HYDROLOGY

This section presents a discussion of the hydrologic analysis completed for the Sausal Creek Watershed. The goal of the analysis was to develop a watershed model that would simulate the rainfall-runoff processes for the 1-year through 100-year storm events over the watershed. The model was developed at a level of detail that would allow for the evaluation of measures designed to reduce the peak discharge to Sausal Creek and its tributaries.

The results of the hydrologic analysis provide flow rates and flow duration data for Sausal Creek and each of its major tributaries. This flow from the watershed model was used as input to a hydraulic model to determine the flow characteristics along the creek system. This section contains the results of the hydrologic analysis completed by Hydrologic Systems, Inc. (HSI) and revised and completed by NewFields River Basin Services, LLC (NRBS).

Watershed Delineation

The Sausal Creek watershed drains from the Oakland Hills to the San Francisco Bay. The total watershed area is approximately 4.5 square miles. The upper elevation of the watershed is approximately 1,525 ft. NGVD and the lower elevation is approximately sea level or 0 ft. NGVD (Figure 2).

To accurately define the runoff characteristics of the Sausal Creek Watershed, the watershed was divided into 33 separate sub-basins. The sub-basins were defined through common runoff characteristics that were distinct from adjacent basins. Figure 23 is a plan view of the watershed showing the location and extent of the different sub-basins. The acreage of each sub-basin is provided in Table 4. Storm drains can redirect flow from the boundary of one watershed into a different one. This effect occurs along the Sanborn Road area of Sausal Creek drainage where some areas are now draining into the adjacent Peralta Creek watershed.

Precipitation

Methodology

There are a variety of precipitation gages in the vicinity of the Sausal drainage. To develop precipitation data sets for the 2-, 5-, 10-, 25-, and 100-year, 24-hour storm events, a long-term hourly precipitation gage is required. There was no single gage that accurately reflected the precipitation across the watershed. Therefore, a precipitation dataset needed to be created from data collected at gages near the watershed. Figure 24 is a map showing the location of precipitation gages that were identified in the vicinity of the project.

Figure 23: Hydrologic Sub-Basins of the Sausal Creek Watershed

Sausal Creek Watershed Plan

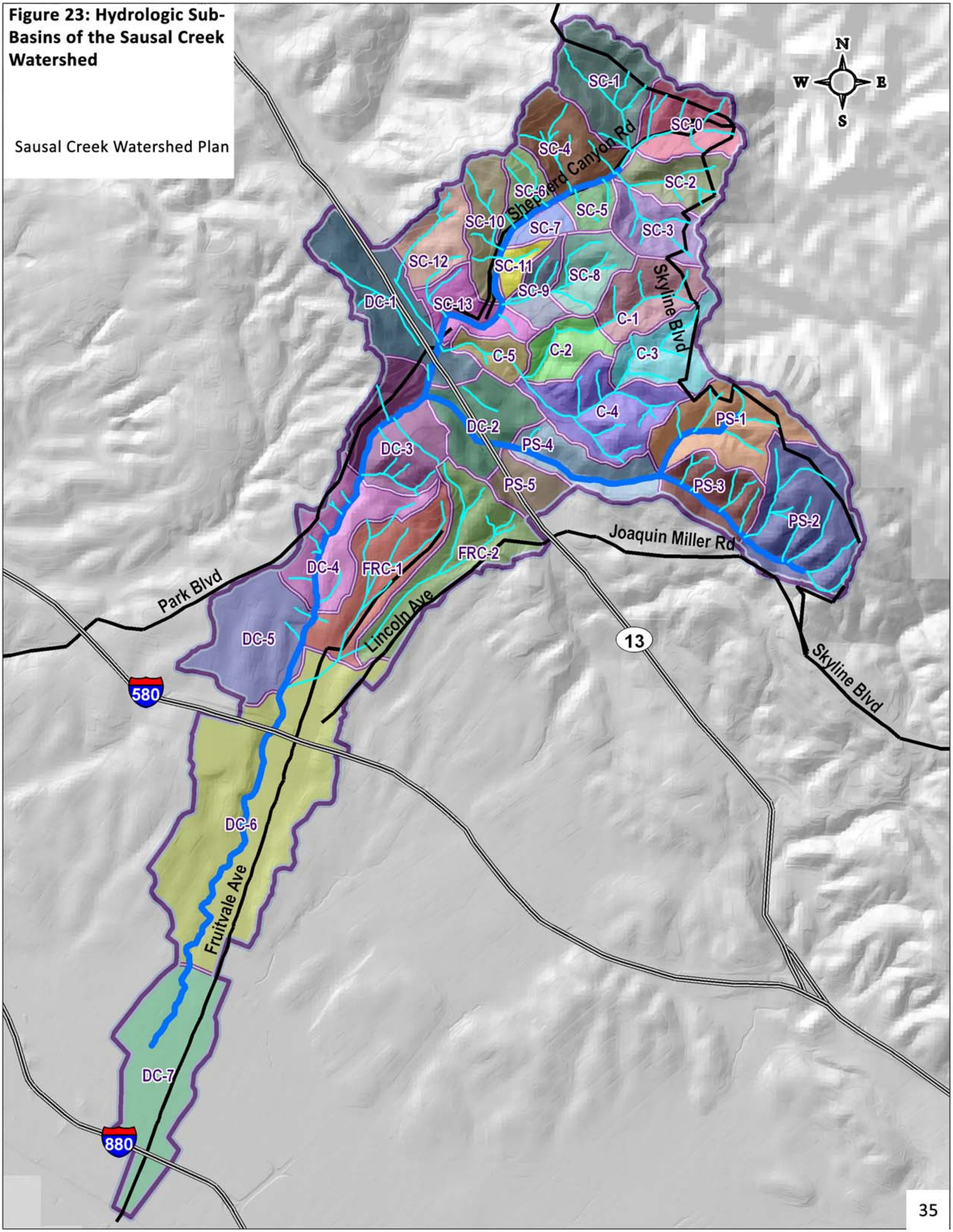
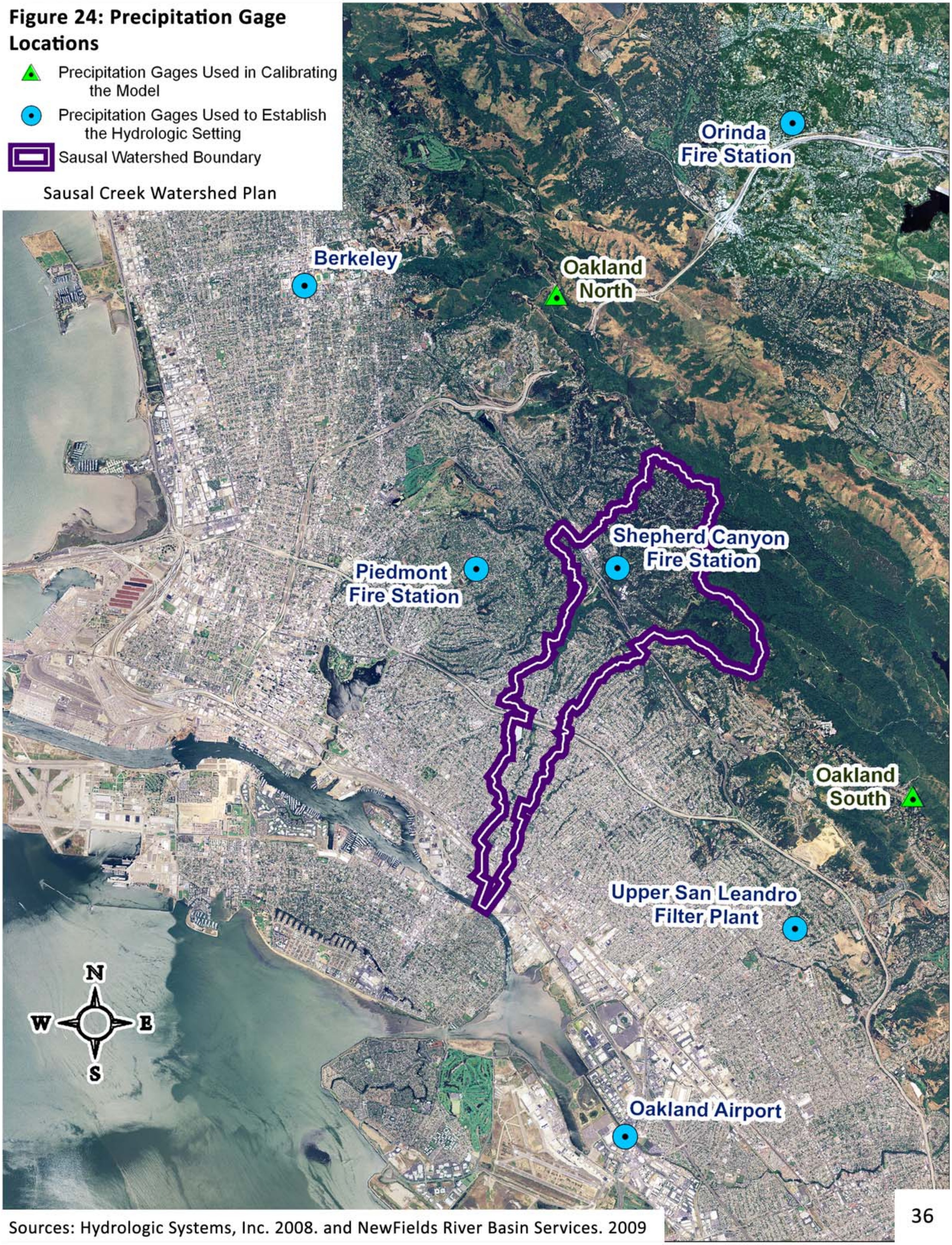


Figure 24: Precipitation Gage Locations



Sources: Hydrologic Systems, Inc. 2008. and NewFields River Basin Services. 2009

Table 4: Sausal Creek Watershed Sub-Basins Used in Hydrologic Model

Sub-Basin	Sub-Basin Name	Total Area (ac)	Total Area (sq mi)
Shephard Creek	SC-0	72.9	0.114
	SC-1	100.3	0.157
	SC-2	54.5	0.085
	SC-3	56.6	0.088
	SC-4	77.9	0.122
	SC-5	23.0	0.036
	SC-6	34.0	0.053
	SC-7	19.2	0.030
	SC-8	51.9	0.081
	SC-9	31.9	0.050
	SC-10	56.1	0.088
	SC-11	18.8	0.029
	SC-12	55.7	0.087
	SC-13	53.5	0.084
Cobbledick Creek	C-1	68.3	0.107
	C-2	45.6	0.071
	C-3	67.5	0.105
	C-4	105.3	0.164
	C-5	27.7	0.043
Palo Seco Creek	PS-1	109.2	0.171
	PS-2	147.5	0.230
	PS-3	64.4	0.101
	PS-4	65.9	0.103
	PS-5	35.9	0.056
Dimond Canyon	DC-1	124.0	0.194
	DC-2	80.4	0.126
	DC-3	108.3	0.169
	DC-4	103.6	0.162
	DC-5	119.9	0.187
	DC-6	387.1	0.605
	DC-7	173.7	0.271
Whittle Creek	FRC-1	107.8	0.168
	FRC-2	129.3	0.202
	Total	**2,777**	**4.34**

Precipitation varies across the San Francisco Bay Area, both in the east to west direction as well as north to south. Precipitation also typically varies with elevation, with precipitation increasing as the elevation increases. The winter season storms that produce the greatest amount of precipitation typically come out of the northwest. As storms move to the southeast, the orographic effect of the Oakland Hills increases the precipitation intensity. To account for this effect, the precipitation gage must be representative of the location, elevation, and orientation of the Oakland Hills. The precipitation data from an adjacent gage has to be adjusted to reflect the conditions specific to the Sausal Creek watershed.

Precipitation gages in the vicinity of the watershed include: Oakland Airport, Upper San Leandro Filter Plant, Piedmont Fire Station, and Orinda Fire Station, and Berkeley (Table 5, Figure 24). Another gage is located at the Shepherd Canyon Fire department, but it only collects daily precipitation data. This gage is useful because it is located in the center of the Sausal Creek Watershed, and provides a good estimate of annual precipitation. The remaining gages were either too far away from the project site, or were located at elevations that were not representative of the watershed.

Of the 6 available gages, the most appropriate gage was the Upper San Leandro Filter Plant Gage (USLFP), which is located approximately 2.6 miles south of the watershed. For hourly precipitation, this gage has a period of record from 1945 through 1989, providing a total of 45 years of record. From 1989 to present, only daily precipitation totals have been recorded.

The Mean Annual Precipitation (MAP) for the period of record at the Upper San Leandro Filter Plant is 24.37 inches. Figure 25 is a time-series plot of the annual precipitation for the Upper San Leandro Filter Plant gage. Figure 26 is a plot of the Mean Annual Precipitation at each gage as a function of the gage elevation. The data for the USLFP gage was adjusted to reflect the elevation of the Sausal Creek Watershed. Three separate modifications to the precipitation data were preformed. The first modification adjusted the precipitation data to reflect the elevation of the upper basin. Two additional adjustments were developed for the middle and lower basin elevations. For the final model, three separate precipitation datasets were used for the watershed.

A tabulation of the annual data from the USLFP gage has been provided in Appendix A. Table 6 shows the precipitation intensity-duration data for the upper, middle, and lower basins. Figure 27 is a plot of the frequency distribution for the 1-hour, 2-hour, 3-hour, and 6-hour continuous precipitation volumes. The return frequencies were computed using a Log Pearson Type III analysis of the peak annual precipitation values from the USLFP gage. The details of the Log Pearson Analysis are provided in Appendix A.

