Belmont Creek Watershed Study, Creek Assessment, and Recommendations for Sustainable Improvements
San Mateo County, California

Technical Study

Prepared for:

NOVARTIS

Prepared by:

WRECO

September 2014

Belmont Creek Watershed Study, Creek Assessment, and Recommendations for Sustainable Improvements
San Mateo County, California

Technical Study

Submitted to:
Novartis Pharmaceuticals Corporation

This report has been prepared by or under the supervision of the following Registered Engineer. The Registered Civil Engineer attests to the technical information contained herein and has judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.

____
Grant Wilcox, P.E., P.G., CEG

____
Date

September 2014

Table of Contents

Figures

Tables

Photos

Appendices

Contributing Staff

Dr. Han-Bin Liang – Project oversight, alternatives development
Grant Wilcox, P.E., P.G., CEG – Project management, alternatives development
David Mueller, P.E. and Jennifer Abrams, P.E. – Hydrologic and hydraulic modeling, model calibration
Alvin Yim, Cuyler Stapelmann, and Flannery Banks – Geomorphic field assessments, water quality and sediment characterization, technical report preparation

1 INTRODUCTION

This Technical Study has been prepared to determine feasible flood control alternatives for Belmont Creek that would reduce flooding at the Novartis Pharmaceuticals Corporation (Novartis) facility located at 150 Industrial Road in the City of San Carlos, California. As such, the Belmont Creek Watershed Study, Creek Assessment, and Recommendations for Sustainable Improvements Project (Project) studied flood control measures to mitigate the flooding that has historically occurred at the Novartis facility. See Figure 1 for the Project location map and Figure 2 for the Belmont Creek watershed map.

1.1 Project Background

On October 29, 2012, WRECO prepared a Technical Study Memorandum regarding dredging sections of Belmont Creek adjacent to the Novartis facility in the City of San Carlos. The study evaluated the hydraulic effects of dredging Belmont Creek from the upstream property line to the downstream property line at the Industrial Road bridge. The study determined that dredging approximately 500 cubic yards (cy) from Belmont Creek would increase the creek capacity from approximately 630 cubic feet per second (cfs), equivalent to a 4-year storm event, to approximately 750 cfs, equivalent to an 10-year storm event. To remain effective, dredging would be required approximately every 10 years. Moreover, dredging would need to be performed downstream of the Novartis facility, from the culvert at Industrial Road to the cross culvert at US 101. WRECO was requested to expand the study to include the entire watershed of Belmont Creek to determine additional measures within the diffuse watershed that could be implemented to further reduce the flooding at the Novartis facility.

1.2 Project Objective

This Project will investigate areas where hydraulic constrictions occur and flood backwater flows into developed areas; determine which agencies and property owners would need to be involved to accomplish this goal; develop effective and sustainable flood control solutions; and analyze potential constraints, such as drainage-related and permitting issues, in enough detail such that schedules and general costs could be estimated. This report includes the following:

- Review of existing information and previous studies pertaining to Belmont Creek
- Summary of geomorphic assessment and sediment sources
- Hydrologic and hydraulic analysis of existing conditions and proposed flood control measures
- Alternatives analysis of proposed flood control facilities

Additionally, this study is anticipated to provide a watershed management tool for stakeholders and result in improvements to riparian habitat and water quality.

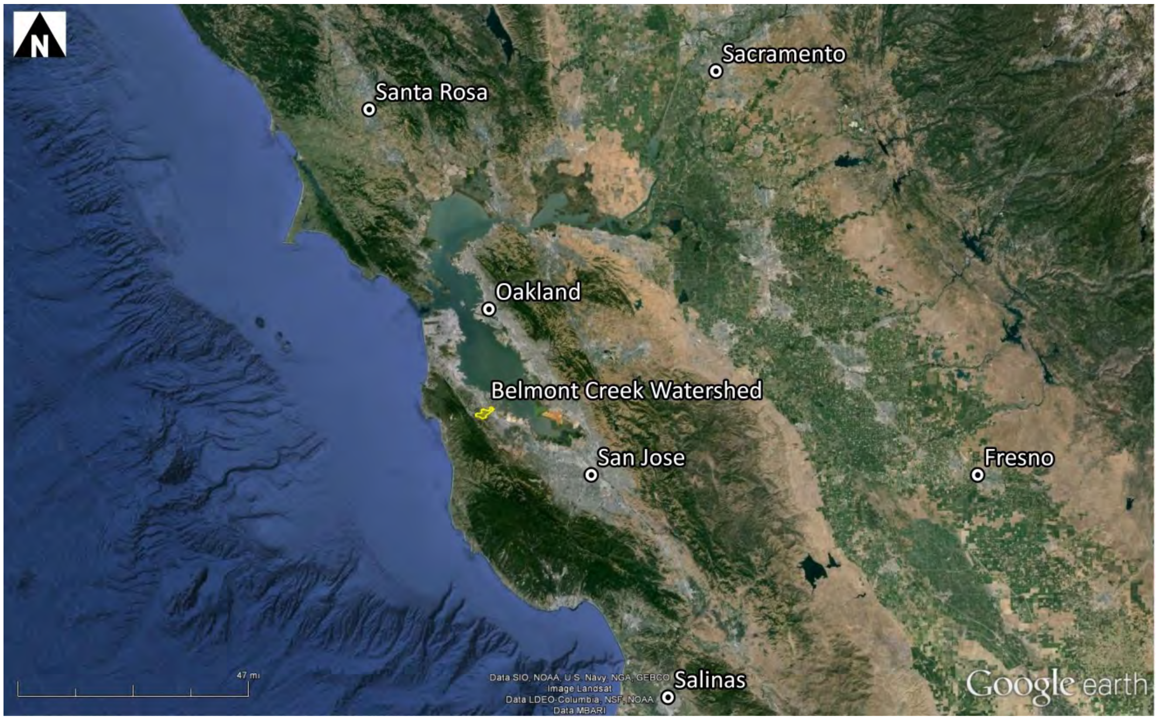

Figure 1. Project Location Map

Source: EPA My Waters (watershed)

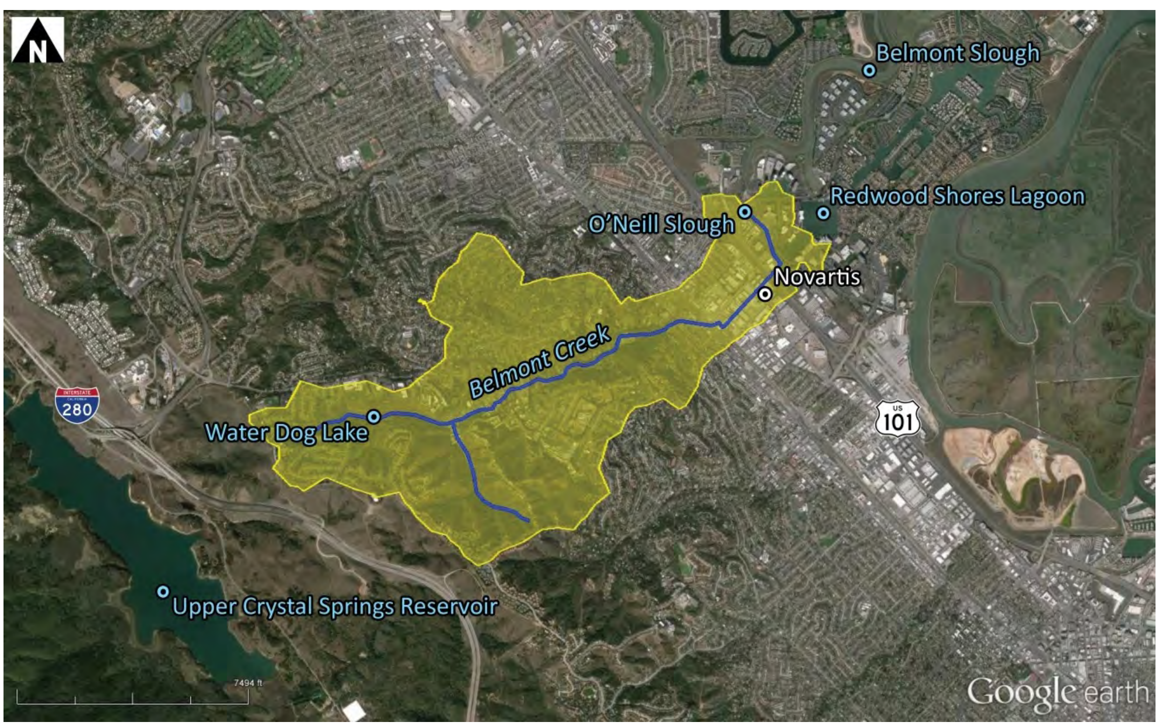

Figure 2. Belmont Creek Watershed Map

Source: EPA My Waters (watershed)

1.2.1 Stakeholders

Though other local businesses and property owners will likely benefit from reducing the flood risk of Belmont Creek, the following entities have been identified as key stakeholders for the Project:

- Novartis
- City of San Carlos
- City of Belmont
- City of Redwood City
- San Mateo County
- Regional Water Quality Control Board (RWQCB), San Francisco Bay
- California Department of Transportation (Caltrans)
- Caltrain

1.3 Previous Studies and Reports

The previous studies and reports described in the following subsections were reviewed in support of the Project.

1.3.1 Harbor Industrial District Storm Drain Report

In a 1998 paper titled the *Harbor Industrial Area Storm Drain Report*, BKF Engineers evaluated the existing drainage facilities serving the 134-acre area bounded by Old County Road, Belmont Creek, US 101, and Holly Street. At that time, the City of San Carlos had recently annexed this area, which had experienced flooding in the past and is at an elevation lower than the banks of Belmont Creek, exacerbating flooding issues.

The report recommends the addition of culverts at the Old County Road, Industrial Road, and US 101 crossings to increase conveyance capacity; sediment removal from the channel to restore channel flow capacity; flood wall construction to prevent overbank flows; and open channels to provide additional drainage in the Harbor Industrial District.

1.3.2 Imperviousness and Channel Modifications for 17 Watersheds

In 2002, the San Mateo Countywide Stormwater Pollution Prevention Program (STOPPP) characterized watershed imperviousness and creek channel modifications in 17 watersheds in San Mateo County. The report was focused on major urban creeks experiencing development pressure in order to assist municipal planners in minimizing the impacts of future development on creek resources. According to the report’s findings, impervious surfaces comprise approximately 42% of the Belmont Creek watershed while only about 26% of the creek channel remains unmodified. The report recommends minimizing increases in imperviousness, especially connected imperviousness associated with new and redevelopment projects, and prioritizing imperviousness minimization in relatively undeveloped areas with unmodified channels.

1.3.3 Unified Stream Assessment in Six Watersheds

In 2006, STOPPP performed creek walks in six watersheds, including Belmont Creek, using the Unified Stream Assessment protocol developed by the Center for Watershed Protection. The primary objective was to characterize physical features of the creek channels and associated riparian corridor as part of screening-level water quality monitoring activities. (EOA Inc. 2007)

The report found that approximately 50% of the creek has been modified, mostly within the downstream reaches, primarily through bank hardening and culverting; a majority of creek crossings have a hard bottom that act as grade control structures; and most of the erosion within the channel can be classified as bank scour and bank failure, primarily within the urbanized areas. (EOA Inc. 2007)

1.3.4 Biological Assessment of Belmont Creek

In 2006 and 2007, STOPPP evaluated the general health and physical habitat of Belmont Creek using the California Stream Bioassessment Procedure to characterize benthic macroinvertebrate (BMI) communities. BMIs are often used to indirectly monitor water quality and creek health by measuring their abundance, taxonomic diversity, and community structure, as they are highly responsive to changes in their aquatic environment. The BMIs collected from Belmont Creek were characterized as moderately pollutant-tolerant with low species richness and diversity at all sampled sites across both years. The taxa identified within the samples were relatively short-lived, taking less than one year to complete a life cycle; this is indicative of an intermittent flow regime, which can contribute to low quality BMI communities. (BioAssessment Services 2007)

1.3.5 Belmont Creek Hydraulic Analysis

In 2010, Schaaf and Wheeler prepared the *Belmont Creek Hydraulic Analysis* memorandum as part of a sediment removal project along Belmont Creek from Old County Road to Harbor Boulevard. The memo explains that this portion of Belmont Creek had aggraded during the previous two years, allowing Old County Road to be overtopped during a 280 cfs flow. The channel downstream of Old County Road has two sharp 90-degree bends, and a capacity of approximately 350 cfs before it is overtopped. The channel between the Caltrain tracks and Old County Road, east of El Camino Real, has a capacity of nearly 1,000 cfs with current sediment levels, without the downstream restrictions of Old County Road and the downstream channel. The Federal Emergency Management Agency's (FEMA's) 10-year and 100-year flow rates at nearby El Camino Real are 570 cfs and 1,200 cfs, respectively.

1.3.6 Pollutants of Concern Monitoring Data

In 2012, a technical report prepared for the San Francisco Estuary Institute's Regional Monitoring Program titled *Pollutants of Concern Monitoring Data, Water Year 2011* assessed 17 small- to medium-sized watersheds with varying land uses to support management decisions regarding mercury and polychlorinated biphenyls (PCBs) load reductions. Of the 17 watersheds studied, Belmont Creek was ranked third cleanest based on mean suspended sediment normalized PCB concentrations and eighth cleanest based on mean suspended sediment normalized mercury concentrations. In decreasing order, the PCB congeners most associated with Belmont Creek include Aroclor 1260, Aroclor 1254, Aroclor 1248, and Aroclor 1242. Aroclors 1260 and 1254 were the main formulations used in electrical equipment manufacturing prior to 1950; Aroclor 1242 was the primary formulation used during the 1950s and 1960s, until it was replaced by Aroclor 1016 in 1971.

The report concludes that management efforts in high-leverage watersheds (those with the highest potential load reductions) are more cost-effective than efforts in low leverage watersheds. In general, watersheds with industrial and urban areas active between 1950 and 1990 are high-leverage for PCBs and mercury.

