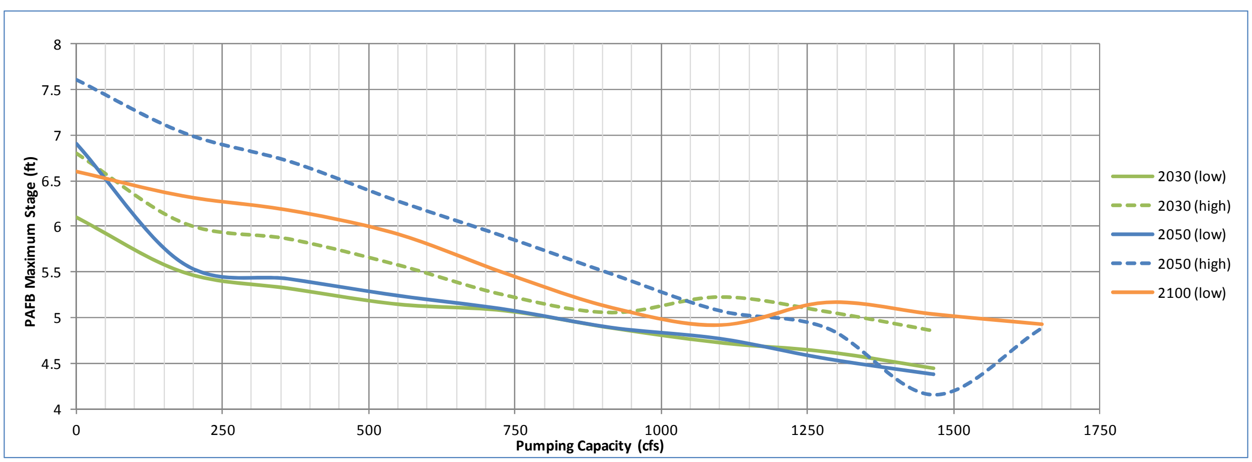

Figure 37. Pumping Capacity Required to Meet Target WSELs in PAFB w/ 32 Tide Gates

Conclusions

The functionality of the Palo Alto Flood Basin is driven by the coincident tide cycle. Maximum 100-year PAFB stage with 90 percent confidence is less than than the coincident 100-year tide in San Francisco Bay, meaning the PAFB has sufficient storage volume for creek runoff during high flow events. The basin provides additional flood protection by controlling starting backwater conditions during the peak discharges of Matadero Creek and Adobe Creek and protects those waterways from direct exposure to San Francisco Bay tides.

With higher low tides than originally accounted for in the basin design, the basin may be too small, depending upon the target maximum stage. The 90 percent confidence limit of the maximum one-percent stage probably exceeds PAFB containment elevations adjacent to East Bayshore Road, and there is less than one foot of containment freeboard above the 50 percent confidence limit of the maximum one-percent stage. Since antecedent storage in the PAFB for the 100-year event is predicated on the storm-surcharged mean lower low tide, dredging the basin below an elevation of about -1.0 foot NGVD will not have any impact on PAFB operation. Furthermore adding discharge capacity during low tide periods by installing additional tide gates appears to be as effective as adding storage volume.

Alternative mitigation measures include rebuilding the tide gate structure with additional and/or larger gates to discharge more flow during the shorter periods of low tide, until rising tide levels force the use of mechanical pumping. If the higher range of predicted sea level rise comes to fruition, not only will outboard levees need to be substantially increased in elevation, but a pumping facility with 2,200 cfs capacity may ultimately be needed for interior flood protection. This would be a facility with something like 6,000 installed horsepower and could cost on the order of $50 million to construct.

Appendices

For convenience appended material is provided digitally under separate cover. Appendices include the following data.

Appendix A. PAFB Watershed Modeling Parameters

Appendix A includes watershed sub-basin delineation; mean annual precipitation at the centroid of each sub-basin; lag parameters including length, length to centroid, basin slope, and basin "N" value; soil data, curve number estimation, and percent impervious cover by sub-basin; stream routing parameters; and the unitized urban storm drain routing curves.

Appendix B. Calculation of Runoff from Precipitation Directly Falling over PAFB

Appendix B contains spreadsheet calculations for the conversion of 15-minute rainfall depth over 72 hours into runoff volume using the SCS rainfall-runoff relationship. Incremental runoff volumes are converted into flow rates for input into the HEC-RAS model as direct inflow into a storage basin by dividing the 15-minute runoff volume by 900 seconds to compute the equivalent constant discharge over 15 minutes that would result in the 15-minute incremental direct runoff volume.

Appendix C. Coincident Tide Data

Appendix C contains spreadsheets that match coincident tide data during the day of the annual peak discharge for San Francisquito Creek, which is ranked using the Median Plotting Position to produce joint probability statistics for the higher high, high, low, and lower low tidal flood elevations.

Appendix D. HEC-HMS and HEC-RAS Models