Table 5: Precipitation Gages Used in the Hydrology Analysis

Name	Gage No.	Operator	Latitude	Longitude	Elevation (ft.)	Type	From	To	Source
Orinda	E40 7661 00	Contra Costa Fire	37.895	-122.170	700	Hourly Precipitation	1973	Present	DWR
Upper San Leandro Filter Plant	E40 9185 00	EBMUD	37.767	-122.167	390	Hourly Precipitation	1948	Present	DWR
Piedmont Fire Station	E40 6856 70	Alameda County	37.823	-122.232	330	Hourly Precipitation	1972	1989	DWR
Oakland Airport	E40 6335 00	HPD	37.733	-122.200	3	Hourly Precipitation	1940	1985	DWR
Berkeley	n/a	UC	37.868	-122.268	205	15 Minute Precipitation	2001	Present	UC
Shepherd Canyon Fire Station	n/a	Oakland Fire Department	37.824	-122.204	630	Daily Precipitation	Unknown	Present	OFD
Oakland North	ONO	Oakland Fire Services Agency	37.867	-122.217	1,300	Hourly Precipitation	11/1/1992	Present	CDEC
Oakland South	OSO	Oakland Fire Services Agency	37.7881	-122.1436	1,180	Hourly Precipitation	11/1/1992	Present	CDEC

Figure 25: Upper San Leandro Filter Plant Annual Precipitation Totals

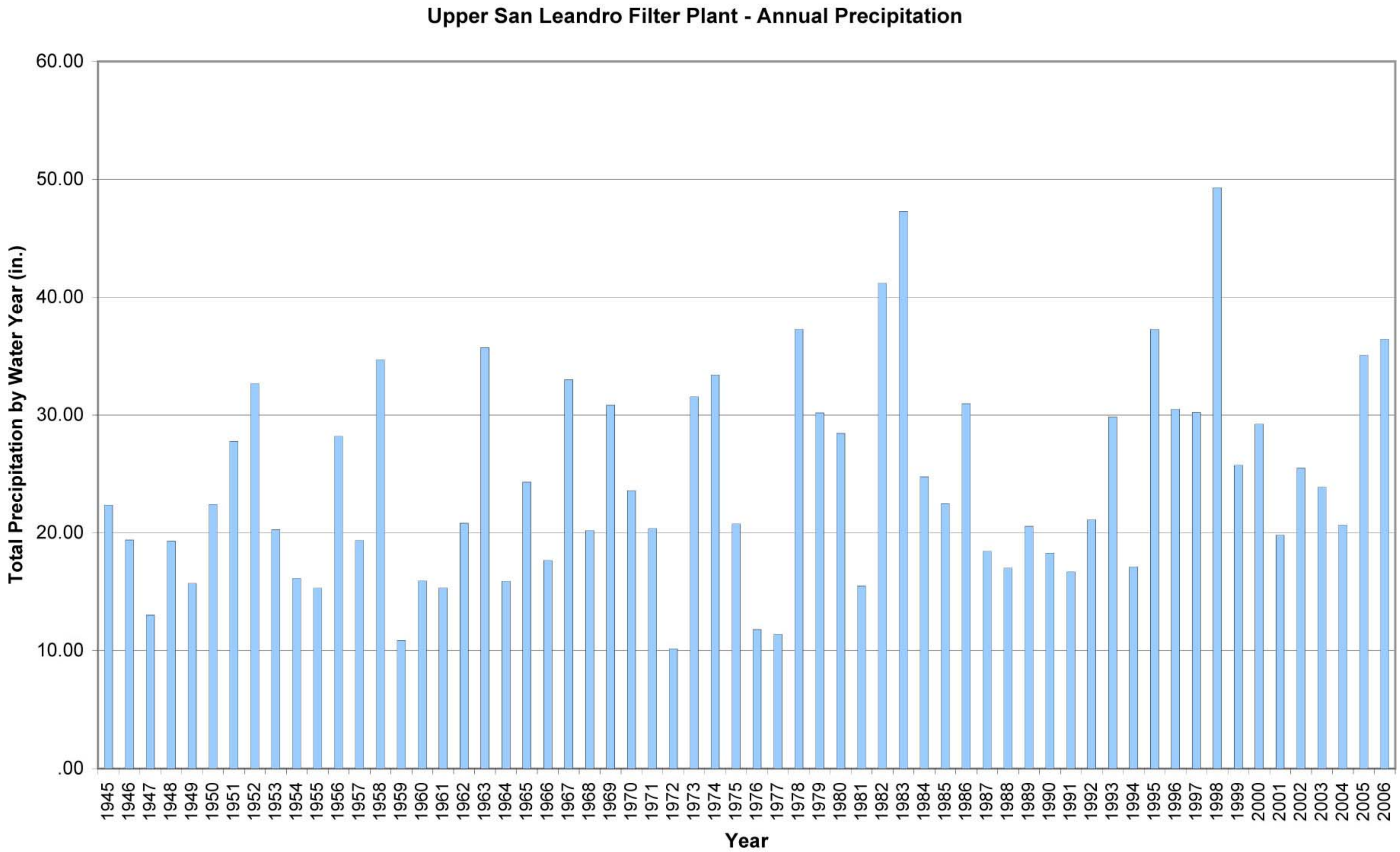


Figure 26: Comparison of Annual Precipitation by Elevation

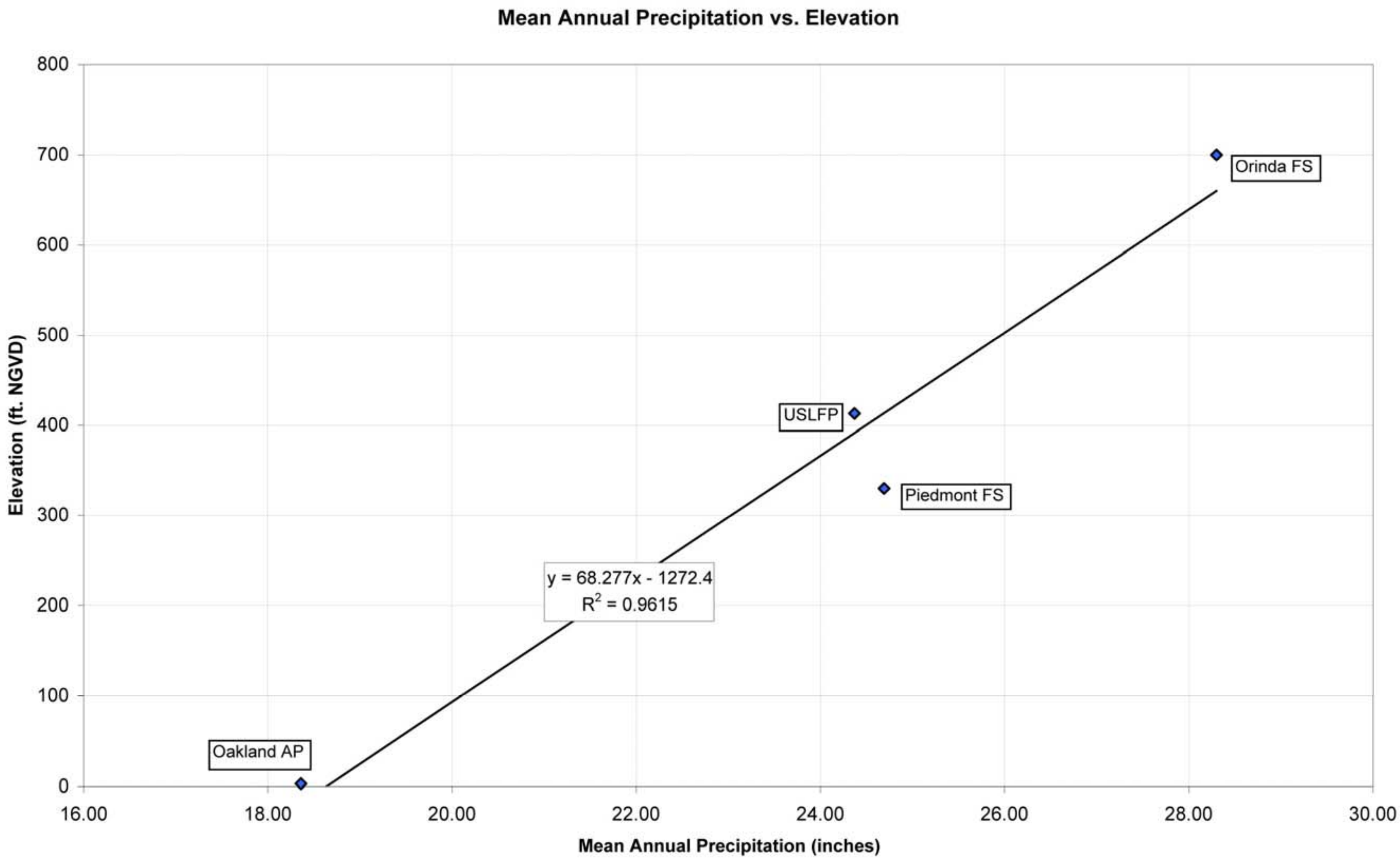
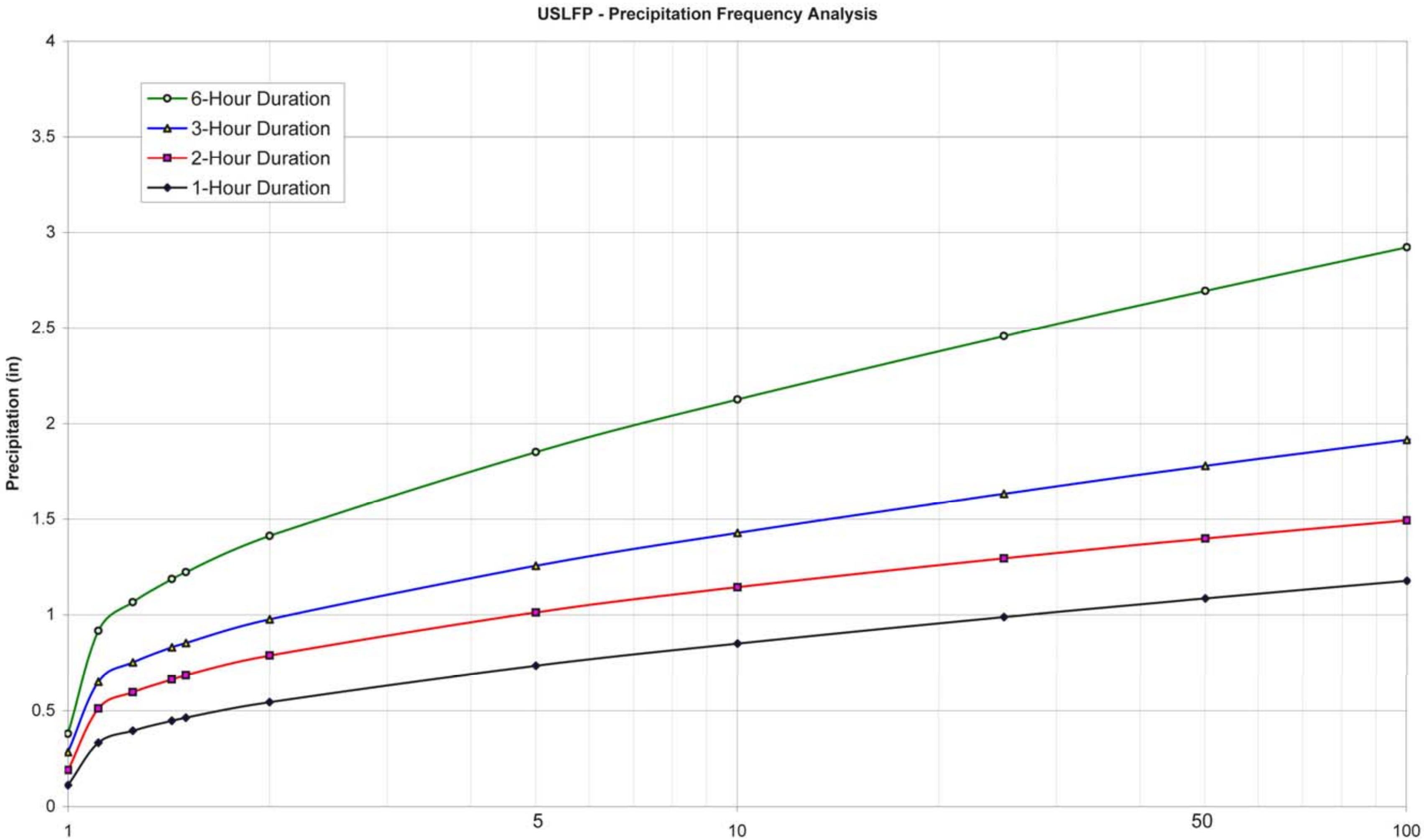


Table 6: Peak Precipitation Volumes for the 1-Hour through 24-Hour Storm Events

Upper Basin							
Elevation	Return Interval	1 HR	2HR	3HR	6HR	12HR	24 HR
1134	2-yr	0.61	0.95	1.22	1.86	2.65	3.33
1134	5-yr	0.83	1.25	1.60	2.49	3.62	4.69
1134	10-yr	0.96	1.42	1.84	2.88	4.26	5.65
1134	25-yr	1.12	1.63	2.13	3.37	5.07	6.92
1134	100-yr	1.35	1.91	2.54	4.06	6.24	8.94
Middle Basin							
Elevation	Return Interval	1 HR	2HR	3HR	6HR	12HR	24 HR
708	2-yr	0.57	0.86	1.08	1.60	2.16	2.75
708	5-yr	0.77	1.11	1.40	2.11	2.94	3.87
708	10-yr	0.90	1.26	1.60	2.44	3.45	4.67
708	25-yr	1.04	1.43	1.84	2.83	4.08	5.74
708	100-yr	1.25	1.66	2.17	3.39	5.02	7.47
Lower Basin							
Elevation	Return Interval	1 HR	2HR	3HR	6HR	12HR	24 HR
98	2-yr	0.52	0.71	0.87	1.21	1.47	1.91
98	5-yr	0.69	0.91	1.10	1.57	1.96	2.69
98	10-yr	0.80	1.02	1.25	1.79	2.28	3.26
98	25-yr	0.93	1.15	1.41	2.06	2.68	4.05
98	100-yr	1.10	1.31	1.64	2.43	3.27	5.37

Figure 27: Return Frequency Analysis for the 1-Hour through 6-Hour Precipitation Duration



Return Interval (years)

Balanced Hyetograph Development

A hyetograph is a time series of precipitation over the duration of a storm event. The hourly precipitation data from the precipitation analysis were evaluated for frequency and occurrence to develop a balanced hyetograph for the 2-, 5-, 10-, 25-, and 100-year storm events. The duration for each of the design storms was 24-hours. A balanced hyetograph is constructed in a way to ensure that the 24-hour hyetograph contains the 1-hour peak rainfall as well as the 2-hour and 3-hour through the 24-hour duration event for the specified return frequency. This allows the 24-hour storm event to be applied to basins that peak within different duration periods. Table 7 lists the hourly precipitation data for the different storm events as calculated from the three synthetic Sausal Creek precipitation gages. These hyetographs were applied to the sub-basins in the upper, middle, and lower sections of the watershed. Figures 28 through 31 are time series plots of each of the storm hydrographs.