1.3.7 Other Studies

- *Belmont Creek Watershed Monitoring Report* by Kinnetic Laboratories Incorporated dated August 9, 2006.
- *Joint Stormwater Agency Project to Study Urban Sources of Mercury, PCBs and Organochlorine Pesticides* by EOA, Inc., and Kinnetic Laboratories Incorporated, dated April 2002.
- *Trash Assessments in Six Watersheds in San Mateo County, California* by STOPPP, dated August 2007.

2 CREEK ASSESSMENT METHODS

2.1 Desktop Investigation

2.1.1 Historical Review

The history of Belmont Creek was investigated to elucidate the historical and active fluvial dynamics and conditions observable within the creek channel and watershed. Moreover, it was unclear at the start of the Project whether flooding in the lower watershed was a historical nuisance or a function of urban development. This effort included reviewing historical cartography and United States Geological Survey (USGS) topographic maps, aerial imagery, municipal documents, and public agency archives as well as primary accounts and anecdotal evidence.

2.2 Field Investigation

2.2.1 Creek Walk

On January 30, 2014, a team of WRECO staff walked the length of Belmont Creek, excluding tributaries, between US 101 and the outfall of Water Dog Lake to document the existing conditions. The following subsections describe methods employed during the creek walk.

2.2.1.1 Longitudinal Surveys and Cross Sections

WRECO staff collected preliminary longitudinal profile and cross sectional information during the creek walk on January 30, 2014. Surveys were performed at two locations: the headwall of the culvert located at O'Neill Avenue and 6th Avenue and just downstream of the Caltrain bridge. As described below, this survey information was utilized to generate stream flows from velocity measurements. This flow information was used in the calibration of hydrologic and hydraulic models developed for the Project.

RSE, Inc. was tasked with surveying Belmont Creek in order to develop cross sections for the hydraulic model. The survey was taken along the creek at various intervals from the upstream end of the US 101 box culvert through Chula Vista Drive and College of Notre Dame.

2.2.1.2 Photo Documentation

Photos were taken during the creek walk to document the existing conditions and active channel dynamics of Belmont Creek. A selection of these photos is provided in Appendix A.

2.2.2 Flow Measurements

Flow measurements were made to calibrate the hydrologic and hydraulic models prepared for Belmont Creek. The following subsections describe these measurements.

2.2.2.1 Velocity Measurements

Flow velocities in Belmont Creek were measured with an FP101 and FP211 Global Water Flow Probe. The probe consists of a free-spinning propeller at the end of a telescoping rod with a digital display that provides instant average and maximum flow readings. To collect flow data, the propeller is submersed in water and the number of revolutions per second is translated into a velocity (ft/s).

On February 2 and February 6, 2014, flow velocities were measured at the headwall of the culvert located at O'Neill Avenue and 6th Avenue (reach B1) and just downstream of the Caltrain bridge (reach B7), which is located near the intersection of El Camino Real and Harbor Boulevard. The nearest weather station to the Project with National Weather Service data is station KSFO located at San Francisco International Airport. According to station KSFO, 0.85 in. of rain fell on February 2 and 0.64 in. of rain fell on February 6.

2.2.2.2 Pressure Measurements

Time series data of the runoff caused by the precipitation event on February 28, 2014, was developed for use in the calibration of the hydrologic model. The water depth was measured using a Solinst Levelogger. The Levelogger measures pressure data at a time interval dictated by the user. The device is set up and the data are retrieved using a wired connection to a computer with Solinst software. A Solinst Barologger was used to compensate for atmospheric pressure measurements by the Levelogger. The pressure measured by the Barologger is subtracted from the pressure measured by the Levelogger, yielding the pressure due to flow in the creek. The pressure is converted into ft of water, which is calibrated to depth measurements taken in the stream at a specific time. The correlation between the observed depth measurements and the Levelogger depth measurements is shown in Figure 3. The offset between the Levelogger base elevation and the channel bottom was approximately 0.44 ft.

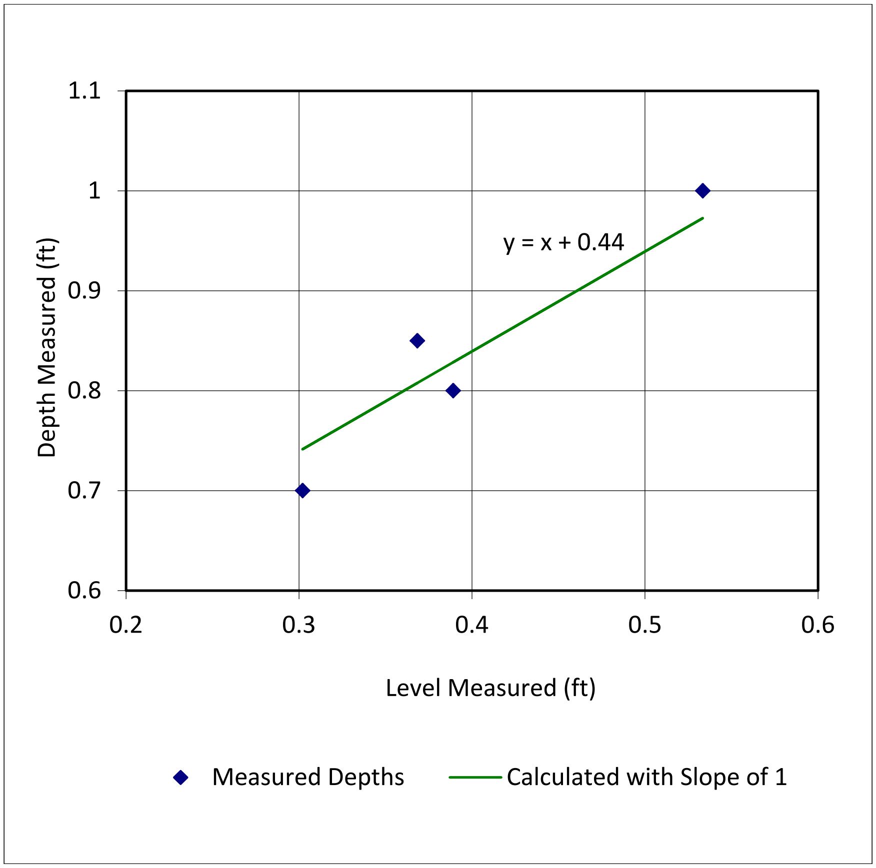

Figure 3. Relationship between Levelogger and Observed Depths

The Levelogger was suspended in the stream using a 3-in. diameter plastic perforated pipe attached to a piece of rebar inserted into the creek bed. The system is shown in Figure 4, when the water depth was approximately 0.8 ft. In the morning, after the event, some debris had collected on the upstream side of the pipe, which may have altered the water surface elevation slightly. There is an abrupt change in the water surface measurements around the time that the debris was removed, but it was smaller than the uncertainty in the depth used for the flow rate calculation (below). Therefore, the interference due to the debris was considered insignificant.

Figure 4. Perforated Pipe with Levelogger Inside

Based on these observations, the maximum depth reached by the flow overnight was 2.0 ft. This matched with observations of the water line in the field. The depths measured over the monitoring period are shown in Figure 5. The constant depths before the first rise in the hydrograph are because the Levelogger was suspended above the channel bed and could not measure depths below itself.

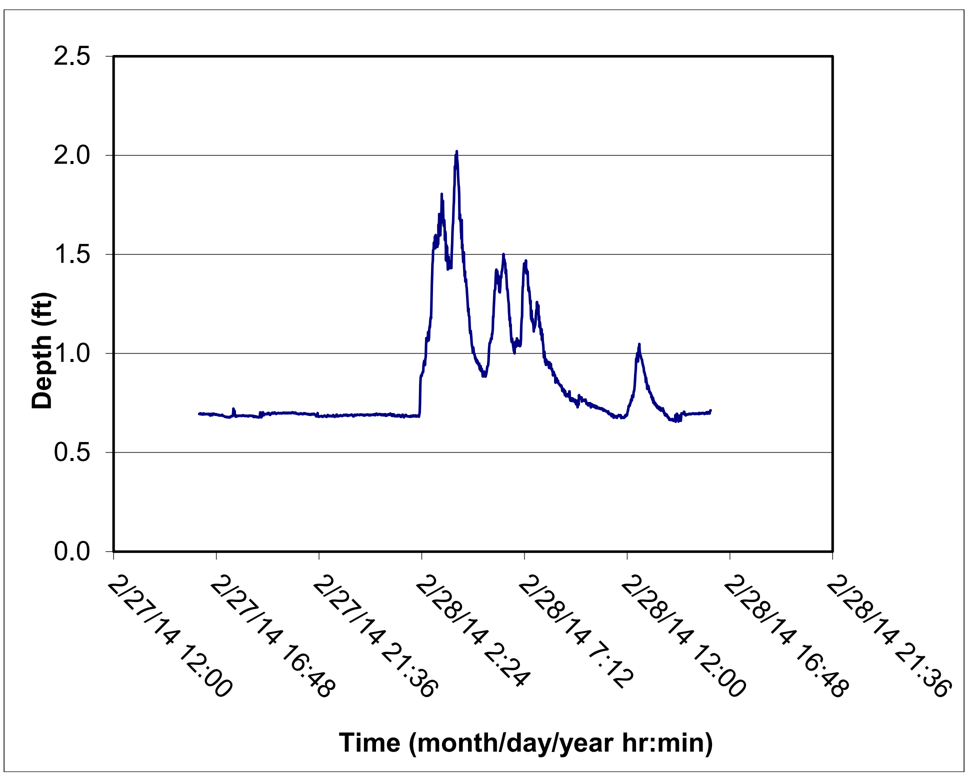

Figure 5. Depths Observed During Monitoring Period

2.2.2.3 Creek Cross Sections

The geometry of the creek at the time and location of the pressure measurements was calculated by suspending a tape measure across the creek and measuring the distance from the ground surface to the rod. The measurements were not corrected for sagging in the rod. The cross section was taken approximately 12 ft downstream of the gage because of the difficulty in accessing the fence to tie the tape measure at the bank next to the depth gage. Figure 4 shows the setup that was used, and the cross section is shown in Figure 6.

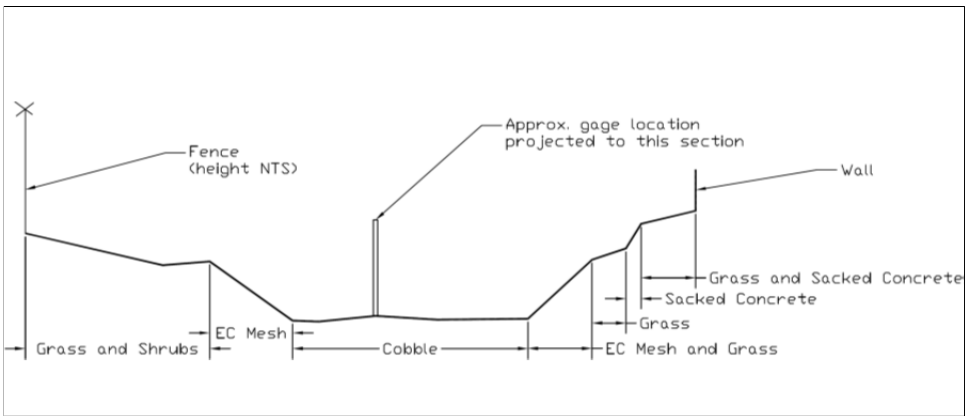

Figure 6. Channel Cross Section near Levelogger (No Scale)

Notes:
EC = erosion control
NTS = not to scale

2.2.2.4 Flow Rate Calculation

The flow rate in the creek at a known depth and time was calculated by dividing the creek width into 2- to 3-foot segments and measuring the average velocity in each segment. The average velocity was calculated by raising and lowering the Global Water Flow Probe slowly until the average velocity reading stabilized, approximately 30 seconds to 1 minute later. The flow rate in each segment was calculated by multiplying the cross section area within the segment by the average velocity. The total flow rate in the channel was approximately equal to the sum of the flow rates in each segment.

The velocity measurements were performed twice during the morning after the precipitation event, when flows were approximately 0.9 ft deep. The resulting flow rate was 37 cfs.

Figure 7. Installation of Levelogger in Belmont Creek

2.2.3 Water Quality Measurements

On March 26, 2014, WRECO collected surface water grab samples from Belmont Creek during a storm event that produced 0.58 in. of precipitation at the Belmont Canyons weather station (KCABLEMO11) and 0.36 in. of precipitation at the nearest weather station with official National Weather Service data (KSFO). Samples were collected near peak flow to assess sediment loads and sources associated with stormwater flows. Samples were analyzed for Total Suspended Solids by Curtis & Tompkins Environmental Laboratory. WRECO also measured general water quality parameters with a YSI Model 63 instrument at each sample location immediately after collection; these parameters included temperature, pH, conductivity, specific conductance, and salinity. The YSI Model 63 was calibrated prior to field implementation with a 3-point test using National Institute of Standards and Technology solutions.

In addition, WRECO monitored general water quality parameters at both crossings of Shoreway Road over Belmont Creek during a flood tide, peak high tide, and ebb tide. One location was at Shoreway Road near Marine Parkway and the other was at Shoreway Road adjacent US 101. These data were collected in order to establish baseline salinity of this reach of Belmont Creek as Project proposals could modify salinities in this area, which could affect the distribution of existing freshwater and salt marsh habitats in lower Belmont Creek.

2.3 Creek Reach Delineations

For the purpose of the geomorphic assessment Project, Belmont Creek was divided into the reaches delineated in the Unified Stream Assessment report (EOA Inc. 2007); see Table 1 and Figure 8. The reaches were divided in this way so observations and data generated by the Project would complement and improve upon existing information. Closed-conduit systems were not identified as separate reaches.

Table 1. Belmont Creek Reach Information

Reach	Boundary of Reach[1]	Length[2] (ft)	Notes
B1	6th Street to Chula Vista Drive	2,900	--
B2	Chula Vista Drive to upstream of Maywood Drive	3,100	--
B3	50 ft upstream of Maywood Drive bridge to culvert at south end of shopping center	700	Not included in creek walk.
B4	Live Oak culvert to Water Dog Lake	1,900	--
B5	Upstream of Water Dog Lake	3,000	Not included in creek walk.
B6	Baylands	2,700	Not included in creek walk.
B7	US 101 to El Camino Real	2,700	--
B8	Open Space	2,060	Not included in creek walk.

[1] Boundaries of reaches are identified per the EOA (2007) study. Reaches not defined in the EOA report are defined only in this study.
[2] Lengths are approximate and taken from Google Earth

Source: EOA Inc. 2007

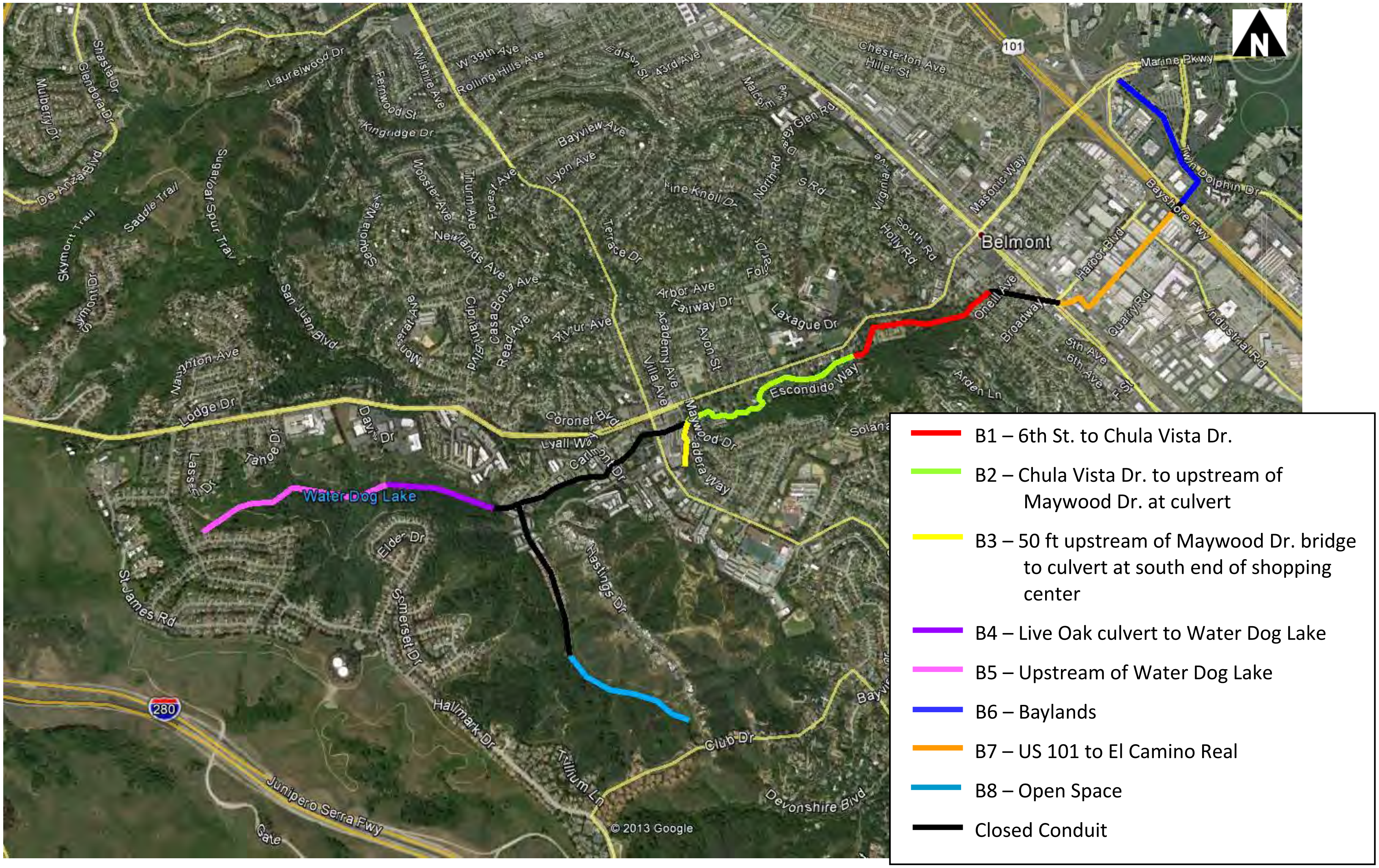

Figure 8. Belmont Creek Reaches

Source: Google Earth

2.4 Modeling

2.4.1 Hydrologic Modeling Approach

The hydrologic model of the Belmont Creek watershed was developed with Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software. Hydrologic modeling was performed to establish sub-watershed areas that could be linked together to more accurately identify the flows and times of concentration coming from different areas within the watershed. The Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) was used in ESRI's ArcMap to aid in the delineation of watersheds. In addition, calculations for various watershed and reach properties were performed using ArcMap and HEC-GeoHMS.

Following the development of the watershed data in ArcMap, the model was imported to HEC-HMS using HEC-GeoHMS. The Initial and Constant loss method and Soil Conservation Service (SCS) Unit Hydrograph transform methods were used for the watersheds, and the Muskingum-Cunge method was used for reach routing. Precipitation information was used in HEC-HMS to calculate the hydrographs at several locations within the watershed, which are utilized in the hydraulic model. Monitored flow data adjacent to the Novartis property from the storm event on February 28, 2014, were used to calibrate the model. The hydrographs to be used in the hydraulic model were saved as time series, and imported into PCSWMM.

2.4.2 Hydraulic Modeling Approach

A two-dimensional (2D) hydraulic model was developed in PCSWMM based on survey data, where available, and a digital elevation model (DEM). The hydraulic model focuses on the reach between El Camino Real and the discharge point to Belmont Slough; however, the model also includes areas higher in the watershed that are of use for evaluating some of the alternatives. The areas west of El Camino Real upstream were evaluated using a one-dimensional (1D) model. Hydrologic inputs from the HEC-HMS model were used in PCSWMM. Channels, storm drains, and bridge crossings were modeled as conduits in the PCSWMM model. A 2D mesh was added in areas within the FEMA floodplain. The model was run with various input conditions to represent the existing condition and various alternatives. Calibration data were not available for the hydraulic model.

3 CREEK ASSESSMENT

3.1 Watershed Description

Belmont Creek has a dendritic watershed of approximately 1,952 acres (3.1 sq mi) that originates east of the Pulgas Ridge in the hills above Hallmark Drive. The creek runs approximately parallel to Ralston Avenue through Water Dog Lake and Twin Pines Park. The creek exits the City of Belmont upstream of Old County Road at Harbor Boulevard, then forms the boundary between the Harbor Industrial Area and the City of San Carlos. It reenters the City of Belmont in the Island Park neighborhood, and then flows through O'Neill and Belmont sloughs before discharging to San Francisco Bay. In addition, there are several substantial tributaries to Belmont Creek from the side canyons at Carlmont Drive, Alameda de las Pulgas, and University of Notre Dame de Namur (City of Belmont 2004).

Belmont Creek passes through public and private lands, undeveloped open channels and closed conduits, and accessible parks and open spaces as well as inaccessible residential neighborhoods. Elevations within the study area vary from approximately sea level at the US 101 crossing to an elevation of approximately 2,050 ft near the outfall from Water Dog Lake. At the US 101 crossing, the annual mean precipitation is 18 inches, whereas the annual mean precipitation at the outfall of Water Dog Lake is 24 inches.

3.1.1 Geologic Setting

The Belmont Creek watershed is located within the northwest-southeast trending Coast Range physiographic province. The hills in the upper watershed are a result of the accretion of Mesozoic oceanic crust that was once part of the now-subducted Farallon Plate onto the North American Plate. Accretion ceased roughly 30 million years ago after subduction of the Farallon Plate, and the San Andreas Fault began to form. The oceanic rocks, formally known as the Franciscan Complex or Franciscan Assemblage, are extremely faulted, fractured, contorted, and are often mixed with each other due to the accretion process. A single outcrop of the Franciscan Complex can host a wide variety of oceanic rocks.

Geology in the Belmont Creek channel and near the outlet near San Francisco Bay is classified as alluvium of Holocene age. Mesozoic Franciscan Complex geology is present at higher elevations within the watershed and in some outcroppings along the channel banks. Figure 9 presents the geologic map with an outline of the watershed in yellow. Note the Franciscan Complex chert in the present Novartis location. It was quarried out in the past and used for building materials. This is the reason for the name of the adjacent street Quarry Road.

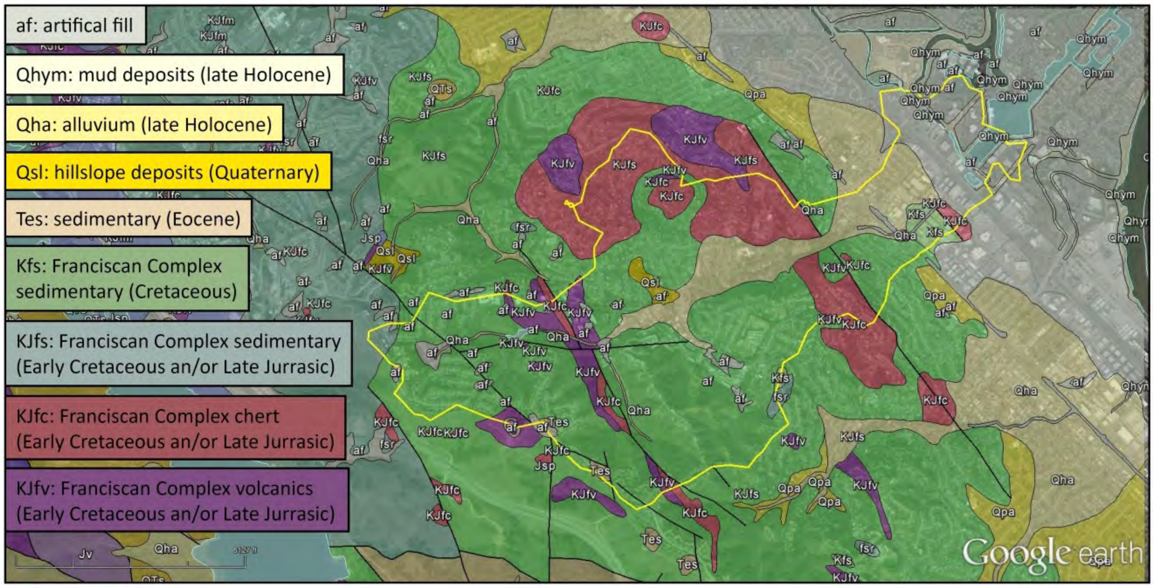

Figure 9. Geologic Map with Watershed Boundary

Source: Graymer et al. 2006

Several of the geologic units within the watershed area are prone to erosion and mass wasting, as noted by the landslide maps created by the Association of Bay Area Governments (ABAG). Sedimentary rock units within the watershed are heavily faulted and fractured, and thus have an increased hazard associated with erosion and landslides. However, the detailed landslide mapping conducted by ABAG (Figure 10) has noted that the Belmont Creek watershed hosts very few active or potential landslide threats. The dominant rock type in the watershed area is Mesozoic sandstone that is part of the Franciscan Complex. The extensive fracturing of these rocks allows substantial infiltration of water during rain events, which ascribes some landslide potential to this unit. More typically, landslides associated with the Franciscan are within the units dominated by a Shale matrix. Figure 10 presents the landslide potential within the Belmont Creek watershed.

The mapped sedimentary rock units also have the potential to contribute sediments to Belmont Creek through mechanical erosion processes, such as wind, rain, and high flows. Other units, including the Franciscan Complex chert and volcanics, are more resistant to erosion than sandstones. Evidence of historical landslides exists within the area as mapped by the USGS. Field investigations by WRECO also found several very small failures along the creek bed and areas of bank undermining that could lead to future bank failures.

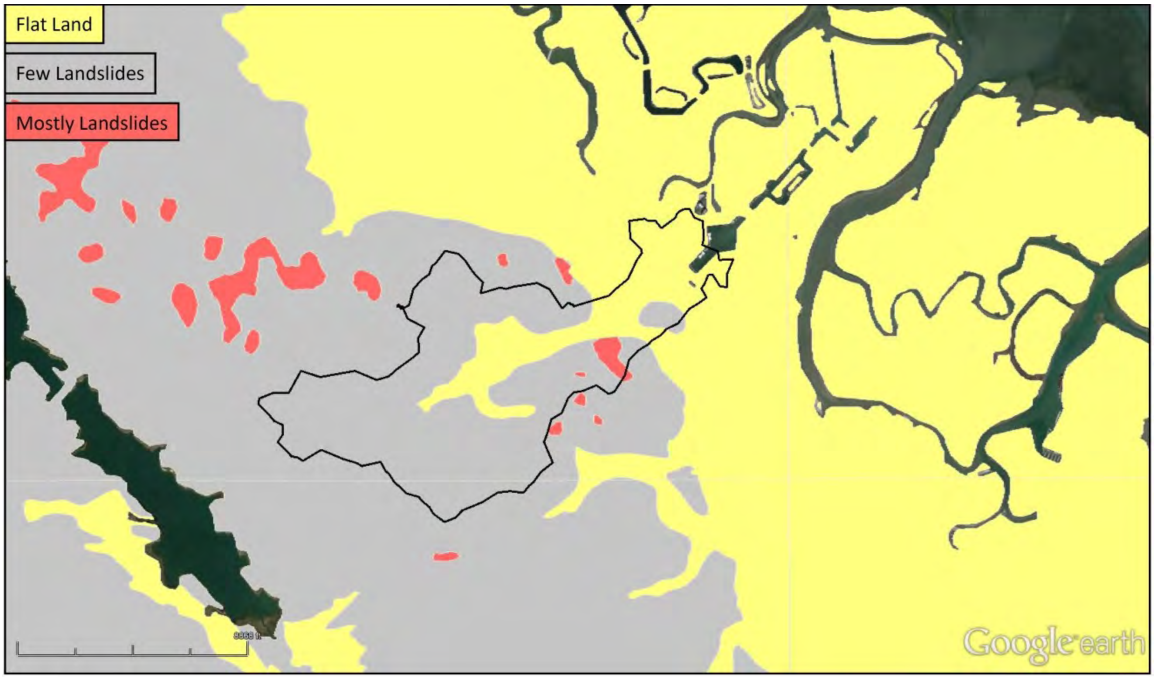

Figure 10. Landslide Potential in the Belmont Creek Watershed

Source: ABAG, Google Earth

3.2 History of Belmont Creek

Aerial imagery, historical paintings, archived topographic maps, municipal documents, and Caltrans inspection databases were reviewed to determine the historical changes to Belmont Creek. The history of Belmont Creek channel is provided below:

- By **1856** – County Road (currently Old County Road) crossed over Belmont Creek with a bridge; lower Belmont Creek had a meandering planimetric form, and a pool or cut bank is depicted at a meander bend; alluvial fan drainages existed to the south and west of Belmont Creek on Belmont Hill (Dewing 1977).
- **1865** –Belmont Creek and the tributary near the College of Notre Dame meandered through an open meadow used for agriculture, suggesting the presence of an accessible floodplain along this reach; the painting (Figure 12) also depicts a bridge over Belmont Creek (Lee 1865).
- Early **1870s** – the earthen dam at Water Dog Lake was constructed to supply William Ralston's estate with drinking water (William Lettis & Associates and San Francisco Estuary Institute 2007; Dewing 1977).
- **1888** – An alluvial fan drainage existed to the north of Ralston Avenue and to the east of the tributary referred to as "Narrow Tooth" (Belmont Creek was referred to as "Arroyo Diablo"), and it was approximately perpendicular to Alameda de las Pulgas; "the water supply [of the Mezes Ranch] is pure and ample" (Dewing 1977).