Table 7: Sausal Creek Lower Watershed 24-Hour Storm Hyetographs (inches)

Hour	1-Year Event	2-Year Event	5-Year Event	10-Year Event	25-Year Event	100-Year Event
1	0.00	0.02	0.05	0.06	0.09	0.13
2	0.01	0.04	0.06	0.08	0.11	0.14
3	0.01	0.04	0.06	0.08	0.11	0.17
4	0.01	0.04	0.06	0.08	0.11	0.18
5	0.01	0.04	0.06	0.08	0.11	0.18
6	0.01	0.04	0.06	0.08	0.11	0.18
7	0.01	0.04	0.06	0.08	0.11	0.18
8	0.01	0.04	0.07	0.09	0.11	0.18
9	0.02	0.08	0.09	0.10	0.13	0.18
10	0.02	0.11	0.15	0.17	0.20	0.24
11	0.03	0.15	0.19	0.22	0.25	0.29
12	0.10	0.52	0.69	0.80	0.93	1.10
13	0.04	0.20	0.22	0.22	0.27	0.33
14	0.03	0.14	0.19	0.21	0.22	0.26
15	0.02	0.09	0.14	0.16	0.20	0.21
16	0.01	0.06	0.09	0.10	0.12	0.18
17	0.01	0.04	0.06	0.08	0.11	0.18
18	0.01	0.04	0.06	0.08	0.11	0.18
19	0.01	0.04	0.06	0.08	0.11	0.18
20	0.01	0.04	0.06	0.08	0.11	0.18
21	0.01	0.04	0.06	0.08	0.11	0.18
22	0.01	0.04	0.06	0.08	0.11	0.18
23	0.01	0.03	0.06	0.07	0.10	0.14
24	0.00	0.02	0.04	0.05	0.07	0.11
Peak Hour Intensity	**0.103**	**0.52**	**0.69**	**0.80**	**0.93**	**1.10**
24-Hour Total	**0.39**	**1.91**	**2.69**	**3.26**	**4.05**	**5.37**

Table 8: Sausal Creek Middle Watershed 24-Hour Storm Hyetographs (inches)

Hour	1-Year Event	2-Year Event	5-Year Event	10-Year Event	25-Year Event	100-Year Event
1	0.01	0.05	0.08	0.10	0.14	0.20
2	0.01	0.05	0.08	0.10	0.14	0.20
3	0.01	0.05	0.08	0.10	0.14	0.20
4	0.01	0.05	0.08	0.10	0.14	0.20
5	0.01	0.05	0.08	0.10	0.14	0.20
6	0.01	0.05	0.08	0.10	0.14	0.20
7	0.01	0.07	0.11	0.14	0.18	0.24
8	0.02	0.10	0.14	0.18	0.22	0.28
9	0.03	0.14	0.19	0.22	0.25	0.31
10	0.04	0.18	0.24	0.28	0.32	0.39
11	0.04	0.22	0.29	0.34	0.39	0.43
12	0.11	0.57	0.77	0.90	1.04	1.25
13	0.06	0.28	0.33	0.36	0.41	0.51
14	0.04	0.20	0.27	0.31	0.36	0.42
15	0.03	0.15	0.21	0.25	0.31	0.39
16	0.02	0.11	0.16	0.20	0.24	0.31
17	0.02	0.08	0.13	0.16	0.20	0.26
18	0.01	0.06	0.10	0.12	0.16	0.22
19	0.01	0.05	0.08	0.10	0.14	0.20
20	0.01	0.05	0.08	0.10	0.14	0.20
21	0.01	0.05	0.08	0.10	0.14	0.20
22	0.01	0.05	0.08	0.10	0.14	0.20
23	0.01	0.05	0.08	0.10	0.14	0.20
24	0.01	0.05	0.08	0.10	0.14	0.20
Peak Hour Intensity	**0.113**	**0.57**	**0.77**	**0.90**	**1.04**	**1.25**
24-Hour Total	**0.55**	**2.75**	**3.87**	**4.67**	**5.74**	**7.47**

Table 9: Sausal Creek Upper Watershed 24-Hour Storm Hyetographs (inches)

Hour	1-Year Event	2-Year Event	5-Year Event	10-Year Event	25-Year Event	100-Year Event
1	0.01	0.06	0.09	0.12	0.15	0.23
2	0.01	0.06	0.09	0.12	0.15	0.23
3	0.01	0.06	0.09	0.12	0.15	0.23
4	0.01	0.06	0.09	0.12	0.15	0.23
5	0.01	0.06	0.09	0.12	0.15	0.23
6	0.01	0.06	0.09	0.12	0.15	0.23
7	0.02	0.10	0.16	0.19	0.25	0.32
8	0.03	0.13	0.20	0.24	0.29	0.38
9	0.04	0.18	0.25	0.29	0.35	0.43
10	0.04	0.22	0.30	0.35	0.41	0.50
11	0.05	0.27	0.36	0.42	0.50	0.56
12	0.12	0.61	0.83	0.96	1.12	1.35
13	0.07	0.34	0.42	0.46	0.51	0.63
14	0.05	0.24	0.32	0.38	0.44	0.53
15	0.04	0.19	0.26	0.31	0.38	0.48
16	0.03	0.15	0.22	0.26	0.32	0.48
17	0.02	0.12	0.17	0.21	0.27	0.35
18	0.02	0.09	0.14	0.18	0.22	0.30
19	0.01	0.06	0.09	0.12	0.15	0.23
20	0.01	0.06	0.09	0.12	0.15	0.23
21	0.01	0.06	0.09	0.12	0.15	0.23
22	0.01	0.06	0.09	0.12	0.15	0.23
23	0.01	0.06	0.09	0.12	0.15	0.23
24	0.01	0.06	0.09	0.12	0.15	0.23
Peak Hour Intensity	**0.121**	**0.61**	**0.83**	**0.96**	**1.12**	**1.35**
24-Hour Total	**0.67**	**3.33**	**4.69**	**5.65**	**6.92**	**8.94**

Figure 28: 24-Hour Storm Hyetographs, Upper Sausal Creek Watershed

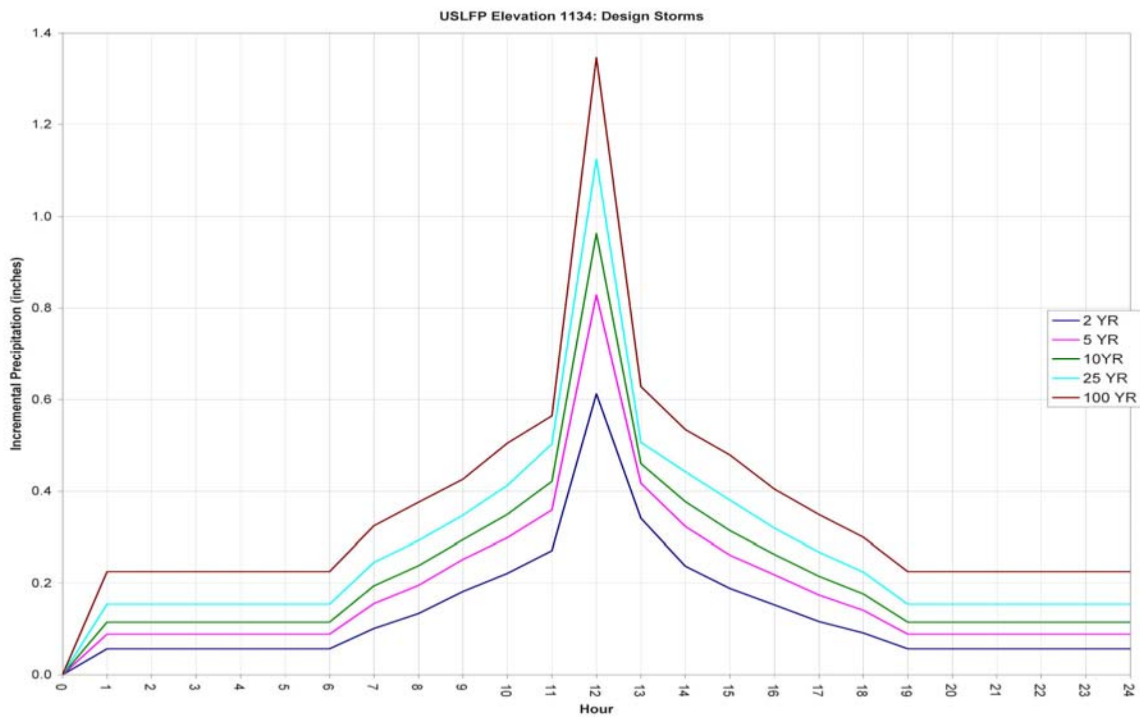


Figure 29: 24-Hour Storm Hyetographs, Middle Sausal Creek Watershed

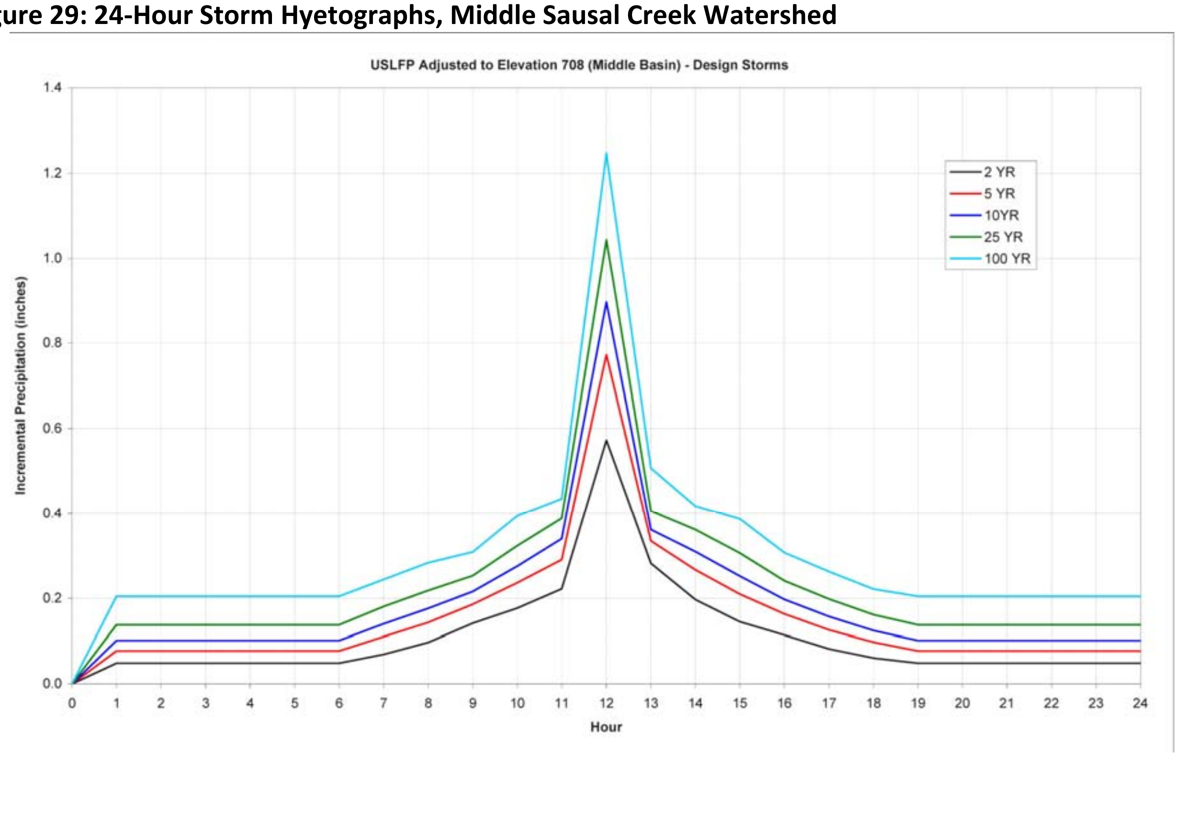
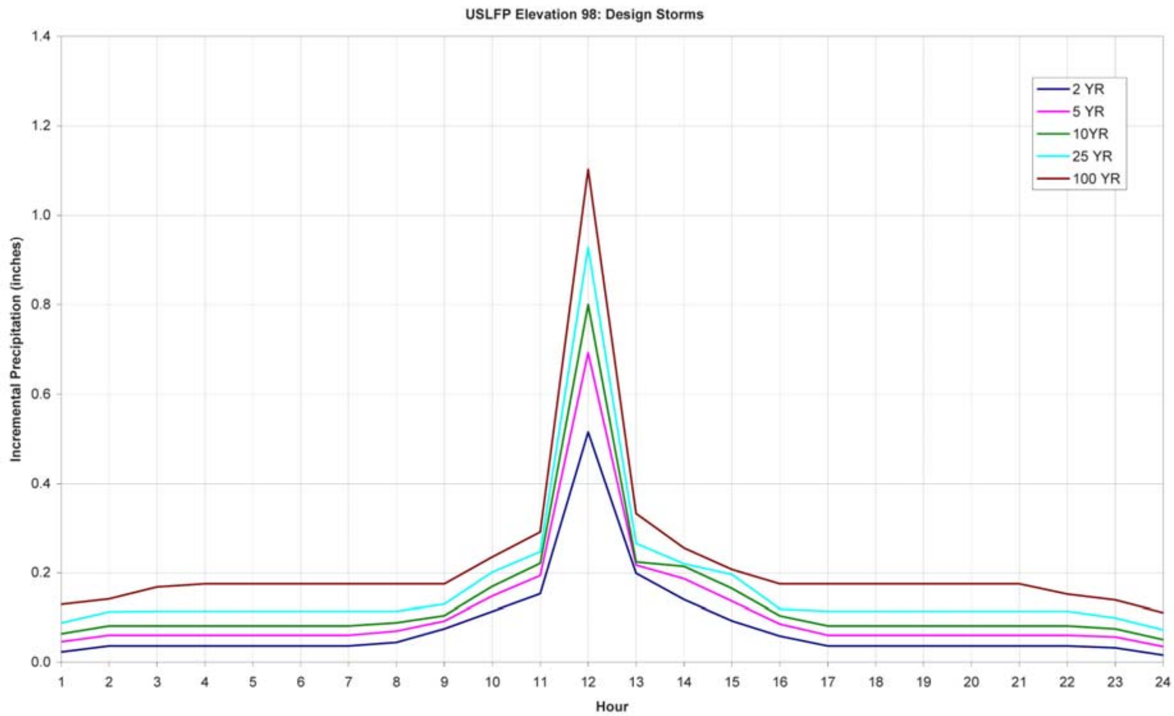


Figure 30: 24-Hour Storm Hyetographs, Lower Sausal Creek Watershed



Soils Analysis

One of the key components in evaluating runoff in the watershed is the soil type and texture. The data developed in the Soil Conservation Service Soil Survey for Alameda County (SCS 1978) were used to characterize the soils within the watershed. Soil characteristics can have a strong impact on the volume and timing of runoff. The two factors that have the greatest influence on runoff are soil permeability and hydrologic soil group. These factors have been determined by the Natural Resource Conservation Service (NRCS) in their analysis of Alameda County soils.