- **1896** – Channel morphology on the 1896 USGS map is consistent with Belmont Creek channel morphology on 1856 map (USGS 1896; USGS 1856), suggesting geomorphic stability in this reach for at least 40 years; channel morphology on 1896 USGS map is also consistent with morphology of the tributary near the College of Notre Dame on the 1888 map (Dewing 1977). Alameda de las Pulgas and a road near the current intersection of 6th Avenue at O'Neill Avenue crossed over Belmont Creek with hydraulic structures.
- Between **1899** and **1915** – No observable changes to channel morphology on USGS topographic maps (USGS 1899; USGS 1915).
- **1907** – A real estate poster for the Belmont Terrace subdivision states "Belmont has the Best Facilities for Natural Drainage of any Suburban Town in the Vicinity of San Francisco," "No Floods," and "Soil is a Rich, Sandy Loam;" the image on the poster shows Belmont Creek flowing through a woodland between Waltermire Street and El Camino Real (Dewing 1977).
- Between **1915** and **1939** – Belmont Creek was rerouted from its original discharge location to Belmont Channel, which is now called Redwood Shores Lagoon (USGS 1915; USGS 1939).
- Between **1915** and **1947** – Belmont Creek was culverted from O'Neill Avenue at 6th Avenue to El Camino Real at Harbor Boulevard (USGS 1915; USGS 1939; USGS 1947).
- **1930** – The culvert carrying Belmont Creek beneath US 101 was constructed (Caltrans 2011).
- Between **1939** and **1947** – The tributaries along Alameda de las Pulgas and College of Notre Dame were culverted; topographic contours indicate potential channel incision downstream of the culverted tributaries (USGS 1939; USGS 1947).
- Between **1947** and **1956** – Belmont Creek was rerouted from its previous discharge location at Belmont Channel to Belmont Slough; the 90-degree-angle bend just downstream from Old County Road was constructed (USGS 1947; USGS 1956).
- Between **1956** and **1993** – The tributary that runs north along Carlmont Drive was culverted; Belmont Creek was culverted between Carlmont Drive to just upstream of Maywood Drive (USGS 1956; USGS 1993).
- Between **1993** and **2012** – No observable changes to channel morphology on USGS topographic maps (USGS 1993; USGS 2012).

Figure 11 through Figure 16 show historical maps and depictions of Belmont Creek. Figure 17 illustrates the Belmont Creek Watershed and Historical Channel Location Map, where blue lines represent open channels, dotted red lines represent existing culverts, solid red lines represent new, open channels, and green lines represent historical channel locations.

Figure 11. Molitor Subdivision Map, Color Added to Belmont Creek, 1856

Source: Dewing 1977

Figure 12. Ralston Hall and Its Grounds, San Mateo County

Source: Lee 1865

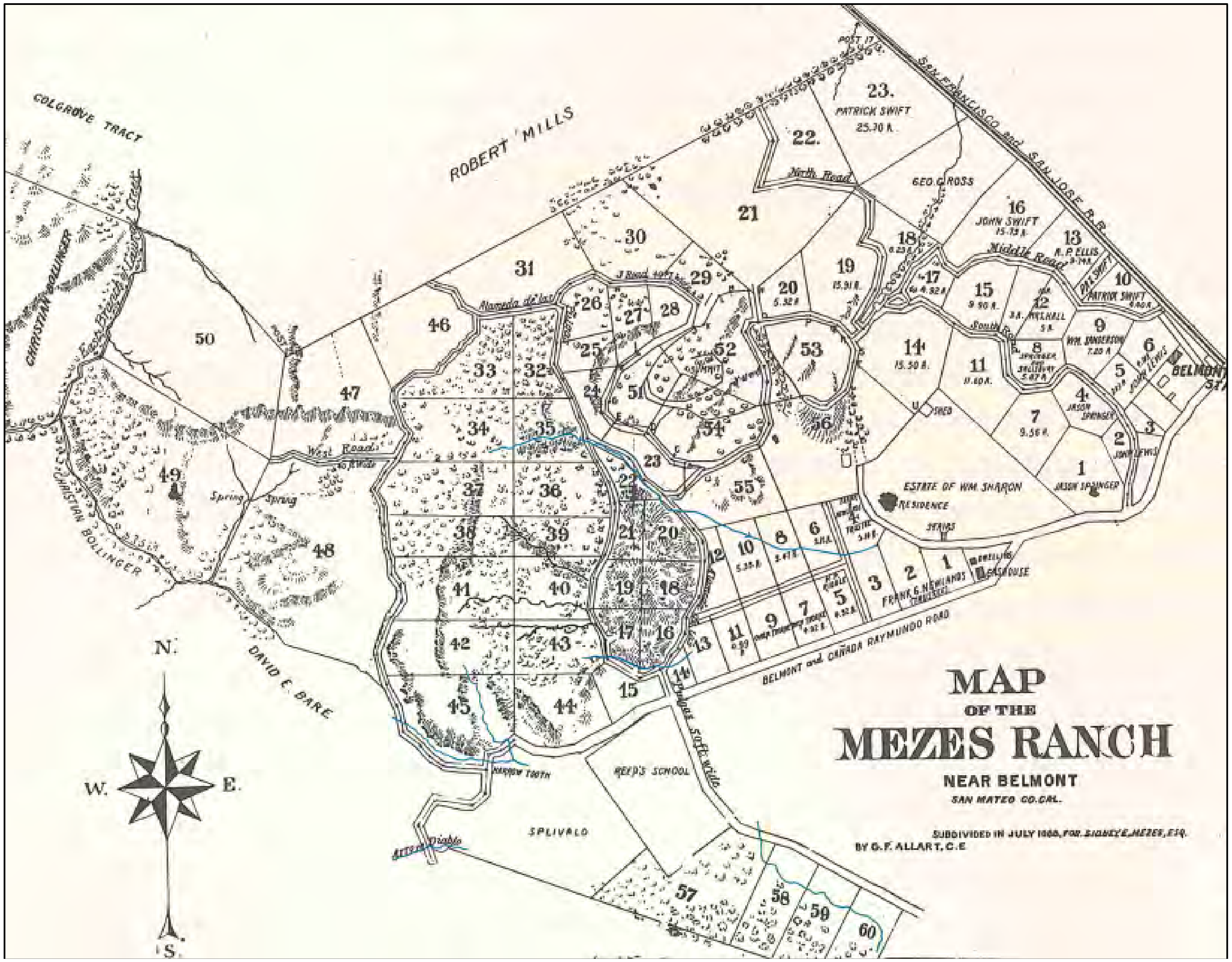

Figure 13. Mezes Ranch Map, Color Added to Belmont Creek (Arroyo Diablo) and Tributaries, 1888