Hydrologic Soil Group

Soils are classified by the NRCS into four Hydrologic Soil Groups based on the soil's runoff potential. The four Hydrologic Soils Groups are A, B, C and D. Soils classified as an A generally has the smallest runoff potential, whereas D's have the greatest potential. Details of this classification can be found in 'Urban Hydrology for Small Watersheds' published by the Engineering Division of the Natural Resource Conservation Service, United States Department of Agriculture, Technical Release–55. The four different soil groups are shown below.

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

Group B soils consist of silt loam or loam. It has a moderate infiltration rate when thoroughly wetted and consists chiefly or moderately deep to deep, moderately well- to well-drained soils with moderately fine to moderately coarse textures.

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

Group D soils consist of clay loam, silty clay loam, sandy clay, silty clay or clay. This soil group has the highest runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material.

Figure 22 shows soil types in the Sausal Creek watershed. A description of the soil type, infiltration rate, and the hydrologic soil group is provided in Table 10.

Table 10: Soils Characteristics

MUSYM Number	Name	Permeability (inches/hour)	Soil Class (A-D)
126	Maymen loam, 30-75% slopes	0.6-2.0	D
127	Maymen-Los Gatos complex, 30-75% slopes	0.6-2.0	C
129	Millsholm silt loam, 50-75% slopes	0.6-2.0	D
130	Montara-Rock outcrop complex, 30-75% slopes	0.2-0.6	D
146	Urban land		
148	Urban land-Clear Lake complex		
150	Urban land-Tierra complex, 2-5% slopes	0.6-2.0	D
151	Urban land-Tierra complex, 5-15% slopes	0.6-2.0	D
152	Urban land-Tierra complex, 15-30% slopes	0.2-0.6	C
158	Xerorthents-Los Osos complex, 30-50% slopes	0.2-0.6	C
159	Xerorthents-Millsholm complex, 30-50% slopes	0.6-2.0	D
162	Water		W

Curve Number Determination

The runoff curve number (CN) is a land cover/use index that represents the effect of soils, land use, and antecedent moisture conditions on rainfall runoff. Curve numbers for this project were taken from the textbook Hydrologic Analysis and Design by Richard McCuen (McCuen, 1989). For any particular land use, the CN can change depending on the hydrologic soil group underlying the basin.

The area of each soil type within each of the sub-basins was calculated to come up with a composite hydrologic soil group for each of the sub-basins. This information together with the land use within each of the sub-basins was used to compute a composite runoff curve number for each sub-basin. Table 12 is a listing of each of the sub-basins in the analysis along with the distribution of soil groups, and composite curve numbers. The composite curve number was computed for each sub-basin based on the land use and soil types represented in the basin.

Flow Path and Lag Time

The runoff, as computed by the model, is a function of precipitation, basin topography, land use and soil types. The basin topography dictates the slope and path that runoff will follow through the basin. The topography of the watershed was obtained from a 5-foot DEM of the City of Oakland conducted in 2004. The topographic information was entered into the project GIS to determine the slope and path lengths that overland flow will follow. Through an analysis of the flow path, the timing of the runoff from each watershed was determined. The runoff timing is converted into a lag time parameter which represents the difference in timing between the peak of the precipitation and the peak of the runoff. Table 11 is a summary of the runoff flow path analysis. The area and percent impermeable in the table were developed from the watershed GIS.

This hydrologic and hydraulic analysis is expressed in terms of recurrence intervals, which are used to estimate the probability of the occurrence of a given precipitation or streamflow event. For example, assume there is a 1 in 50 chance that 4.20 inches of rain will fall in a certain area in a 24-hour period during any given year. Thus, a rainfall total of 4.20 inches in a consecutive 24-hour period is said to have a 50-year recurrence interval. The recurrence interval is based on the probability that the given event will be equaled or exceeded in any given year. Rainfall recurrence intervals are based on both the magnitude and the duration of a rainfall event, whereas streamflow recurrence intervals are based solely on the magnitude of the annual peak flow.

Recurrence intervals for the annual peak streamflow at a given location change if there are significant changes in the flow patterns at that location, possibly caused by an impoundment or diversion of flow. The effects of development (conversion of land from forested or agricultural uses to commercial, residential, or industrial uses) on peak flows is generally much greater for low-recurrence interval floods than for high-recurrence interval floods, such as 25- 50- or 100-year floods. During these larger floods, the soil is saturated and does not have the capacity to absorb additional rainfall. Under these conditions, essentially all of the rain that falls, whether on paved surfaces or on saturated soil, runs off and becomes streamflow.

Stormwater Runoff Analysis

A watershed runoff model was developed to determine the runoff characteristics that result from precipitation over the Sausal Creek Watershed. For this analysis, the Environmental Protection Agency's Storm Water Management Model (SWMM) was used. This computer model allows for the analysis of stormwater runoff from multiple linked basins with various runoff characteristics. It is commonly used in urban watershed studies due to its ability to accurately analyze multiple linked storm drain pipe networks. The basins can be connected by common channels, overland flow or storm drain pipes.

A preliminary effort at characterizing and modeling the hydrologic and hydraulic properties of the Sausal Creek watershed was performed by Hydrologic Systems, Inc (HSI), and documented in the August, 2008 report entitled *Draft Technical Memo: Sausal Creek Watershed Analysis Hydrologic and Hydraulic Assessment* (HSI 2008). HSI developed a SWMM-XP hydrologic model, a propriety software platform that expands the capabilities of EPA SWMM 5.0, a free, publicly-available hydrologic modeling application developed by the USEPA. Because the HSI SWMM-XP license only allowed a limited number of nodes and conduits in the model (100), this computational model actually consisted of two separate models, one a more detailed model of the upper watershed, and one a more simplified model of the entire system. HSI also developed a HEC-RAS model of the Dimond Canyon section of the Sausal Creek channel, up to and including the lower sections of the Palo Seco Creek tributary.

Table 11: Sub-Basin Runoff Characteristics

Basin	Area	Percent Impermeable	Pervious Area Curve Number	Total Lag Time
	(acres)		CN	(minutes)
SC-0	72.9	35.09	80	8.94
SC-1	100.3	34.23	84	10.56
SC-2	54.5	38.86	84	13.40
SC-3	56.6	38.38	84	17.44
SC-4	77.9	33.44	84	10.89
SC-5	23.0	32.22	84	11.56
SC-6	34.0	28.79	84	18.03
SC-7	19.2	33.83	84	11.47
SC-8	51.9	34.22	84	12.55
SC-9	31.9	31.72	84	16.55
SC-10	56.1	30.97	84	17.80
SC-11	18.8	28.82	80	16.74
SC-12	55.7	34.79	79	12.26
SC-13	53.5	23.68	77	14.18
C-1	68.3	33.55	80	25.64
C-2	45.6	33.47	78	15.74
C-3	67.5	30.46	80	17.14
C-4	105.3	23.62	82	24.35
C-5	27.7	22.95	75	17.08
PS-1	109.2	4.95	81	21.39
PS-2	147.5	4.84	78	25.72
PS-3	64.4	0.00	81	13.86
PS-4	65.9	3.35	81	20.26
PS-5	35.9	32.13	79	15.45
DC-1	124.0	48.57	77	16.40
DC-2	80.4	35.37	75	17.27
DC-3	108.3	25.96	75	20.63
DC-4	103.6	32.96	75	21.04
DC-5	119.9	78.67	79	
DC-6	387.1	79.03	83	
DC-7	173.7	79.61	84	
FRC-1	107.8	76.67	76	
FRC-2	129.3	69.94	78	

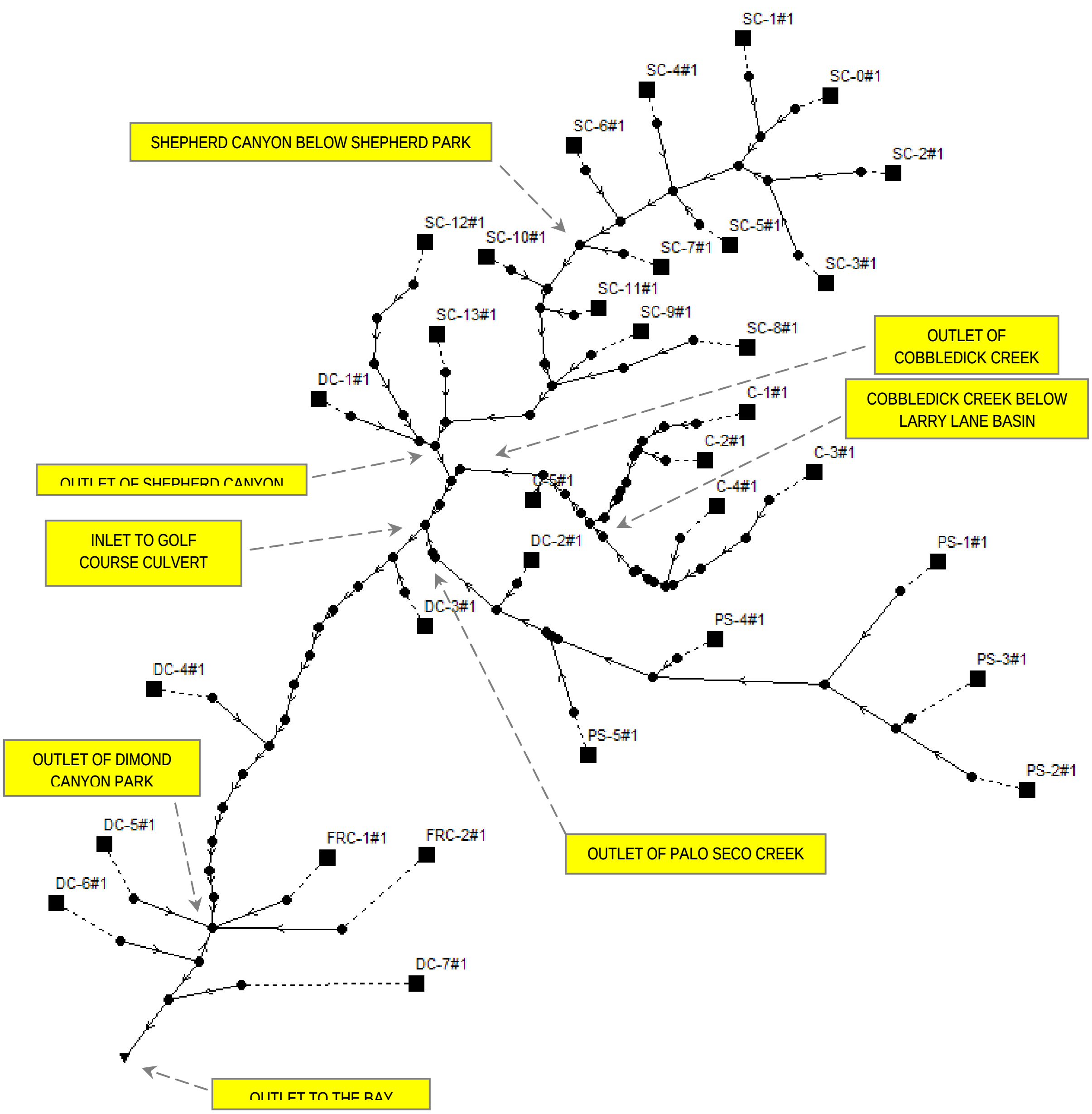
NewFields River Basin Services, LLC (NRBS) was retained in October 2009 to complete the existing conditions hydrology and hydraulic models, calibrate the models to verify accuracy, and model the hydrologic and hydraulic effects of proposed stormwater detention facilities in the watershed.

Watershed delineation, precipitation data and hyetograph development, soils analyses, and sub-basin runoff characteristics were developed by HSI. The two Sausal Creek SWMM-XP models previously developed by HSI were converted to the EPA SWMM 5.0 platform. When this conversion was made, some data did not export correctly and was re-tabulated and re-entered into the Sausal Creek EPA

SWMM 5.0 model. This included sub-catchment slope values and some of the conduit transects. This data was obtained from HSI and re-entered into the model. In addition, the two Sausal Creek XP-SWMM models were combined into one Sausal Creek hydrologic model in EPA SWMM 5.0 to enable consistency and ease of use (Figure 31).

In addition, NRBS developed a 1-year rainfall hyetograph that was not included in the original HSI hydrology analysis (Table 12). The Sausal Creek EPA SWMM 5.0 existing conditions model was run using dynamic wave analysis and time steps of 1 second or less for the 1 –year, 2-year, 5-year, 10-year, 25-year, and 100-year storm events.

Figure 31: Schematic of the Sausal Creek EPA SWMM 5.0 existing conditions model. Sub-basins are labeled and represented by the black squares, nodes by the black circles, the watershed outfall by a black triangle and conduits by the lines. Arrows show the direction of flow; yellow boxes indicate locations of interest.



The SCS Unit Hyetograph procedure is used within the SWMM Model to calculate the runoff that would result from the design storm precipitation. This procedure involves calculating the direct runoff from a basin after fulfilling the requirements of initial abstraction and basin storage. The basic methodology was developed by the Soil Conservation Service (SCS 1972). Numerically the process is shown in Equations 1 through 3 below:

Equation 1:

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

Equation 2:

$$S = \frac{25400 - (254\text{-}CN)}{CN}$$

Equation 3:

$$I_a = 0.2\,S$$

Where:

Q = direct runoff
P = precipitation
I_a = initial abstraction
S = maximum storage
CN = NRCS curve number

Two additional key parameters in the SCS Unit Hyetograph procedure are the runoff lag time and the runoff curve number. The lag time is the time (in hours) between the center of mass of the excess rainfall to the time to the peak discharge. This lag time can be estimated by Equation 4:

Equation 4:

$$T_{lag} = 0.6\,T_c$$

Where:

T_{lag} = the lag time in hours
T_c = the time of concentration of runoff to the point of discharge

Model Calibration

Existing conditions output for the Sausal Creek hydrology model was calibrated to streamflow measurements during significant storm events on October 19, 2009 and November 20, 2009. However, because the October 19 storm was larger and NRBS was able to conduct measurements during the runoff peak on this date, we relied primarily on these measurements for model calibration.

Because of access limitations, calibration measurements were collected in an approximately 2000-foot long reach upstream of El Centro Avenue. NRBS measured the high water mark elevation at the upstream end of the culvert at El Centro Avenue (Figure 32) for the peak discharge on October 19, 2009. NRBS also surveyed the longitudinal profile slope through the culvert and assessed the roughness of the concrete culvert. Then, using a standard calculation for flow through a partially filled culvert, the discharge in Sausal Creek that corresponded to the high water mark elevations for the October 19 event was calculated as 167 cfs. Discharge was calculated as the product of velocity and cross-sectional area ($Q=VA$). A high water mark was measured in the culvert and the area of the pipe filled with water was calculated to provide A. Velocity (V) was measured in the reach just upstream of the culvert and the two numbers are multiplied to get the Q or discharge value.

Figure 32: The photo on the left shows the October 19, 2009 peak discharge water surface elevation at the upstream end of the culvert at El Centro Avenue. For comparison, the photo on the right shows conditions during low or base flow.

Hourly precipitation data for the October 19 storm event was obtained from the California Data Exchange Center (CDEC) website for two stations, the Oakland North station which lies 11,000 feet to the north of the Sausal Creek watershed at an elevation of 1,300', and the Oakland South station which lies 11,500 feet to the south of the Sausal Creek watershed at an elevation of 1,180' (Figure 24).

Hourly precipitation data for the October 19 event at these two stations is presented in Figure 33 below. The October 19, 2009 flow event was slightly larger than a 1-year recurrence event.

Figure 33: Hourly Precipitation Data at Oakland North and Oakland South Stations for October 19, 2009 Storm Event

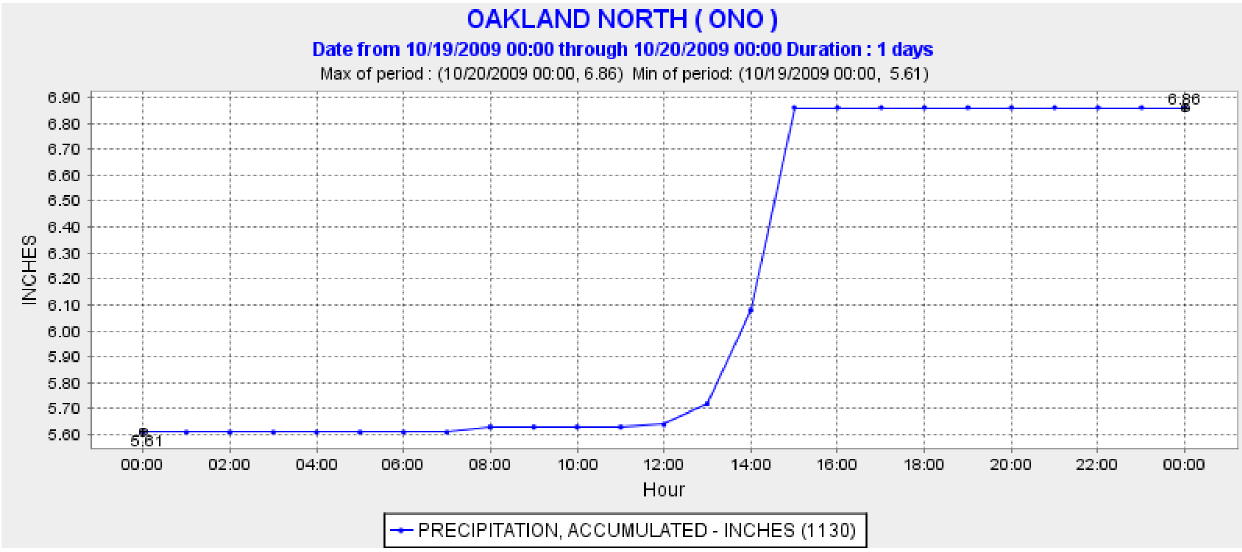
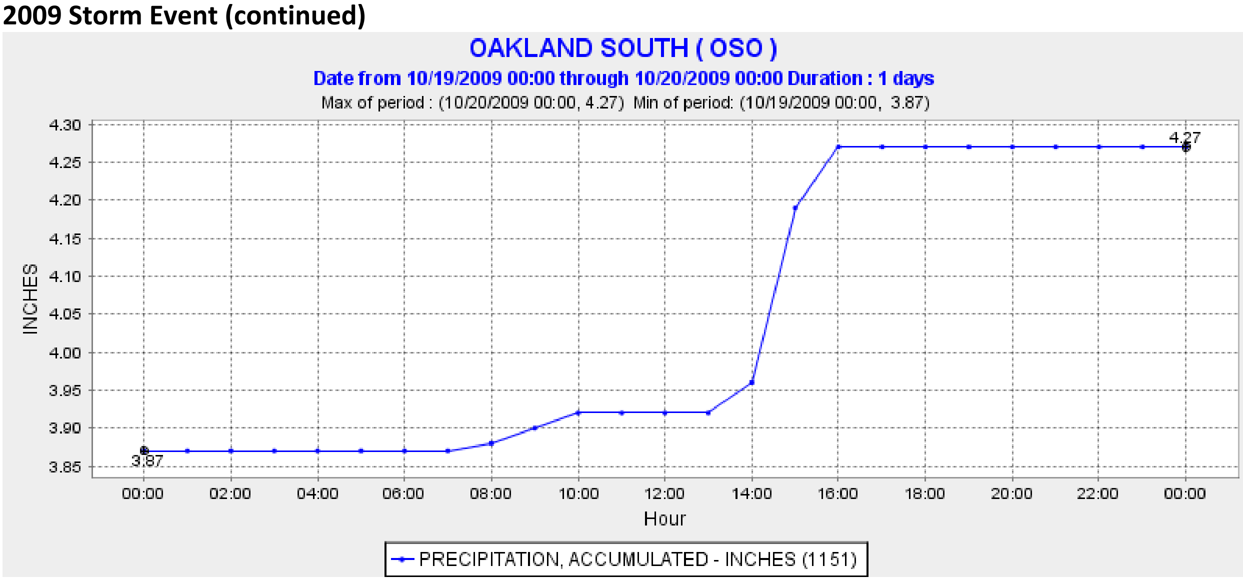


Figure 33: Hourly Precipitation Data at Oakland North and Oakland South Stations for October 19, 2009 Storm Event (continued)



Source: DWR, CDEC website, 2009

The highest elevation in the Sausal Creek watershed is approximately 1,300' at the crest of the Oakland Hills. This is similar to the Oakland North and Oakland South stations, which lie at 1,300' and 1,180' feet elevation, respectively. Because the midpoint of the Sausal Creek watershed lies halfway in between these stations, the calibration rainfall values were averaged between the two stations for the specific events. A 24-hour hyetograph for each station was developed (Table 12), and the averaged values were input into SWMM to mimic the rainfall during the calibration storm event.

Table 12: Hyetograph for the October 19, 2009 Storm Event

Date / Time	Oakland South Station		Oakland North Station		Average
	Elevation of Rain in Gage (Inches)	Hourly Rainfall (Inches)	Elevation of Rain in Gage (Inches)	Hourly Rainfall (Inches)	Hourly Rainfall (Inches)
10/19/09 0:00	3.87	0.00	5.61	0.00	0.00
10/19/09 1:00	3.87	0.00	5.61	0.00	0.00
10/19/09 2:00	3.87	0.00	5.61	0.00	0.00
10/19/09 3:00	3.87	0.00	5.61	0.00	0.00
10/19/09 4:00	3.87	0.00	5.61	0.00	0.00
10/19/09 5:00	3.87	0.00	5.61	0.00	0.00
10/19/09 6:00	3.87	0.00	5.61	0.00	0.00
10/19/09 7:00	3.87	0.00	5.61	0.00	0.00
10/19/09 8:00	3.88	0.01	5.63	0.02	0.01
10/19/09 9:00	3.9	0.02	5.63	0.00	0.01
10/19/09 10:00	3.92	0.02	5.63	0.00	0.01
10/19/09 11:00	3.92	0.00	5.63	0.00	0.00
10/19/09 12:00	3.92	0.00	5.64	0.01	0.00
10/19/09 13:00	3.92	0.00	5.72	0.08	0.04
10/19/09 14:00	3.96	0.04	6.08	0.36	0.20
10/19/09 15:00	4.19	0.23	6.86	0.78	0.51
10/19/09 16:00	4.27	0.08	6.86	0.00	0.04
10/19/09 17:00	4.27	0.00	6.86	0.00	0.00
10/19/09 18:00	4.27	0.00	6.86	0.00	0.00
10/19/09 19:00	4.27	0.00	6.86	0.00	0.00
10/19/09 20:00	4.27	0.00	6.86	0.00	0.00
10/19/09 21:00	4.27	0.00	6.86	0.00	0.00
10/19/09 22:00	4.27	0.00	6.86	0.00	0.00
10/19/09 23:00	4.27	0.00	6.86	0.00	0.00
10/20/09 0:00	4.27	0.00	6.86	0.00	0.00
24-Hour Total		**0.40**		**1.25**	**0.83**

The SWMM existing conditions model was run with the calibration rainfall data. Comparison of modeled flow versus observed flow is provided in Table 13.

Table 13: Comparison of Measured Versus Modeled Flow at the El Centro Culvert in Dimond Canyon

	Location	Maximum Flow (cfs)	Maximum Depth (ft)	Time of Measurement / Max Occurrence
Measured Discharge	El Centro culvert	167	1.8	15:45
SWMM Model Output	Node 153 (approximate location of El Centro culvert)	211	2.25	16:06

The final modeled flow rate of 211 cfs is 26% larger than the measured rate of 167 cfs at the El Centro culvert. However, because of the scale and complexity involved in modeling the Sausal Creek watershed in SWMM, and limitations in the resolution of model input parameters describing watershed conditions, this calibration was determined to be acceptable as validation of the order of magnitude of predictions from the model. This exercise also provided information on the magnitude of potential error associated with model output that could be reduced with additional characterization of the watershed and associated development of the model. Potential sources of error between the measured and modeled flows include the abstraction of available hourly precipitation data from stations outside of the watershed, errors involved in measuring flow through the culvert during the calibration measurements, and errors in the architecture of the hydrology model.

Hydraulic Model

Hydrologic Systems, Inc. initiated development of a hydraulic model for portions of the Sausal Creek watershed using the Hydrologic Engineering Center River Analysis System, commonly referred to as HEC-RAS (USACE 2008). The model focuses on the reach of Sausal Creek through Dimond Canyon. NRBS analysis included completion of the Sausal Creek hydraulic model (including model calibration) and evaluation of hydraulic characteristics under existing hydrologic conditions and “proposed” hydrologic conditions with implementation of watershed improvements to reduce peak magnitudes of runoff from rainfall in the watershed. The completed HEC-RAS model consists of two (2) cross sections in Palo Seco Creek and thirty-seven (37) cross sections in the mainstem of Sausal Creek between Highway 13 at the upstream end and the northern end of Dimond Avenue at the downstream end (Figure 34). The hydraulic model also includes four culverts in the modeled reach. NRBS evaluated the preliminary model cross sections and culvert parameters against measured field conditions to validate the model prior to calibration.

Details of the culverts within the watershed were obtained from storm drain design maps that are maintained by the City of Oakland. This information was augmented with a detailed channel invert survey of Dimond Canyon between Highway 13 and Dimond Park. Figure 34 shows the profile of Sausal Creek between the outlet at the bay and Highway 13 at the upper end of Dimond Canyon.

Model Calibration

The objective of hydraulic model calibration is to make model predictions of hydraulic parameters such as water surface elevation and velocity consistent with actual conditions in the modeled system. Model calibration is especially important for this analysis of Sausal Creek because the changes in hydrology from watershed projects are relatively small.

The preliminary hydraulic model completed by HSI utilized uniform Manning’s *n* (roughness) values of 0.035 throughout the entire model domain for both the in-channel and overbank areas. While 0.035 is generally accepted as a reasonable starting roughness value for natural creeks, NRBS determined based on a reconnaissance evaluation of the modeled reaches that 0.035 was not an appropriate roughness value for all of the modeled reaches in Sausal Creek, especially for hydraulic analysis of more frequent peak discharges (i.e. discharges with recurrence intervals of less than 2-years) that appeared likely to be influenced by significant channel bed complexity and dense riparian vegetation in many areas.

Therefore, the hydraulic model was calibrated with measurements of high water marks and flow velocities at a known discharge. NRBS collected these calibration measurements during significant storm events on October 19, 2009 and November 20, 2009. However, because the October 19 storm was larger and measurements were collected during the runoff peak on this date, NRBS relied primarily on these measurements for model calibration.

Figure 34: Schematic Diagram of the Sausal Creek Hydraulic Model as Implemented in HEC-RAS. Numbers indicate locations of channel cross section included in the model. Yellow polygons represent culverts and bridges included in the model as hydraulic structures. Flow is from top to bottom as indicated by the orange arrow.