Source: Dewing 1977

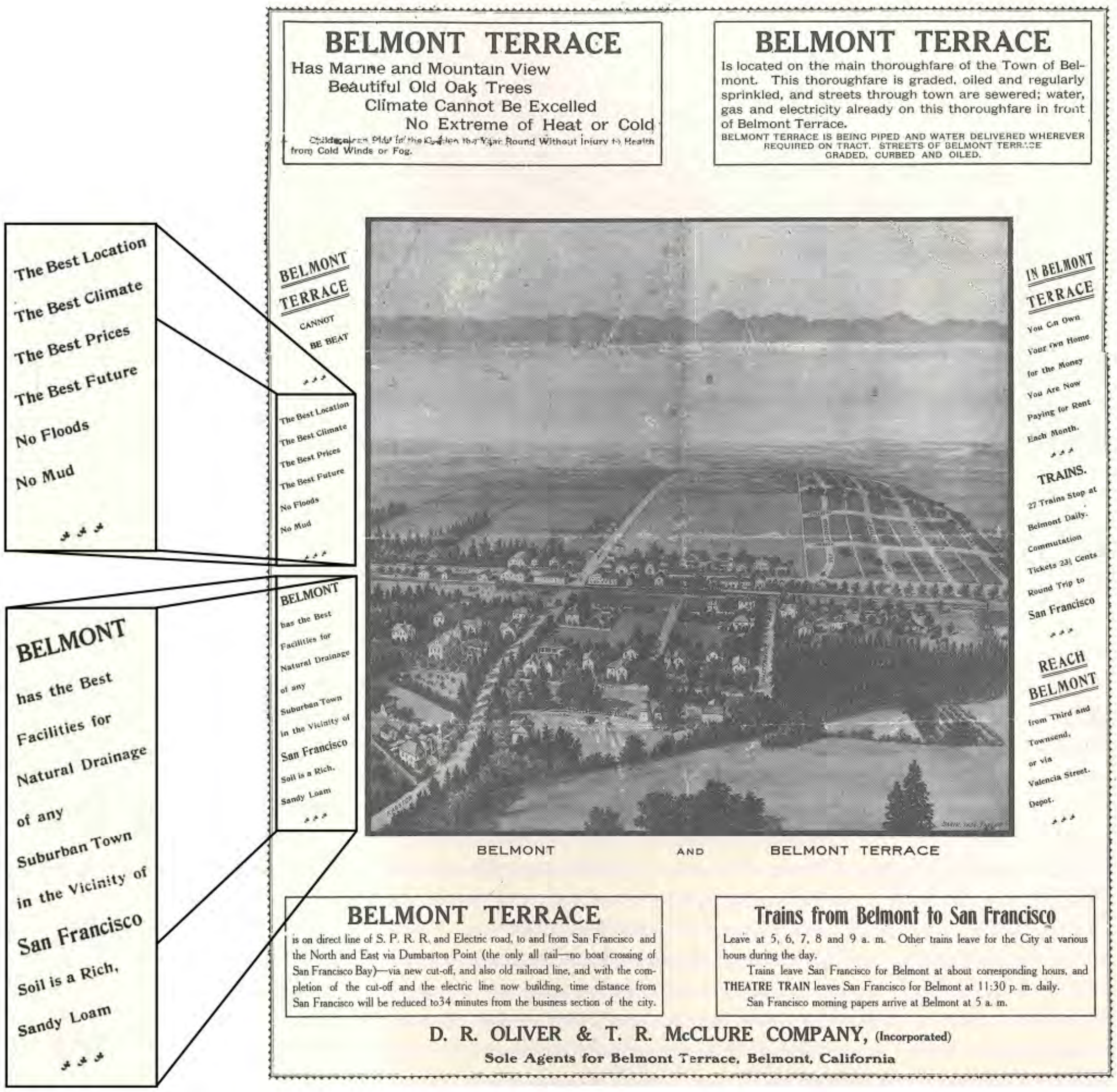

Figure 14. Advertisement for the Belmont Terrace Subdivision, 1907

Source: Dewing 1977

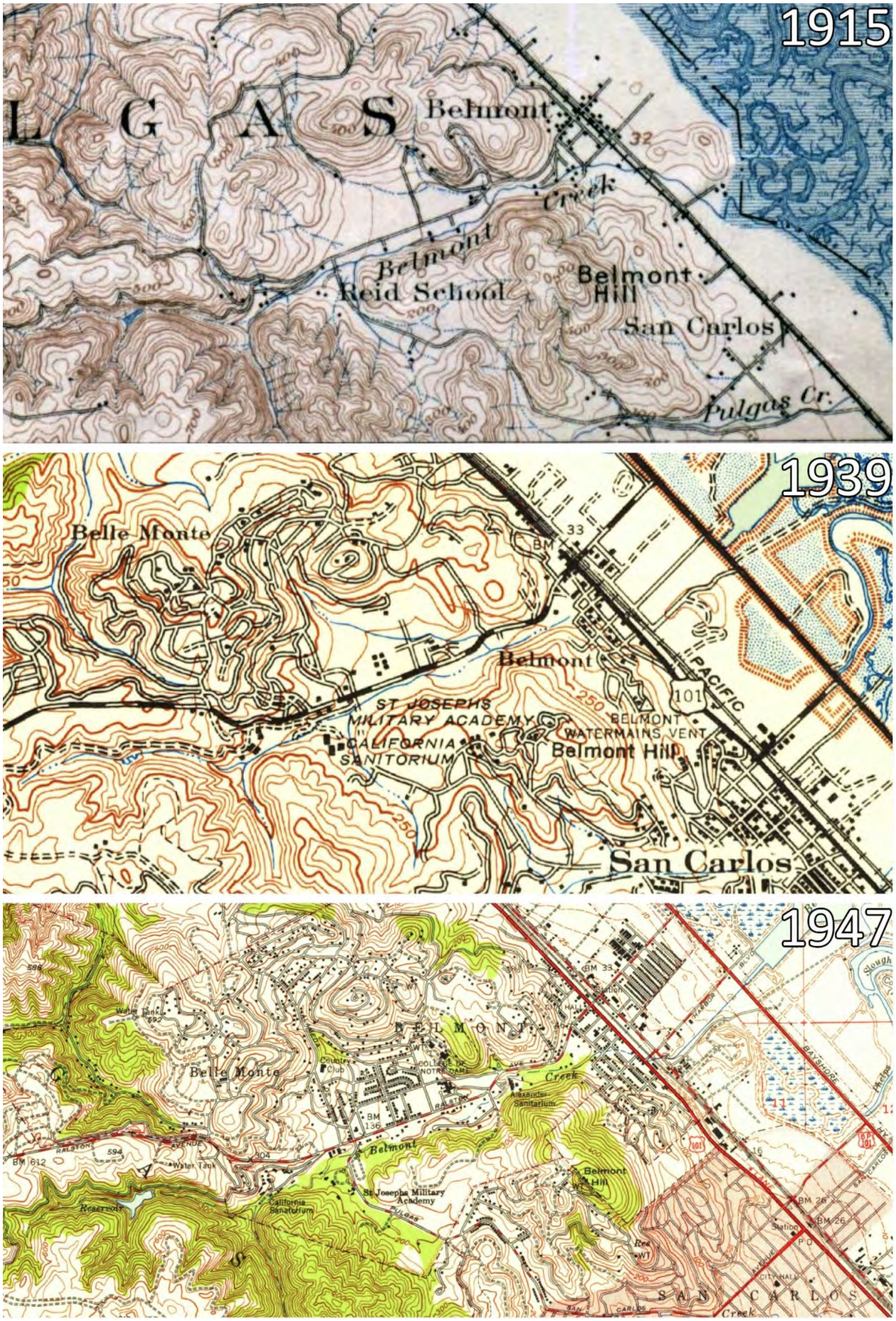

Figure 15. Comparison of Historical USGS Topographic Maps

Source: USGS 1915; USGS 1939; USGS 1947

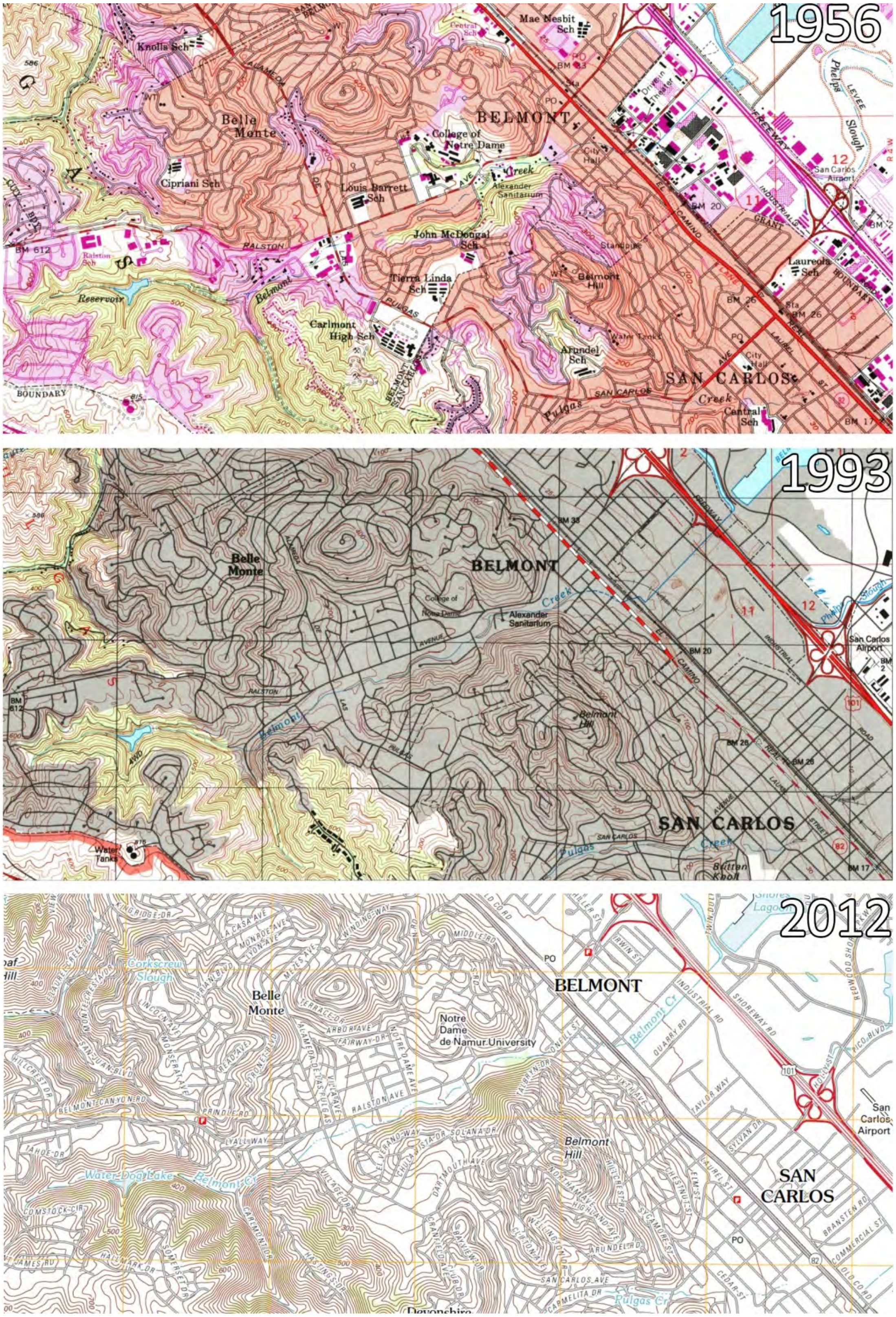

Figure 16. Comparison of Historical USGS Topographic Maps (continued)

Source: USGS 1956; USGS 1993; USGS 2012

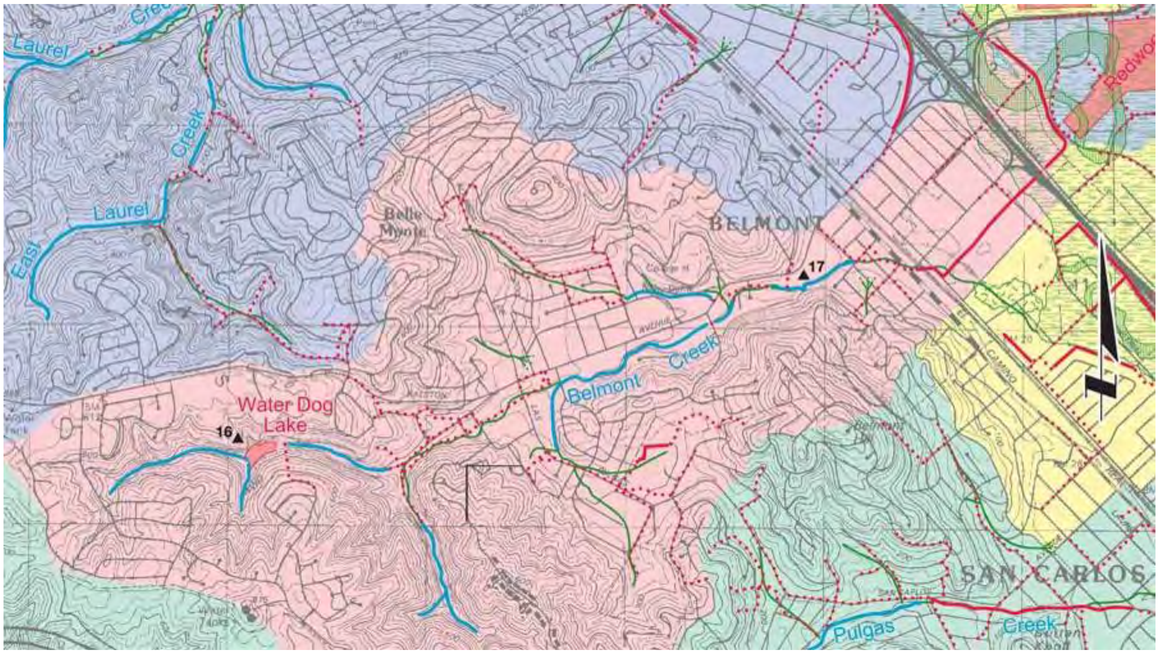

Figure 17. Belmont Creek Watershed and Historical Channel Location Map

Source: William Lettis & Associates and San Francisco Estuary Institute 2007

3.3 Channel Stability

The historical review indicates a channel in relative equilibrium prior to the period of 1915 to 1939, when Belmont Creek was channelized, straightened, and rerouted. During this time period, a series of channel modifications were coincident with the post-war boom in urban and suburban development. These coalescing changes, particularly straightening, created conditions of instability and disequilibrium by interrupting the dissipation of the moving water's kinetic energy (Rosgen 1994).

Presently, urban and suburban development along the top of bank in addition to bank hardening prevents the creek's ability to adjust to modified hydraulic conditions through lateral meandering and changes to the planimetric form throughout much of the watershed. As such, channel incision and channel widening would be the processes by which the creek expends excess energy. In some locations, exposed bedrock along the channel bottom indicates that channel widening is the dominant channel dynamic in those locations. In 1998, BKF noted that armoring and channels linings appeared to have stabilized the creek bed and banks, and no signs of erosion were observed during site visits. However, subsequent creek walks have noted the presence of erosion along significant portions of the channel in the mid-watershed (EOA Inc. 2007, WRECO 2014).

WRECO's assessment of channel stability is provided in Table 2. In general, the stability of reaches B4 and B7 exceeds that of B1 and B2. It appears that channel modifications, as discussed in the following section, have exacerbated existing erosional areas, while straightening fosters conditions of instability. Although reaches B4 and B7 are relatively more stable than B1 and B2, high flow velocities, as well as the discharge of "hungry water" from Water Dog Lake, contribute to ongoing erosion.

Table 2. Stability

Reach	Stability Assessment	Reference Photos
B1	Generally not stable. The creek is incised throughout this reach (up to 20 ft in Twin Pines Park), and incision appears to be an active channel dynamic, evidenced by undercut channel modifications. Channel straightening in the downstream portion of the reach has altered energy dissipation patterns, creating bank scour and failures upstream of the 6th Avenue culvert.	See Photo 40, Photo 41, Photo 42, and Photo 43.
B2	Generally not stable. This reach has been extensively modified to protect adjacent residences from damage through erosion and bank failures. These modifications have created overhangs during high flow events, suggesting erosion, undercutting, and incision are active channel dynamics in this reach. The channel is incised, creating over-steepened and undercut banks (up to 6 ft); evidence of past bank failures was observed.	See Photo 21, Photo 22, Photo 23, Photo 24, Photo 27, Photo 32, and Photo 33.
B4	Somewhat stable. Water Dog Lake appears to create instability through "hungry water" and high velocity discharges from the outfall. The bankfull channel is not incised throughout much of this reach, but undercutting, exposed roots, some incision, over-steepened banks, and prior bank failures were observed.	See Photo 5, Photo 6, Photo 10, Photo 11, Photo 15, and Photo 18.
B7	Generally not stable. This reach is an undersized, constructed channel with a straight alignment. This reach is incised with steep side slopes, contains four hydraulic constrictions (culverts at Caltrain tracks, Old County Road, Industrial Road, and US 101), and has two unnatural 90-degree bends. In addition, this reach experiences aggradation such that dredging is periodically required.	See Photo 45, Photo 46, Photo 48, Photo 49, Photo 50, and Photo 51.

3.4 Channel Modifications

Belmont Creek is a highly modified fluvial system, with approximately 50% of reaches B1, B2, B3, and B4 having been modified in some way (EOA Inc. 2007). The mid- to lower watershed (reaches B1, B2, B3, B6, B7) has been substantially modified with culverting, channelization, straightening, and rerouting during the last 100 years. Reaches B6 and B7 are constructed channels. By contrast, the upper watershed (reaches B4, B5, B8) consists almost entirely of natural channels, though the tributaries along Carlmont Drive and Alameda de las Pulgas are mostly culverted. The most common channel modifications are culverting and bank armoring along residential, commercial, and industrial areas. Uncoordinated creekside development has resulted in a piecemeal system of bank armoring consisting of sacked concrete, concrete rip rap, shotcrete, rock gabions, bricks, chain link fencing, and tarp.

Table 3 lists the channel modifications observed in each reach assessed during the creek walk, and corresponding photos are provided for reference.

Table 3. Channel Modifications

Reach	Channel Modifications / Extent of Modifications[1]	Reference Photos
B1	Sacked concrete, concrete apron, straightening, rock gabion basket, bank hardening, and culverting / 1,562 ft or 54% of reach.	See Photo 6 and Photo 7.
B2	Sacked concrete, private outfalls without velocity dissipation devices, rock armoring, concrete apron, poured concrete, tarp erosion control, shotcrete, hydraulic constriction at private driveway bridge, concrete wall, cinderblock wall, rip rap, chain link fencing, fabric, brick armoring, concrete grade control structure, and culverting / 4,267 ft or 65% of reach.	See Photo 22, Photo 24, Photo 26, Photo 27, Photo 28, Photo 29, Photo 31, Photo 32, Photo 33, Photo 35, and Photo 37.
B4	Impoundment at Water Dog Lake and associated structures/outfalls, and culverting / 13 ft or 1% of reach.	See Photo 6 and Photo 7
B7	Constructed channel with unnatural 90-degree-angle bends, rock gabion basket, outfalls without velocity dissipation devices, culverting, poured concrete, rip rap, sacked concrete, shotcrete, tiered concrete banks, concrete drop structure, and flood walls	See Photo 45, Photo 46, Photo 47, Photo 48, and Photo 51.

3.4.1 Hydraulic Constrictions

In the Belmont Creek watershed, a majority of creek crossings are culverts beneath roadways, although a footbridge in reach B1 and a private residential driveway bridge in reach B2 exist. Of these creek crossings, four hydraulic constrictions have been identified: the Water Dog Lake outfall (Photo 1), the Old County Road culvert (Photo 2), the Industrial Road culvert (Photo 3), and the US 101 culvert (Photo 4). In the 1998 Harbor Industrial District Storm Drain Report, BKF recommended installing additional culverts to reduce constriction and increase conveyance capacity at these locations, except at Water Dog Lake.

[1] Source: EOA Inc. 2007

Photo 1. Water Dog Lake Outfall

Photo 2. Old County Road Culvert

Photo 3. Industrial Road Culvert

Photo 4. Old County Road Culvert

The culvert at Industrial Road is causing significant aggradation within the culvert and the area immediately upstream. This process is likely exacerbated by backwater conditions during floods or high tides, during which this culvert can be completely submerged by tidal waters.