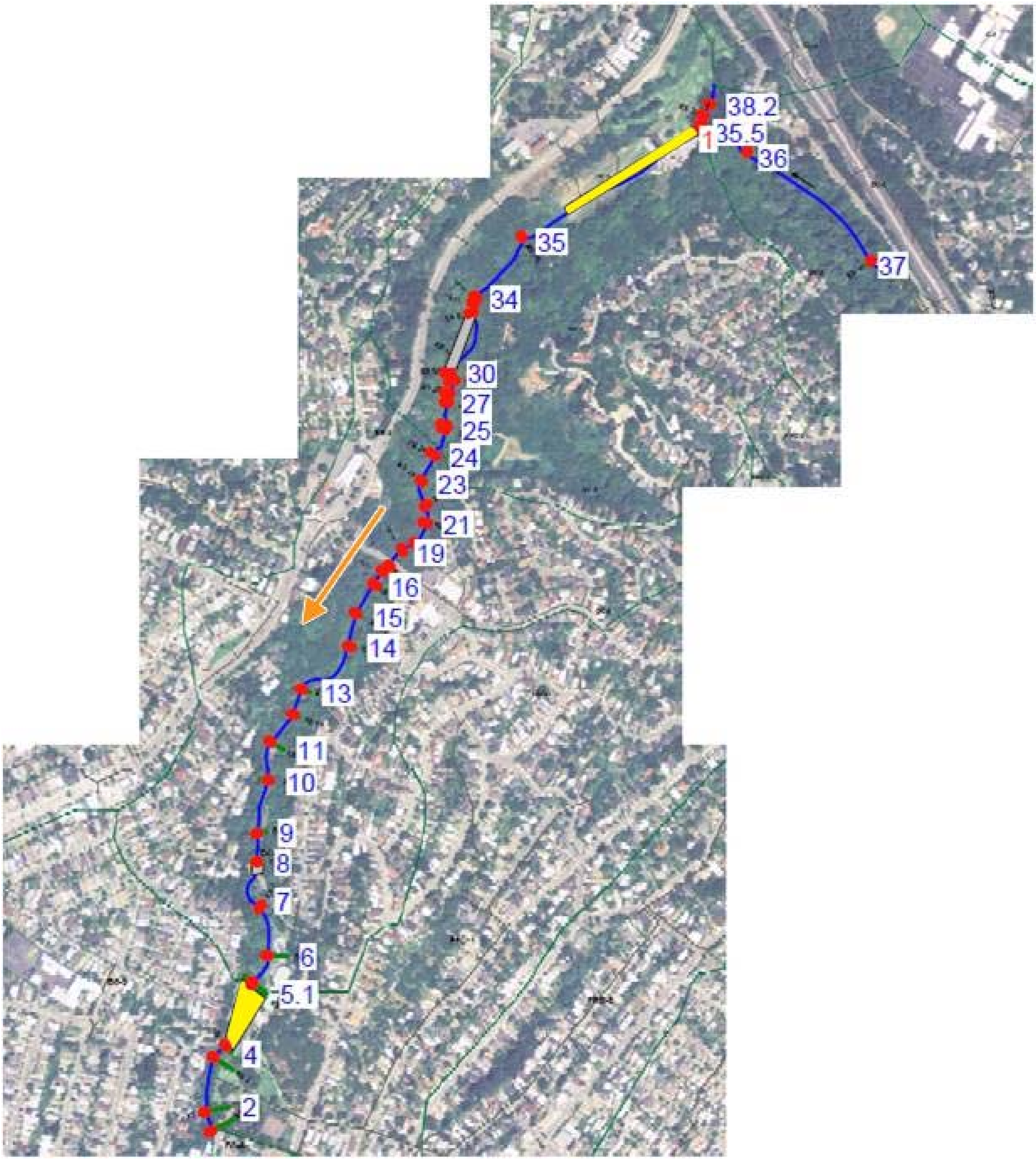


Figure 35: Sausal Creek Channel Profile, Station 0 to 8000

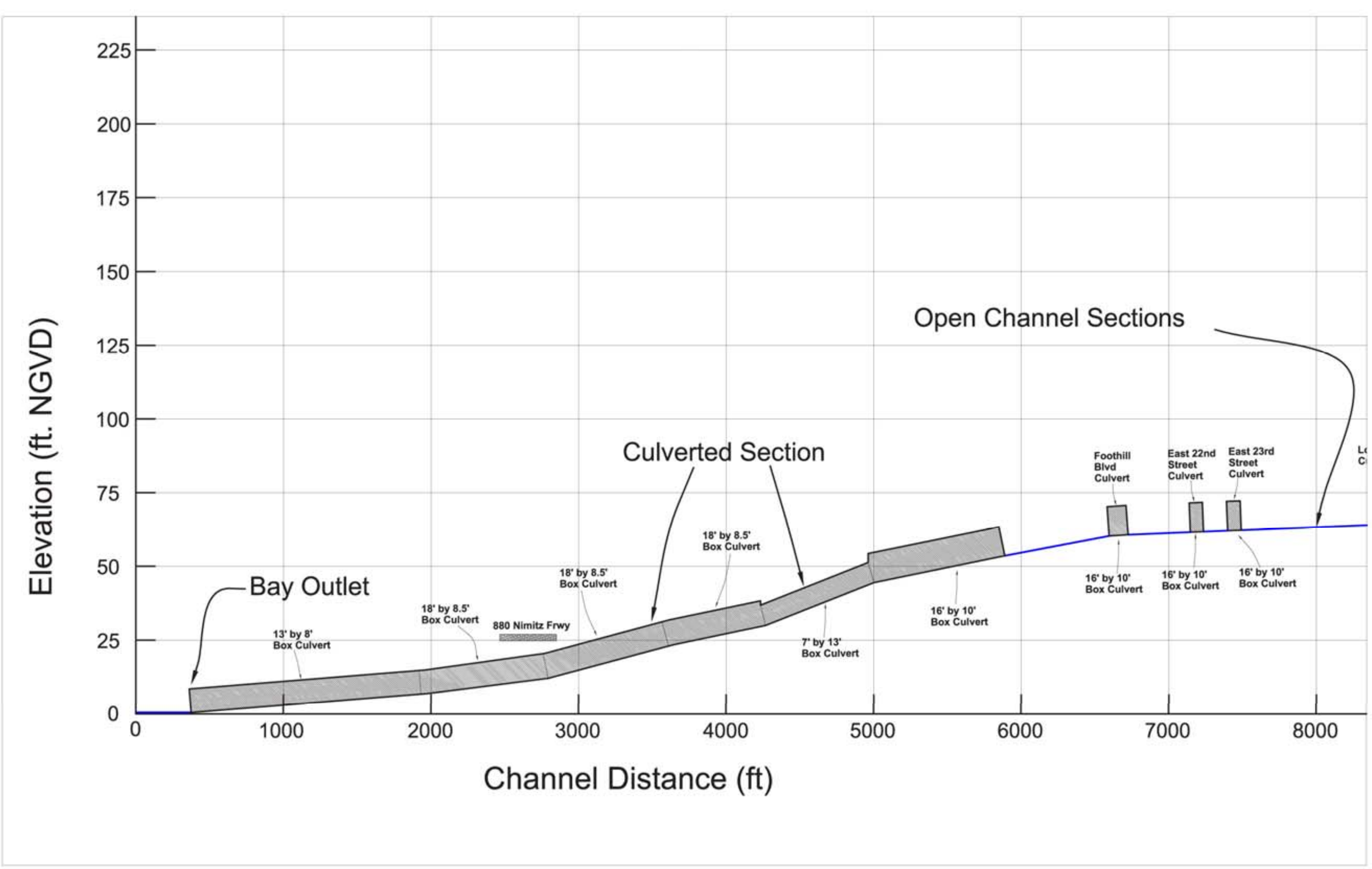


Figure 35 (continued): Sausal Creek Channel Profile, Station 8000 to 16000

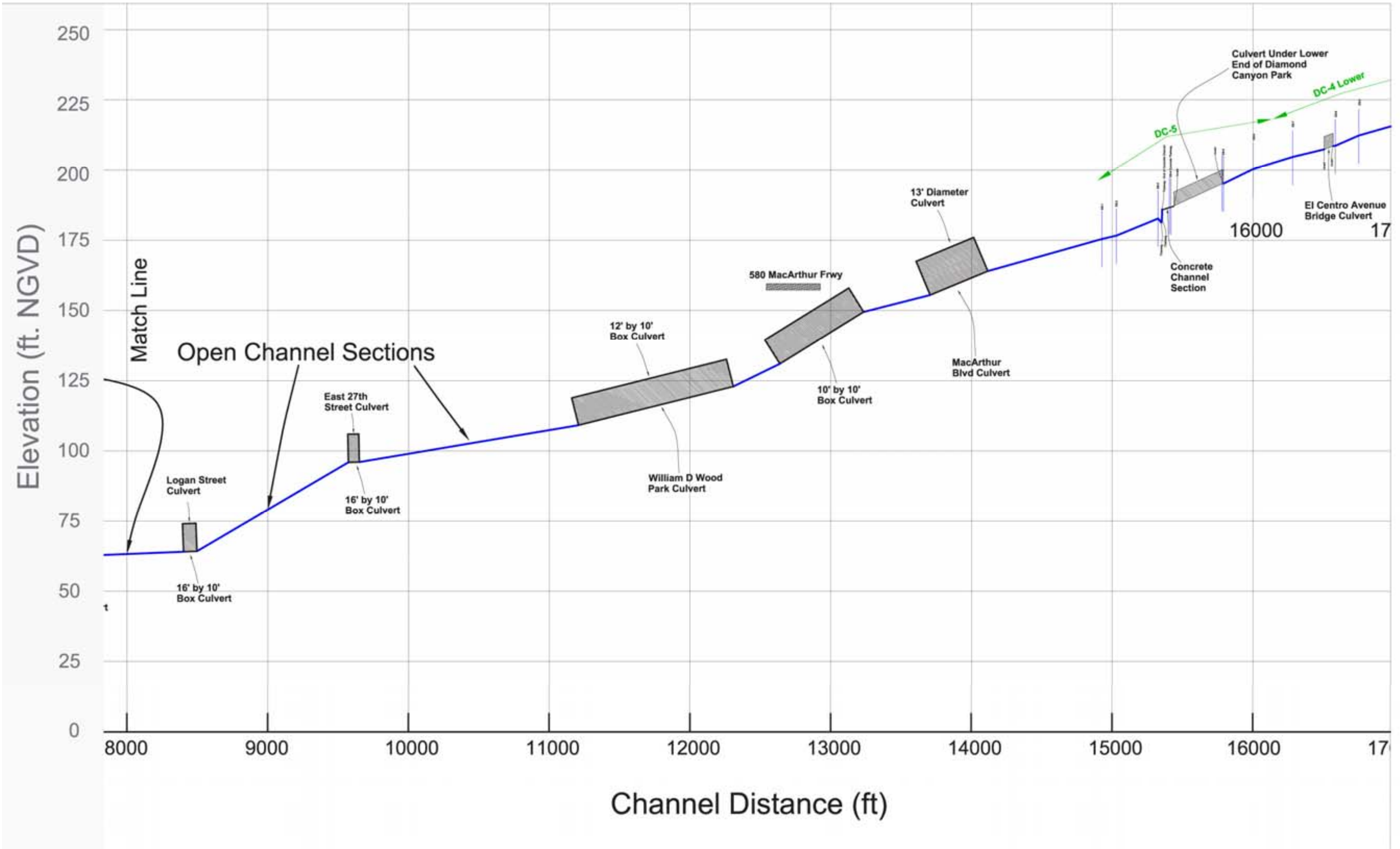
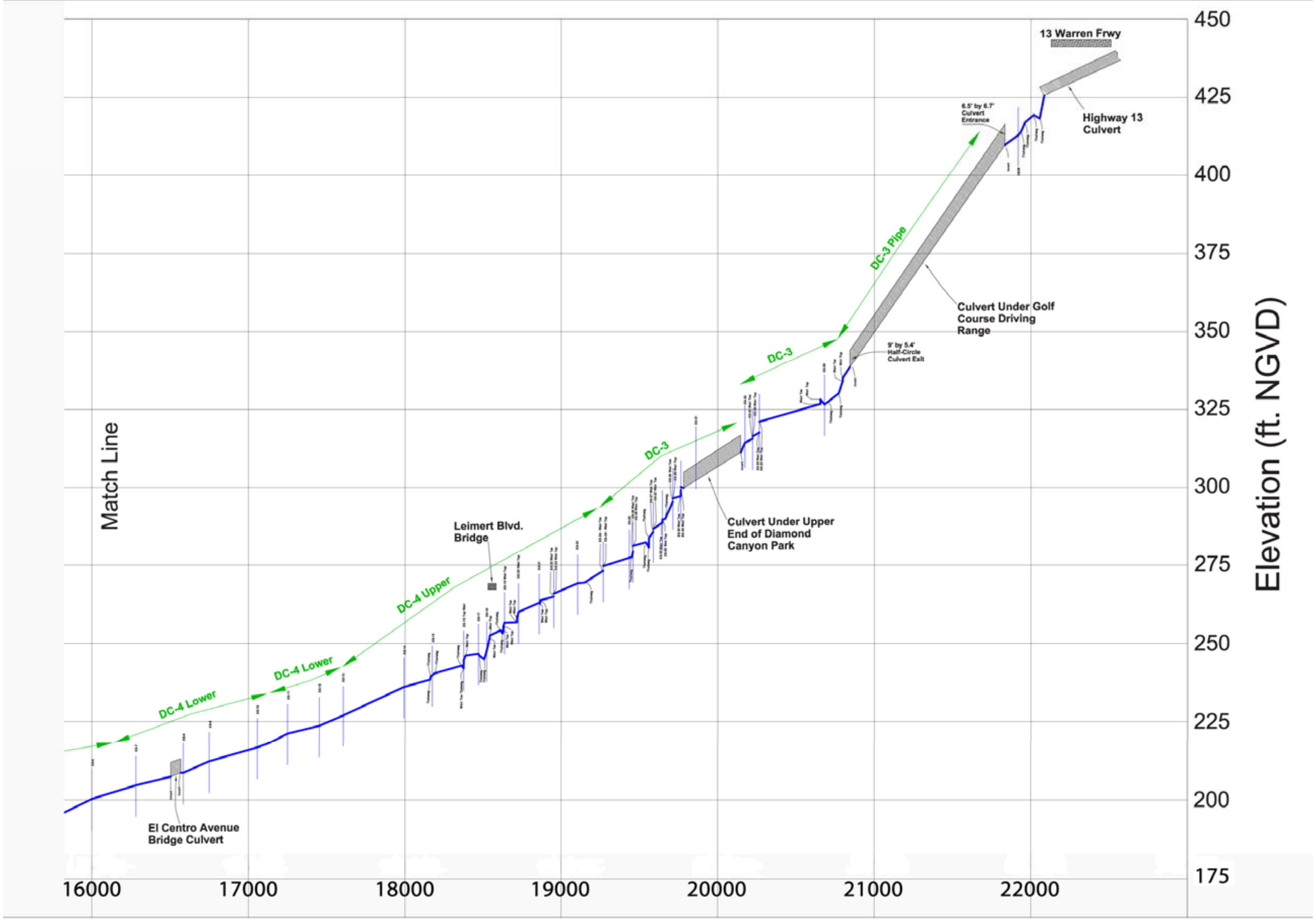


Figure 35 (continued): Sausal Creek Profile, Station 16000 to 230000



Because of access limitations, calibration measurements were collected in an approximately 2000-foot long reach upstream of El Centro Avenue. NRBS applied calibrated roughness values from this reach to the remaining reaches of the model. The calibration exercise consisted of field measurements of discharge, high water mark elevations, and flow velocities, and iterative adjustment of roughness values in the HEC-RAS model until modeled water surface elevations matched measured water surface elevations at the calibration flow.

Figure 36: Typical high water marks measured in Sausal Creek including flattened riparian vegetation, debris lines, and sediment deposition.

High Water Mark Elevations

Because discharge in Sausal Creek changes rapidly during and after rainfall events in the watershed, NRBS measured both flow depth and high water mark elevations. Figure 36 illustrates a typical high water mark used in this set of calibration measurements.

Flow Velocity

NRBS used a neutrally buoyant float to estimate flow velocity in several areas of the Dimond Canyon reach of Sausal Creek. The travel time of the float over a 100 foot distance was recorded and velocity was calculated as distance divided by time. Estimated velocities ranged from 4 ft/sec to 5.3 ft/sec.

NRBS ran the uncalibrated HEC-RAS model for the 167 cfs discharge and compared the resulting modeled water surface elevation profile with the measured water surface elevation profile for the 2000-foot long reach upstream of El Centro Avenue. Figure 37 is a plot of this comparison and shows that the model predictions using a uniform roughness of 0.035 for the channel and overbank areas are reasonable between Station 1600 and Station 1800. NRBS determined that this was consistent with the relationship between the relatively homogeneous channel conditions in this area and the uniform roughness of 0.035 in the uncalibrated model. Therefore, roughness values were not adjusted for this portion of the model. The first two hundred feet upstream of the culvert (Stations 0 through 200) were not used in this calibration because backwater effects from the culvert made identification of reliable high water marks impossible.

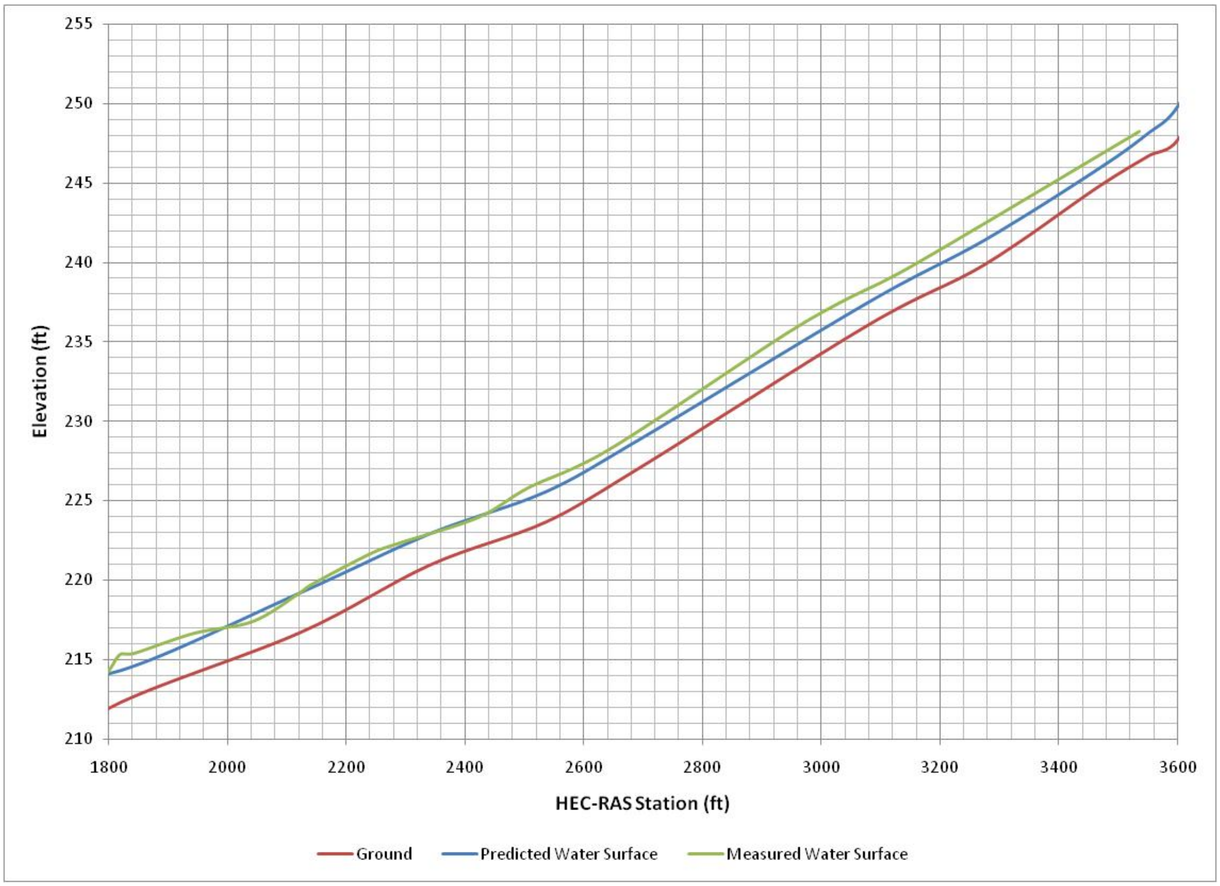


Figure 37: Comparison of model predicted and measured water surface elevations upstream of El Centro Avenue for the 167 cfs discharge on October 19, 2009 using the uncalibrated HEC-RAS model. Note the consistently high predicted water surface elevations upstream of Station 1800.

Figure 37 also shows that the modeled water surface elevations were consistently lower (by approximately one foot) than the measured water surface elevations upstream of Station 800. NRBS determined that this was consistent with the discrepancy between the more complex channel conditions in these reaches and the uniform roughness of 0.035 in the uncalibrated model. Therefore, roughness values were adjusted in the model to achieve a better fit between predicted and measured water surface elevations. Initially, NRBS tested the sensitivity of water surface elevation to changes in the overbank roughness values. There was no sensitivity to overbank roughness because flow is confined to the main channel during the calibration discharge of 167 cfs. Next, more appropriate roughness values for the calibration cross sections were estimated using guidance from the United States Geological Survey (USGS 1990). Final roughness values varied from 0.075 in typical reaches of the channel to 0.1 for reaches with large sheet-pile flow obstructions in the channel. Figure 38 shows the improved fit between modeled and measured water surface elevations in the final calibrated HEC-RAS model. The typical channel roughness of 0.075 was also applied in model reaches outside of the calibration reach where no additional site specific information was available. It is important to note that roughness likely changes with changes in discharge. However, because the primary focus of this analysis was on the relatively frequent peak discharges that are the most important with respect to habitat and sediment transport, roughness values were not varied with flow in the final model runs. Future efforts could extend the range of calibration flow measurements and improve model performance for higher flows.

As a final check on the performance of the calibrated model, flow velocities measured during the October 19, 2009 peak discharge were compared with predicted velocities from the calibrated model run at 167 cfs. Figure 39 is a plot of this comparison. Most of the modeled velocities are within the range of measured velocities. While two velocity predictions are significantly greater than the range of measured velocities, NRBS did not feel this indicated a significant problem in the model construction, as the measured velocities were averaged over long distances and the modeled velocities are calculated as specific cross sections where local geometry and slope conditions could produce locally elevated velocities.

Existing Hydrologic Conditions

Aggregated peak discharge at the outlets of Shephard Creek, Cobbledick Creek, Palo Seco Creek, Sausal Creek, and other specific locations along the main channel are provided in Table 14. Water surface profiles through the Dimond Canyon reach are provided in Figure 40.

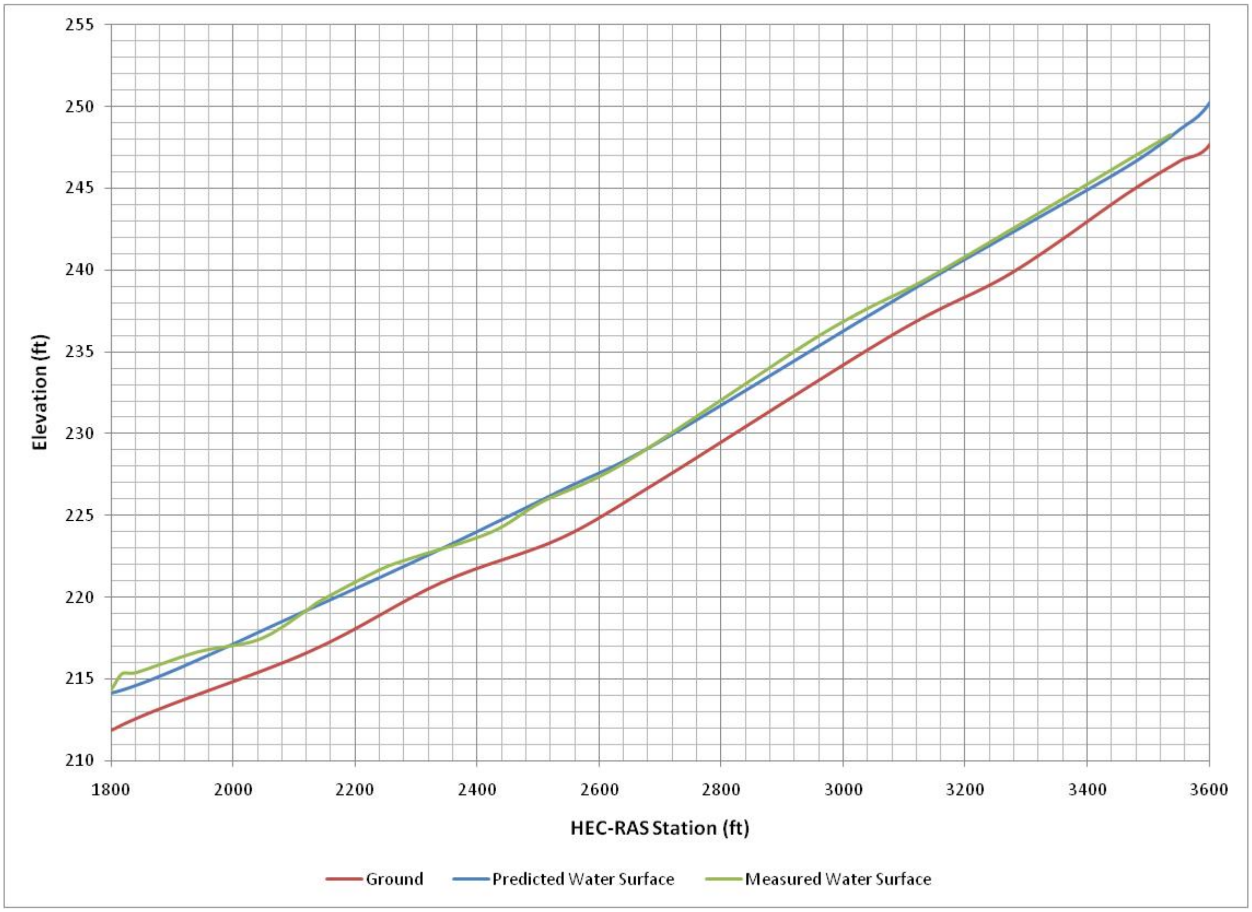


Figure 38: Comparison of model predicted and measured water surface elevations upstream of El Centro Avenue for the 167 cfs discharge on October 19, 2009 using the calibrated HEC-RAS model. Note the improved fit between measured and predicted water surface elevations upstream of Station 1800.

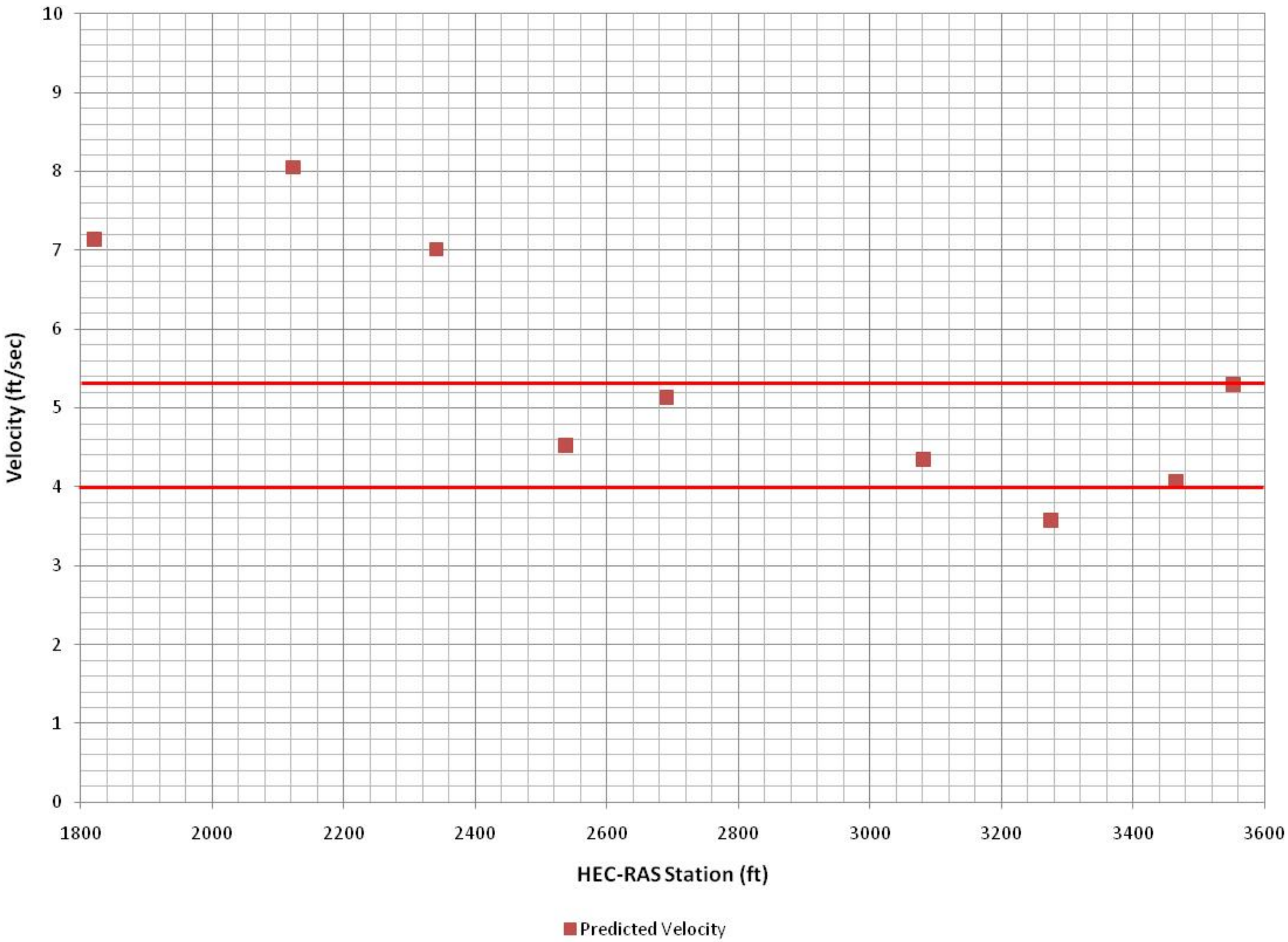
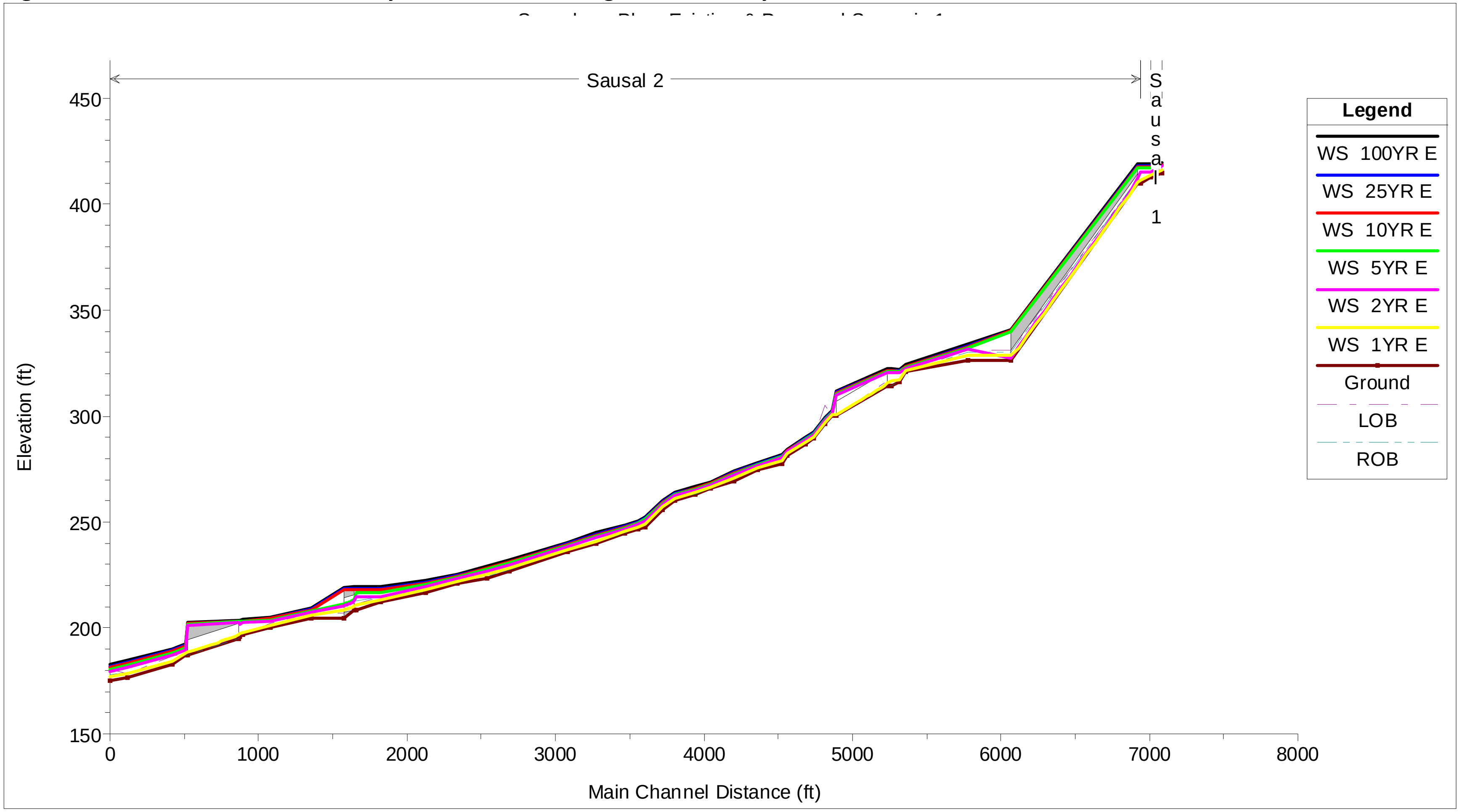