3.5 Sediment Transportation

In general, reaches in the upper and mid watershed downstream of Water Dog Lake (B1, B2, B3, B4) are eroding, while reaches in the flatlands of the lower watershed are aggrading (B6, B7). Upstream of Carlmont Drive and Water Dog Lake, the surrounding land use is open space consisting of vegetated hillsides, and these reaches (B5, B8) have less erosion. This pattern of sediment transportation indicates the discharge of sediment-starved, or “hungry water,” from Water Dog Lake.

Dams interrupt the longitudinal continuity of sediment transportation to downstream reaches. Upstream of Water Dog Lake, suspended sediments and bedloads are deposited within the reservoir or reaches affected by backwater. Downstream of the Water Dog Lake outfall, which does not have a velocity dissipation device, discharges expend excess energy on channel bed and bank erosion, resulting in incision, bank failures, and a coarsening of channel substrate; this effect is known as “hungry water.” (Kondolf 1997)

In the upper watershed, the natural channel between Water Dog Lake and Carlmont Drive (B4) is incised to bedrock along the channel bottom in places, and shows signs of bank erosion and channel widening; however, depositional environments were also observed, suggesting dynamic equilibrium in this reach. The natural channel between Maywood Drive and Silverado Senior Center (B1, B2) shows signs of channel incision, to bedrock in some locations, and bank undermining leading to failure. The channel of Belmont Creek in these reaches has been extensively hardened and the banks have been covered with various treatments to prevent further erosion. In the lower watershed, approximately 3 ft of gravel has accumulated just upstream of the Industrial Road culvert (B7) after dredging occurred adjacent to the Novartis facility in September 2013. Moreover, a dredging project is planned for Belmont Slough near Redwood Shores (B6) to restore channel capacity due to aggradation.

Table 4 provides WRECO's assessment of sediment transportation by reach. In general, the upper watershed is in a state of dynamic equilibrium, whereas the mid-watershed is eroding and the lower watershed is aggrading.

Table 4. Sediment Transportation

Reach	Sediment Transportation / Extent of Erosion[2]	Reference Photos
B1	This reach is actively eroding and contributing sediment to downstream reaches (B6, B7). The channel is incised throughout this reach (up to 20 ft) and contains over-steepened banks. Downstream of a straightened portion of this reach, indications of bank scour and recent failure were observed / 228 ft or 12% of reach.	See Photo 40, Photo 41, Photo 42, and Photo 43.
B2	This reach is actively eroding and contributing sediment to downstream reaches (B1, B6, B7). As described in Table 2 and Table 3, this reach has been extensively modified to protect adjacent residences from damage. High-flow events have created overhands above the modified area, and repairs of undercut bank modifications have subsequently been undercut. These observations suggest that undercutting, erosion, and incision are active channel dynamics. In addition, many private outfalls are present within this reach without velocity dissipation devices / 762 ft or 25% of reach.	See Photo 20, Photo 21, Photo 23, Photo 27, Photo 28, Photo 29, Photo 32, and Photo 33.
B4	In general, this reach is in a state of dynamic equilibrium. Significant erosional features are associated with the outfall of Water Dog Lake; however, depositional areas were observed, such as backwater areas caused by large woody debris, floodplains, and point bars, just a few hundred feet downstream of the outfall / 567 ft or 20% of reach.	See Photo 5, Photo 6, Photo 9, Photo 10, Photo 11, Photo 13, and Photo 16.
B7	Although erosional areas were observed, this reach is actively aggrading. Since dredging occurred adjacent to the Novartis facility in September 2013, approximately 3 ft of gravel has accumulated upstream of the Industrial Road culvert. With a total of four hydraulic constrictions within this reach, which is subject to tidal influence, backwater effects are likely the cause of sedimentation.	See Photo 45, Photo 46, Photo 47, Photo 50, and Photo 52.

[2] Source: EOA Inc. 2007

3.6 Floodplain and Riparian Buffer Conditions

The upper watershed contains more accessible floodplain areas than the lower watershed. In general, urban and suburban development has encroached up to the top of bank on either side of Belmont Creek in reaches B1, B2, B3, B6, and B7. This pattern of development has greatly reduced the available floodplain area and riparian buffer throughout the watershed.

In the upper watershed (B4, B5, B8), the bankfull channel is somewhat incised, but still provides access to a floodplain during high flows. Surrounded by steep hillsides, local topography limits the spatial extent of the floodplain area in these reaches. These reaches are bordered by oak woodland up to the adjacent ridgelines, where residential development has occurred. The woodland in the upper watershed contributes large woody debris to the creek, and therefore actively influences hydrology and channel morphology.

In the mid to lower watershed, by contrast, channel incision and development has minimized floodwaters' access to floodplains (EOA Inc. 2007). In reaches B1, B2, and B3, urban and suburban developments encroached upon the active floodplain and riparian buffer, and effectively eliminated both in these reaches. Although straightened and channelized, reaches B6 and B7 consist of an active low-flow channel with a small floodplain area between constructed earthen embankments; the riparian vegetation in reaches B6 and B7 consists mainly of non-native annual grasses and shrubs.

Historically, Belmont Creek meandered through an open meadow between approximately Alameda de las Pulgas and Chula Vista Drive in the mid watershed (B2) that contained what appears to be a substantial floodplain area (Figure 12). Situated between reaches wherein steep hillsides limit accessible floodplain areas (B1, B4, B5, B8), a floodplain in these reaches would have reduced peak flows downstream (B6, B7). It is possible that this floodplain-attenuated peak flows such that Belmont Creek did not overtop its banks, as suggested by anecdotal evidence that the creek wasn't known to flood (Figure 14). In reach B1, Belmont Creek was bordered by a woodland with abundant oak, California buckeye, and California bay (Dewing 1977). In addition, the lower watershed (reach B7 before channelization and rerouting to Redwood Shores Lagoon) from Waltermire Street and El Camino Real also contained a riparian woodland until at least 1907. This woodland is depicted as a coniferous forest in Figure 14, potentially containing cypress, pine, and redwood (Dewing 1977).

Table 5 provides an assessment of floodplain and riparian buffer conditions by reach.

Table 5. Floodplain and Riparian Buffer Conditions

Reach	Floodplain and Riparian Buffer Conditions	Reference Photos
B1	Floodplain and riparian buffer conditions in this reach are poor, but exceed those in reach B2. The downstream portion of this reach is located within Twin Pines Park, and therefore has a wider riparian buffer and the ability to reestablish a floodplain within the incised channel through bank erosion processes.	See Photo 40.
B2	Floodplain and riparian buffer conditions in this reach are poor. Residential developments have encroached upon the tops of both banks, limiting the available space for these features. Moreover, bank hardening and residential developments limit the ability of the creek to reestablish a floodplain within the incised channel.	See Photo 20, Photo 22, Photo 28, Photo 36, and Photo 38.
B4	This reach flows through a well-vegetated, narrow canyon with steep, wooded hillsides. Near the ridgeline on southern-facing slopes, the riparian oak woodland transitions into scrubland. Although located within a narrow canyon, this reach maintains access to floodplain areas. The presence of a healthy riparian buffer that contributes large woody debris and accessible floodplains allows this reach to remain in dynamic equilibrium.	See Photo 9, Photo 10, Photo 13, and Photo 17.
B7	Floodplain and riparian buffer conditions in this reach are poor. Commercial and industrial facilities have been constructed up to the tops of both banks, limiting the available space for these features. Within the earthen embankments, there is an active low-flow channel within the bankfull channel. Moreover, this constructed channel is undersized, which further limits the available floodplain space.	See Photo 45, Photo 48, Photo 49, and Photo 50.

3.7 Channel Incision

Channel incision was noted in each reach of Belmont Creek. As discussed in Section 3.3, the pattern of urban and suburban development limits the ability of the creek to adjust to modified hydraulic conditions. As such, the dominant channel dynamics available throughout much of the watershed are incision and widening. However, these channel dynamics present financial and safety risks to creek side property owners, who have reinforced and armored the channel banks to prevent failures and undermining. This has further reduced the ability of the creek to adjust to hydraulic conditions, leaving incision as the dominant mechanism by which the creek expends excess fluvial energy.

Channel incision eventually results in bank failures and mass wasting through the creation of steep, near vertical banks subject to undermining and collapse. Channel incision is most prevalent in the mid watershed (B1, B2), where residences and developments line the top of banks and piecemeal bank armoring has been constructed.

In these reaches, banks are undermined up to 6 ft in height above the low-flow channel and high flows have created overhangs where bank armoring is absent. Some locations showed evidence of past bank failures (Appendix A, Photo 32 and Photo 43). In Twin Pines Park, channel incision is extreme, consisting of steep and near vertical banks up to approximately 20 ft in height (Appendix A, Photo 40).

3.8 Sediment and Water Quality

On July 22, 2013, WRECO staff collected four samples of channel substrate to characterize the material to be dredged from the creek adjacent to the Novartis facility (B7). Samples were collected between 0 and 2 ft below ground surface (bgs) and each sample was subdivided once to produce two sets of samples. One set of samples was placed in canvas sample bags and delivered to Cal Engineering & Geology for sieve analysis with hydrometer; the other sample set was placed in preserved glass containers and delivered to Curtis and Tompkins Environmental Testing Laboratory for chemical analysis. See Figure 18 for the sample location map, Figure 19 for the results of the grain size analysis, and Table 6 for the results of chemical testing.

Figure 18. Sample Location Map, July 22, 2013

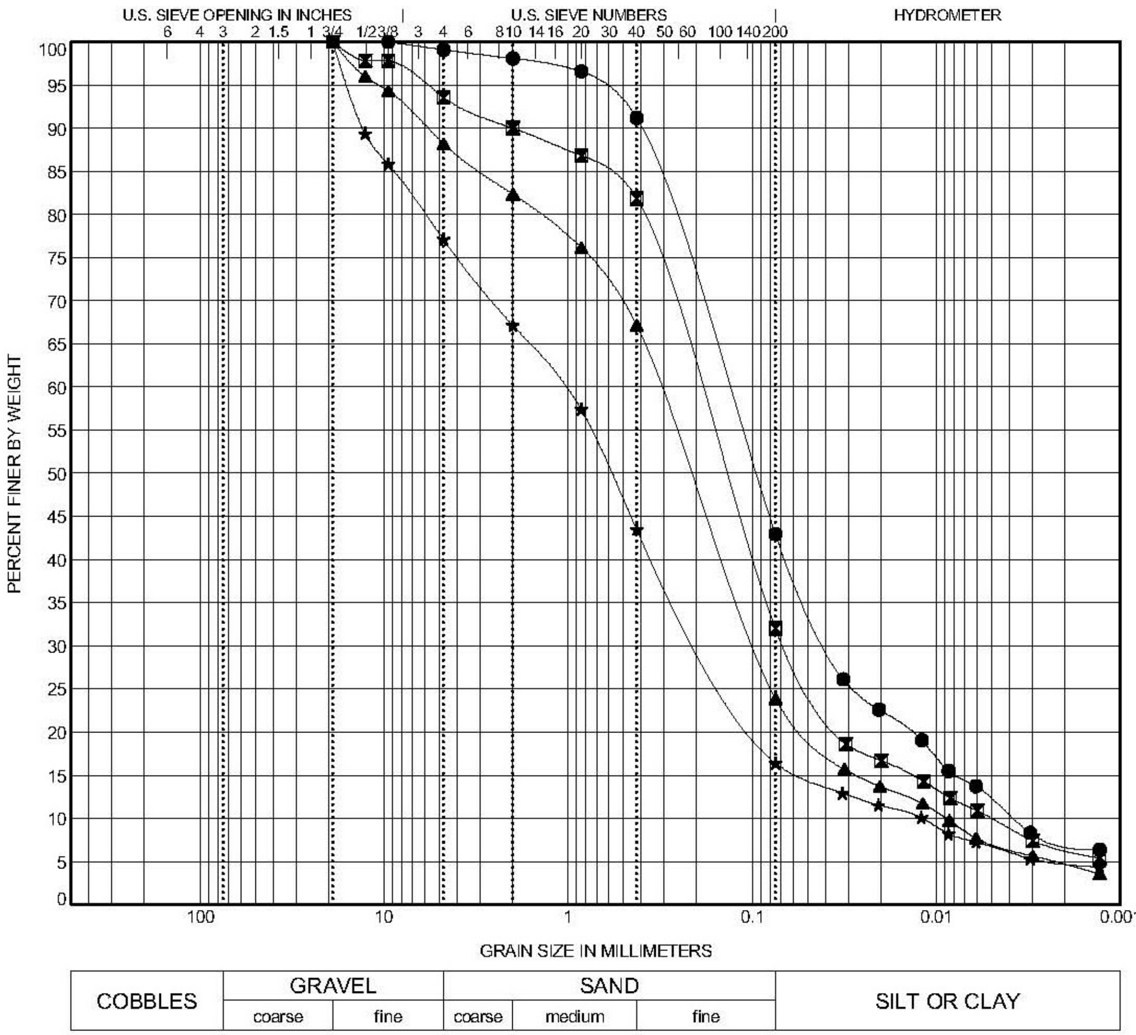

Figure 19. Grain Size Analysis of Channel Substrate in Reach B7

Table 6. Chemical Analysis of Channel Substrate in Reach B7

Analytes	Sample Result (mg/kg)			
	S1	S2	S3	S4
Gasoline C7-C12	ND	ND	ND	ND
Diesel C10-C24	55 Y	81 Y	19 Y	27 Y
Motor oil C24-C36	240	520	170	140
Volatiles				
Acetone	0.046	ND	ND	ND
2-Butanone	0.014	ND	ND	ND
para-Isopropyl Toluene	ND	0.011	ND	ND
California Title 22 Metals				
Antimony	ND	ND	ND	ND
Arsenic	2.2	3.2	1.5	1.7
Barium	120	130	93	90
Beryllium	0.3	0.37	0.39	0.32
Cadmium	0.36	0.49	0.33	0.33
Chromium	34	41	36	35
Cobalt	10	12	12	11
Copper	32	30	26	23
Lead	18	27	13	9.6
Mercury	0.049	0.073	0.072	0.026
Molybdenum	0.33	0.88	ND	0.4
Nickel	35	42	35	40
Selenium	ND	ND	ND	ND
Silver	ND	0.66	2.8	ND
Thallium	ND	ND	ND	ND
Vanadium	37	45	38	36
Zinc	89	120	86	74

On March 26, 2014, WRECO collected water quality data from Belmont Creek during a storm event. Samples and data were collected from each sample location near peak stormwater flow conditions to assess sediment loads and sources in the watershed. See Table 7 for the results of water quality monitoring, and Figure 20 for a depiction of sample collection in relation to rainfall rates.

Table 7. Total Suspended Solids and General Water Quality Parameters

Parameter	Novartis	6th/O'Neill Avenues	DS of Chula Vista Drive	Water Dog Lake Outfall
Time	10:30 AM	10:43 AM	8:24 AM	9:40 AM
Temperature (degrees C)	12.9	13.3	12.3	13.2
pH	8.47	8.05	7.89	7.94
Conductivity (uS)	121.6	110.3	94.0	280.9
Specific Conductance (uS)	158.0	141.8	124.1	362.1
Salinity (ppt)	0.1	0.1	0.1	0.2
Total Suspended Solids (mg/L)	210	94	370	85

uS = microSiemens
ppt = parts per thousand
mg/L = milligrams per liter

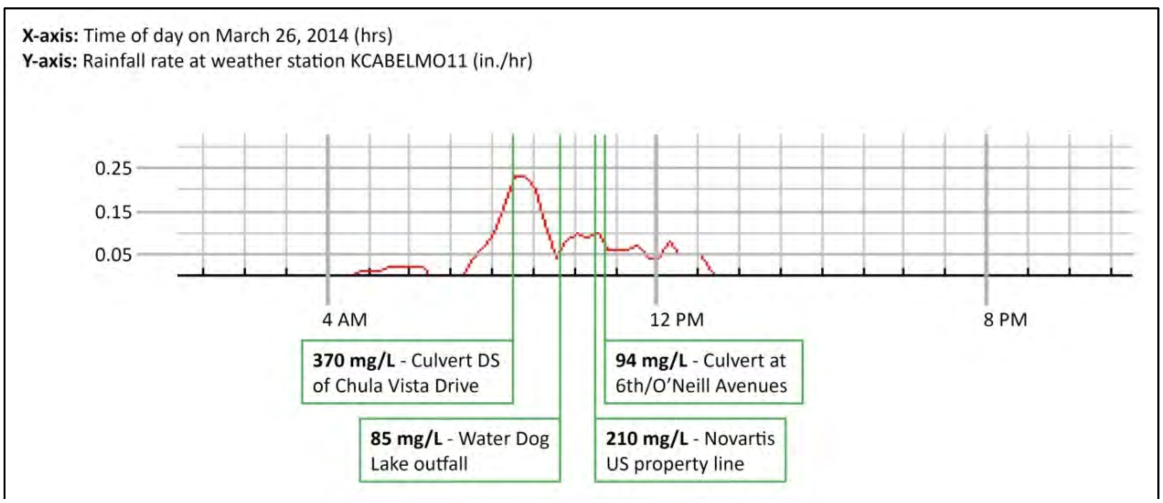

Figure 20. Total Suspended Solids Sample Results in Relation to Rainfall Rates, March 26, 2014

The sample collected from the Water Dog Lake outfall (B4) contained the smallest suspended sediment load (85 mg/L); this result is consistent with the discharge of "hungry water" from Water Dog Lake.

The sample collected in reach B2 just downstream of Chula Vista Drive (near the Silverado Senior Center) contained the highest suspended sediment load (370 mg/L). As previously described, this reach is actively eroding and supplying sediment to downstream reaches.

Regardless of the evidence of heavy erosion, creek incision, and bank failures throughout Twin Pines Park (B1), the sediment load detected at 6th and O'Neill avenues was lower than anticipated. This could be attributed to sample design (grab sampling) and timing (after the "first flush," see Figure 20).

The high sediment load detected at Novartis suggests a sediment source between the Novartis facility and El Camino Real. Potential sediment sources include the transport of deposited sediment out of the culvert that begins at 6th/O'Neill Avenue, or erosion created by the 90-degree-angle bends downstream of Old County Road.

In addition, water quality was monitored at two locations along Shoreway Road to obtain baseline salinity data for lower Belmont Creek (B6). Shoreway Road crosses over the tidally influenced portion of Belmont Creek twice: once near Marine Parkway and once near US 101, representing downstream and upstream locations, respectively.

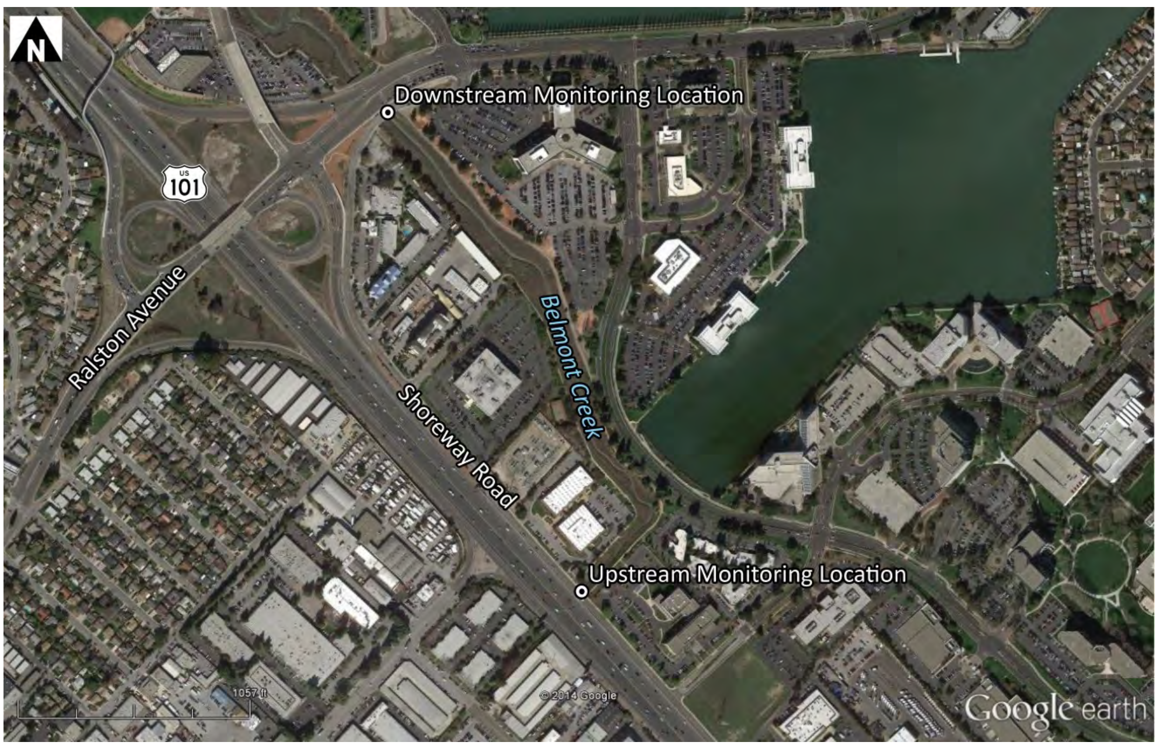

Figure 21. Salinity Monitoring Locations

Conductivity, specific conductance, pH, temperature, and salinity were measured prior to, during, and after a flood tide of 7.9 ft at 9:20 AM, as recorded at the Redwood City tidal gaging station (NOAA Tides and Currents 2014), at both monitoring locations. Project alternatives could potentially alter salinities in lower Belmont Creek, which could subsequently alter the spatial distribution of existing freshwater and estuarine habitats over time. See Table 8 for the salinity data obtained on March 26, 2014, and Figure 22 for a depiction of salinity in relation to rainfall rates and tidal stage.

Table 8. Salinity and General Water Quality Parameters in Belmont Creek

Time	Location	Strata	Temp. (deg. C)	pH	Conductivity (mS)	Specific Conductance (mS)	Salinity (ppt)
6:24 AM	DS	Upper	15.9	7.29	30.90	37.43	23.8
6:36 AM	US	N/A	15.2	7.32	29.35	36.19	22.9
7:18 AM	DS	Upper	15.9	7.20	31.50	38.18	24.3
7:20 AM	DS	Lower	15.9	8.00	31.77	38.48	24.5
7:29 AM	US	N/A	13.1	7.49	11.42	14.77	8.6
8:57 AM	DS	Upper	15.7	7.47	30.64	37.44	23.8
8:58 AM	DS	Lower	15.8	8.25	31.26	37.99	24.2
9:06 AM	US	Upper	12.8	7.39	7.63	8.81	4.1
9:07 AM	US	Lower	12.2	7.91	178.4 uS	.236	0.1
10:57 AM	DS	Lower	12.5	7.13	1328 uS	1.745	0.9
10:58 AM	DS	Upper	12.4	7.69	601 uS	.792	0.4
11:05 AM	US	N/A	12.7	7.69	154.8 uS	.202	0.1

Notes:
US = upstream location near US 101 culvert
DS = downstream location near Marine Parkway
N/A = no stratification observed at time of sampling
mS = milliSiemens
uS = microSiemens

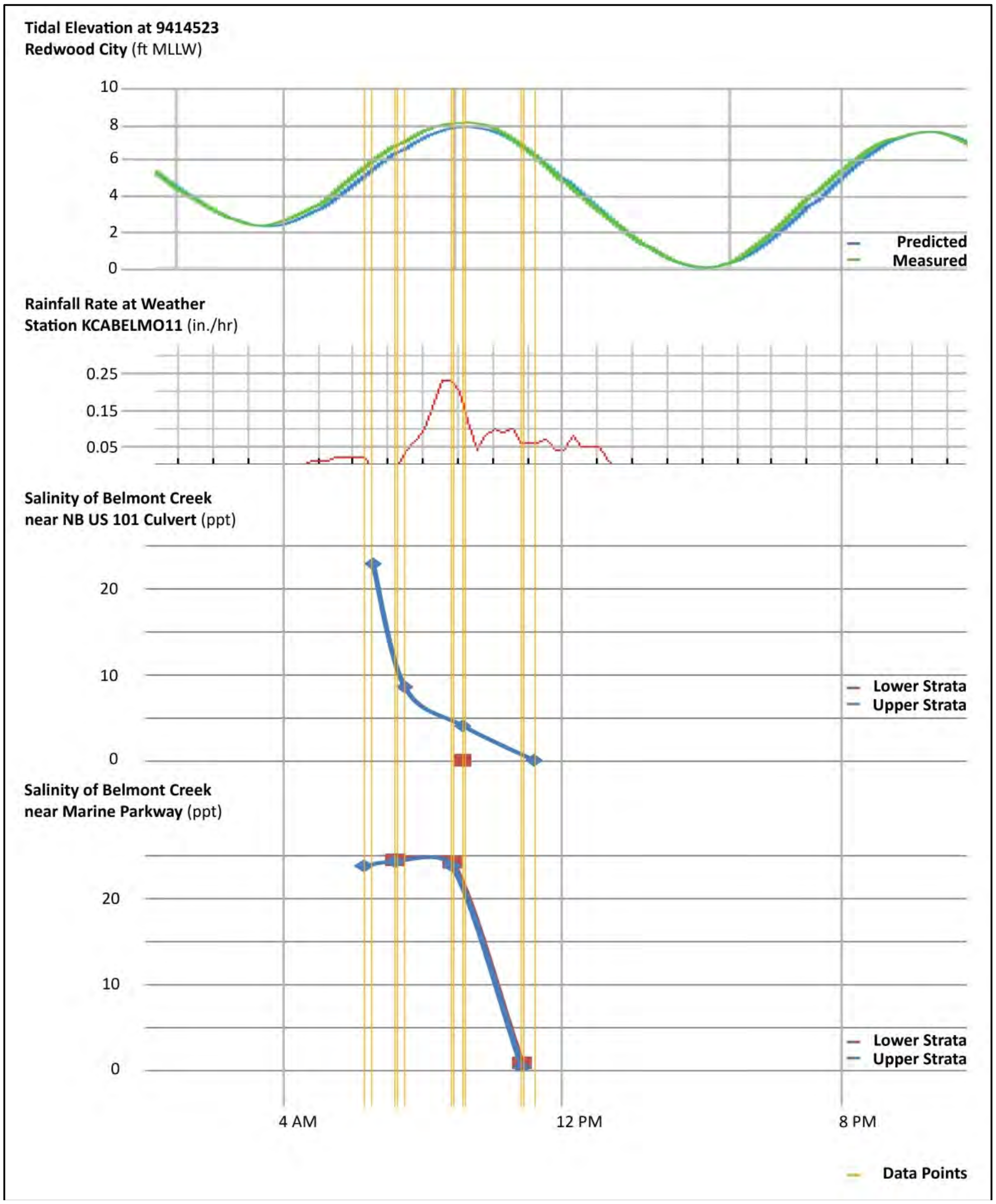

Figure 22. Salinity Data in Relation to Rainfall Rate and Tidal Stage, March 26, 2014

Tidal influence was limited to reach B6 during the storm event by the hydraulic pressure of stormwater flows. Tidal influence has been documented just upstream of Industrial Road adjacent to the Novartis facility during fair weather; however, tidal influence was limited to just downstream of the US 101 culvert during the storm event. If reach B6 of the creek is predominantly freshwater during a relatively mild storm event, preliminary indications suggest using this area as a detention basin during larger storm events should not significantly affect the longitudinal salinity gradient, and the distribution of freshwater and estuarine habitats in reach B6 should not be affected.

4 IMPROVEMENT ALTERNATIVES

As part of the watershed study, WRECO developed nine conceptual alternatives to reduce flooding potential and improve water quality and riparian habitat along Belmont Creek. These improvements can also enhance water quality by lowering concentrations of pollutants entering the storm drains and Belmont Creek. The following sections describe these alternatives.

4.1 Alternatives Matrix

During the stakeholders meeting with Novartis, City of Belmont, City of San Carlos, City of Redwood City, San Mateo County, Caltrain, Caltrans, and San Francisco Bay RWQCB, WRECO informed the stakeholders of the nine alternatives that were developed for the Project. Through a voting process, five alternatives were selected by the stakeholders for assessment through hydrologic and hydraulic modeling. To achieve a robust alternative, at least one alternative located in the upper, middle, and lower watershed would be assessed through modeling. Table 9 presents the following information regarding the proposed alternatives:

- Preliminary cost estimates
- Assessments of implementation complexity, flood protection, environmental impacts, acceptance by regulators on a scale of 1 to 5; and
- Stakeholder votes.