Figure 39: Comparison of model predicted and measured velocities for a discharge of 167 cfs. Red squares are model predicted velocities at specific model cross sections. Red lines represent the upper and lower bounds of the average velocity measurements. The average velocity measurements were conducted in a 100-foot reach upstream of El Centro Ave. that was determined to be representative of flow conditions throughout the modeled reach.

Table 14: Existing Conditions Computed Peak Discharge at Selected Locations along Sausal Creek and Tributary Channels

PEAK RUNOFF (CFS)						
Storm Return Frequency						
Reach	**100 Year**	**25 Year**	**10 Year**	**5 Year**	**2 Year**	**1 Year**
Shephard Creek Culvert Outlet downstream of Shepherd Park (Node SC-F / ShepParkOutlet)	277.92	214.17	176.64	142.91	97.07	17.08
Cobbledick Creek below Larry Lane Basin at Culvert Outlet at Scout Rd. and Ascot Dr. (Node 47)	74.43	57.58	47.19	39.09	27.12	4.90
Outlet of Cobbledick Creek (in pipe) at Confluence with Shephard Creek (in pipe) under Hwy. 13 (Node 30)	138.9	108.4	91.9	76.3	53.9	9.35
Outlet of Shepherd Canyon upstream of Confluence with Sausal Creek (Node SC-R)	472.3	312.5	254.5	208.7	142.0	26.46
Outlet of Palo Seco Creek just upstream of Confluence with Sausal Creek (Node 121)	133.2	100.8	80.2	64.9	44.8	6.72
Shephard Creek Upstream of Palo Seco Creek Confluence (Node 32)	610.9	420.7	346.3	284.9	195.7	35.6
Inlet to the Golf Course Culvert (Node 52)	744.0	521.3	426.3	349.8	240.4	42.26
Outlet of Dimond Park (Node 163)	1106.1	913.1	789.7	538.3	363.2	58.85
Outlet to the Bay (Node 179)	1122.5	881.7	735.2	592.4	408.6	60.12

Figure 40: Sausal Creek Watershed: Hydraulic Profile through Dimond Canyon



Existing Conditions Hydraulic Analysis

Figures 41 and 42 are longitudinal plots of existing velocity and shear stress, respectively, in the modeled reaches of Palo Seco Creek and Sausal Creek for the 1, 2, 5, 10, 25, and 100-year discharges. All of the tabular data in these plots is summarized in the HEC-RAS output tables in Appendix H. Both plots illustrate that existing hydraulic conditions in Sausal Creek could lead to geomorphic instability, especially for 2-year discharges and above.

Table 15 summarizes permissible velocities and shear stresses for bed and bank sediment and vegetation similar to conditions in Sausal Creek. Channel bed and bank stability varies throughout the Sausal Creek watershed, however, in general, the channel bed and banks are likely stable for velocities up to approximately 7 ft/sec and shear stresses up to approximately 2 lbs/ft^2 (Fischenich 2001). NewFields used the US Army Corps of Engineers' Engineering Research and Development Center "Stability Thresholds for Stream Restoration Materials" to determine permissible shear strengths reference for selection of stream restoration materials, and a useful guide to assessing the stability of existing stream materials. While conditions vary throughout the watershed, we made the assumption that the system has geomorphic controls such as riffles and artificial grade control structures with cobble and coarser particle sizes and some woody vegetation on the banks. As shown in Table 15, the reaches with these characteristics would be stable for velocities up to 7 feet per second and shear stresses up to 2 pounds per square foot. Therefore, portions of the Sausal channel network are at risk of erosion under existing hydrologic and hydraulic conditions, even during the 1-year discharge. Even moderate reductions in peak flows could reduce the frequency and duration of erosive flows and contribute to long-term improvements in creek habitat condition.

Table 15: Permissible velocities and shear stresses for channel sediment and vegetation types similar to Sausal Creek (after Fischenich 2001)

Material	Permissible Velocity (ft/sec)	Permissible Shear Stress (lbs/ft^2)
Gravel (2 inch)	3.0 – 6.0	0.67
Cobble (6 inch)	4.0 – 7.5	2.0
Riprap (18 inch)	12.0 – 16.0	7.6
Emergents	n/a	0.1 – 0.6
Grasses	3.0 – 6.0	0.7 – 1.7
Woody Vegetation	3.0 – 10.0	2.1 – 3.1

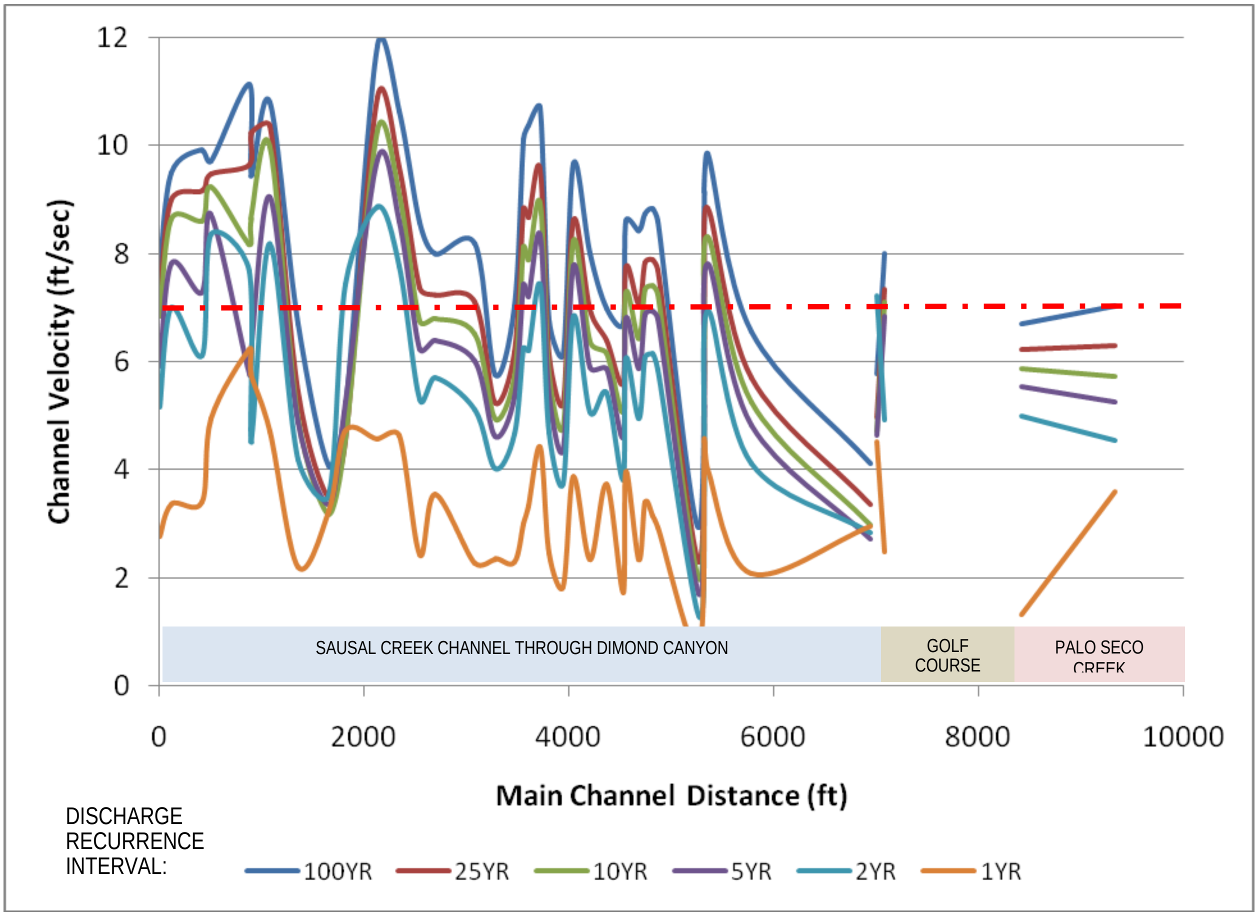


Figure 41: Existing conditions longitudinal velocity plot. Dotted red line signifies approximate stability threshold for typical Sausal Creek sediment and vegetation characteristics. Velocities and shear stresses are largely controlled by channel geometry in this portion of Sausal Creek, with high velocities and shear stresses in narrow reaches.

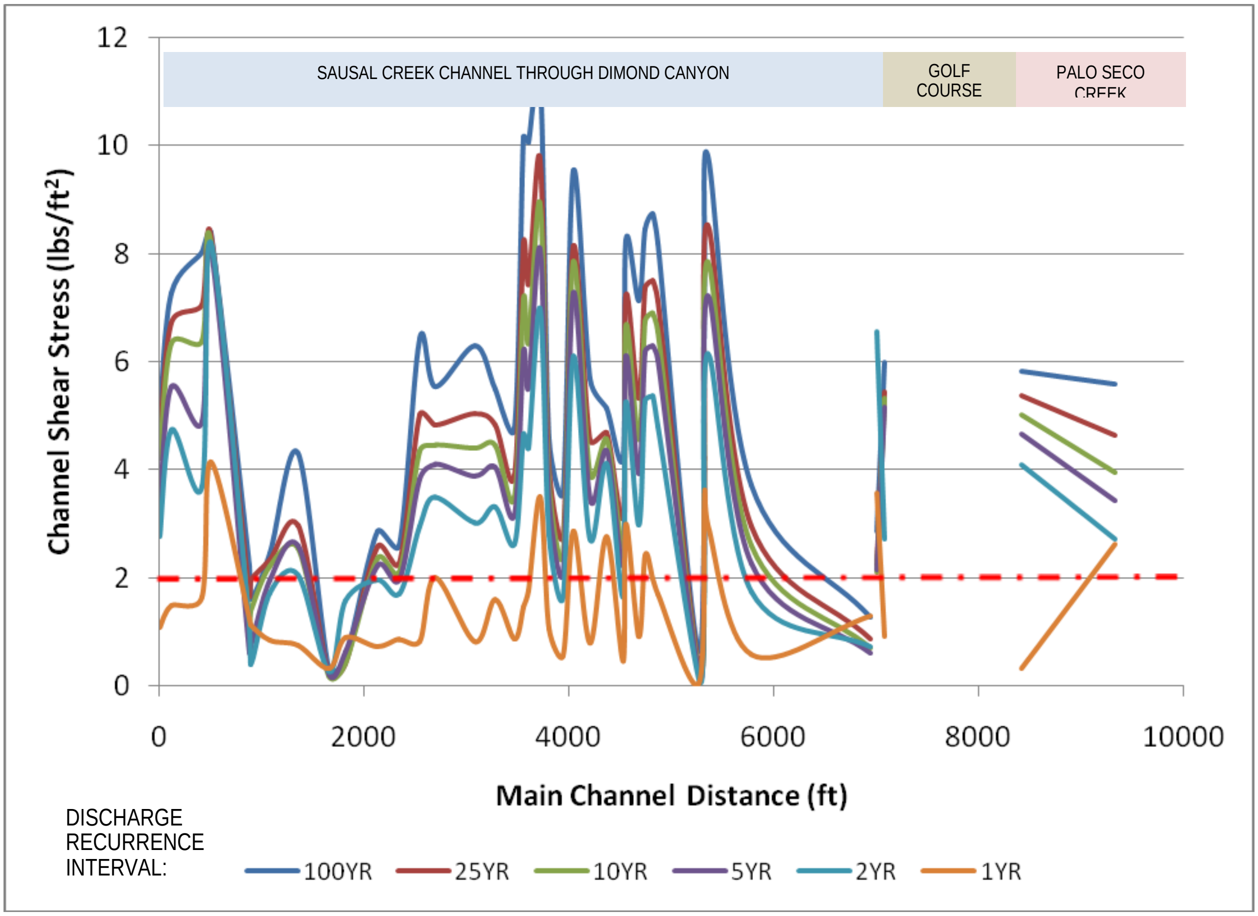


Figure 42: Existing conditions longitudinal shear stress plot. Dotted red line signifies approximate stability threshold for typical Sausal Creek sediment and vegetation characteristics. Velocities and shear stresses are largely controlled by channel geometry in this portion of Sausal Creek, with high velocities and shear stresses in narrow reaches.