Alternatives 1, 5, 6, 7, and 8 were chosen to be modeled.

Table 9. Voting Distribution for Improvement Alternatives

Alternatives	Preliminary Cost Estimate	Implementation Complexity	Flood Advantage	Environmental	Regulatory Acceptance	Votes
Alternative 1: Operations and Design Water Dog Lake/Dam	$5,200,000	Complex	3	2	5	7 [1,2,4,5,6,7,8]
Alternative 2: Upstream Basins at Carlmont Drive, Village Drive, and Carlmont High School Baseball Field Off Club Drive	$2,000,000	Complex	4	1	3	1 [3]
Alternative 3: Low Impact Development Measures	Varies	Moderate	1	5	5	0
Alternative 4: Creek Daylighting Through Silverado Senior Living Facility With Bypass	$2,000,000	Complex	1	5	5	1 [5]
Alternative 5: Floodplain Restoration at Twin Pines Park with Offline Basin for Temporary Storage	$2,000,000	Complex	3	4	5	7 [1,2,3,4,5,6,7]
Alternative 6: Parallel Overflow Pipes from Old County Road down to Harbor Blvd with Culvert Improvements to Industrial Road and US 101	$5,600,000	Moderate to Complex	4	2	2	4 [4,5,6,8]
Alternative 7: New Cross Culvert at Old County Road & Channel Improvement with Short Flood Walls on Lower Belmont Creek and Culvert Improvements to Industrial Road and US 101	$5,500,000	Moderate	4	2	3	5 [1,2,3,7,8]
Alternative 8: Floodgate and Pump at Shoreway Road/Marine Parkway	$2,000,000	Moderate	4	1	1	6 [1,4,5,6,7,8]
Alternative 9: Floodgates and Pump near Oracle Bridge	$8,000,000	Moderate	5	1	1	2 [2,3]

Scale:	Voters:
1 - Low	(1) RWQCB
2 - Low to Moderate	(2) Redwood City
3 - Moderate	(3) Caltrain
4 - Moderate to High	(4) City of Belmont
5 - High	(5) San Mateo County
	(6) City of San Carlos
	(7) Novartis
	(8) Caltrans

4.2 Alternative 1: Operations and Design at Water Dog Lake

This alternative involves studying the operation of Water Dog Lake, specifically the capacity and timing of releases. The City of Belmont has noted that the dam is seismically unsafe, so the volume of water stored throughout the year has been reduced for safety reasons. The goal of this alternative would be to retrofit the dam to bring it up to code, increase the storage capacity, and provide additional storage that could be metered out after large storm events. Additionally, this alternative would assess mechanisms to increase the continuity of sediment transportation to downstream reaches, thereby reducing the erosional potential associated with "hungry water." This alternative was selected for further assessment.

4.3 Alternative 2: Upstream Basins at Carlmont Drive, Village Drive, and Carlmont High School

This alternative would create temporary off-line storage basins that would slowly meter out flows to reduce the hydraulic peaks during storm events. Three preliminary sites were identified as possible locations to construct detention basins or underground storage areas; these locations are presented in Figure 23. These areas were identified based on the available open space as shown in aerial imagery.

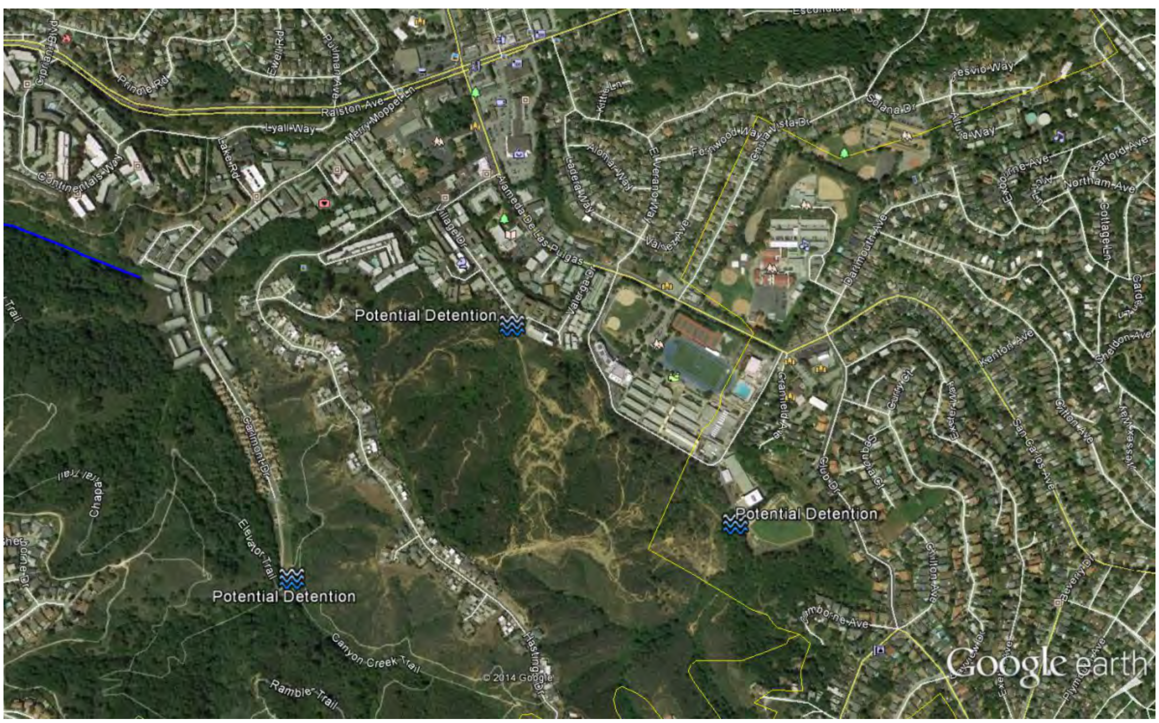

Figure 23. Potential Detention Locations for Alternative 2

Due to the lack of open space within the watershed, the identified areas are located within the upper reaches of the watershed. Real property acquisition may be required to install the basins and achieve water quality and flood relief benefits. This alternative was not selected for further assessment.

4.4 Alternative 3: Low Impact Development Measures

For this alternative, opportunities presented by future projects within the adjacent jurisdictions would be sought to construct Low Impact Development (LID) measures. The goal would be to achieve a cumulative effect from LID measures installed throughout the diffuse watershed to improve water quality and reduce peak flows during storm events. The cities currently have LID requirements for new development, but there are possibilities to provide incentives to private citizens and businesses to install LID measures. These measures may include rainwater harvesting, porous pavement, infiltration planters, etc. This alternative was not selected for further assessment.

4.5 Alternative 4: Creek Daylighting through Silverado Senior Living Facility

This alternative proposes to daylight Belmont Creek from just west of the Silverado Senior Living Facility (Figure 24) to just upstream of Twin Pines Park. The tributary along the College of Notre Dame meets Belmont Creek in this portion of the watershed, and the combined flows are culverted through the Silverado Senior Living Facility. Daylighting would provide greater hydraulic capacity as well as improved channel stability, water quality, and sediment transportation through an active low flow channel, appropriately sized bankfull channel, and a reestablished floodplain, as well as a new riparian buffer planted with native species. In addition, this alternative would present a mitigation opportunity for future projects. This alternative was not selected for further assessment.

Figure 24. Alternative 4 Layout

4.6 Alternative 5: Floodplain Restoration at Twin Pines Park with Offline Basin for Temporary Storage

This alternative would install an off-line, temporary storage basin in Twin Pines Park, as well as restore the reach of Belmont Creek through Twin Pines Park, through the construction of an active low flow channel, appropriately sized bankfull channel, and a reestablished floodplain to provide additional storage capacity. The created floodplains would reduce velocities of larger flows and disperse the flow, relieving downstream reaches of higher peak flows. The floodplains would be vegetated with appropriate native species, based on a reference site within the watershed, to improve riparian habitat. Figure 25 provides a conceptual layout of this alternative. This alternative was selected for further assessment.

Figure 25. Alternative 5 Layout

4.7 Alternative 6: Parallel Overflow Pipes from Old County Road to Harbor Boulevard

This alternative proposes to construct a culvert along Harbor Boulevard, from the point where the creek daylights at Old County Road, to receive high flows during storm events and provide additional hydraulic capacity for the creek (Figure 26). Additional culverts would be installed at Industrial Road and US 101 to provide additional flow capacity for Belmont Creek and to relieve the effects of hydraulic constriction at those points. The proposed bypass culvert would require roadway excavation to construct. Therefore, this alternative also provides opportunities to install LID measures atop the culvert to provide localized flood relief and water quality benefits. These measures could include permeable pavers and bioretention bulb-outs. This alternative was selected for further assessment.

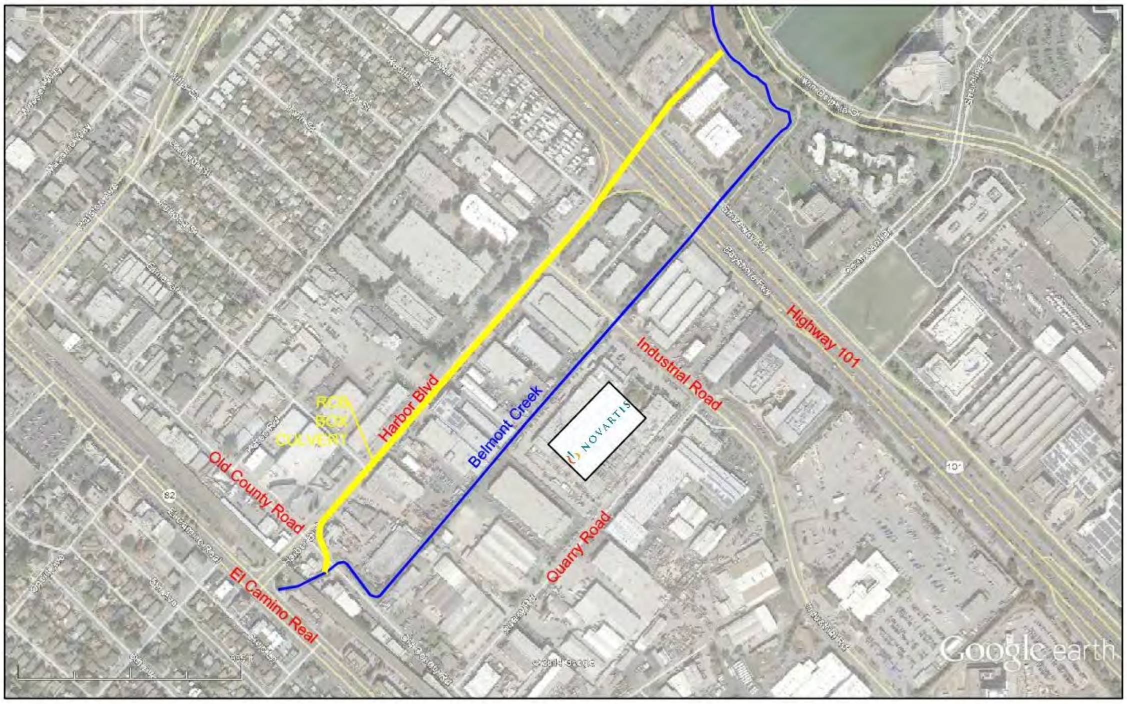

Figure 26. Alternative 6 Layout

4.8 Alternative 7: New Cross Culvert at Old County Road and with Short Flood Walls on Lower Belmont Creek

This alternative would eliminate the two 90-degree-angle bends in lower Belmont Creek through the construction of a double, 12-ft by 8-ft, reinforced concrete box culvert (RCB) to reduce junction losses within the creek (Figure 27). Four-foot flood walls would also be installed between Old County Road and Industrial Road to provide additional hydraulic capacity within this reach, which would minimize inundation of the area. As with Alternative 6, this alternative also proposes additional culverts at Industrial Road and US 101 to reduce the hydraulic constriction at those crossings and provide additional hydraulic capacity. This alternative was selected for further assessment.

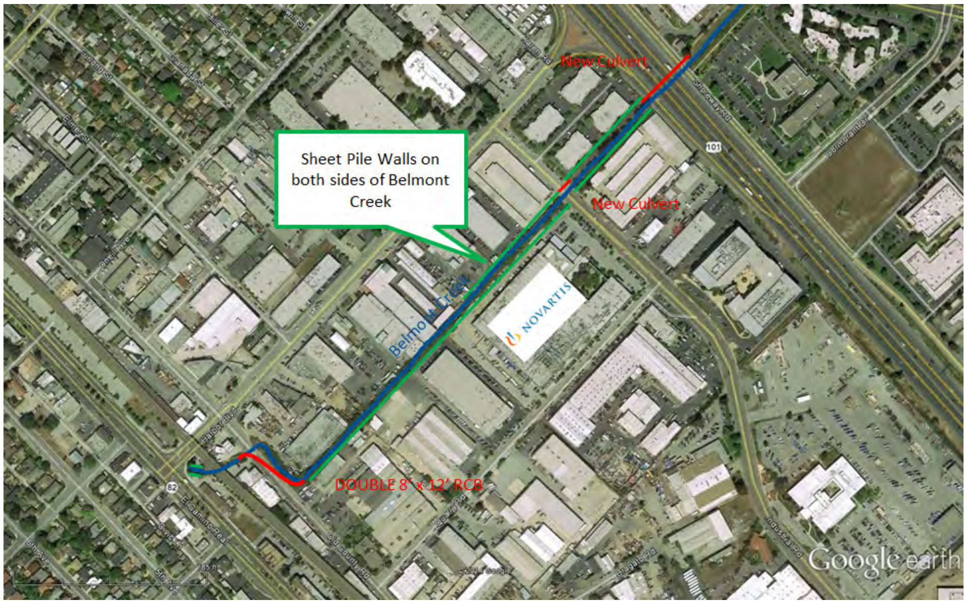

Figure 27. Alternative 7 Layout

4.9 Alternative 8: Floodgate and Pump at Shoreway Road/Marine Parkway

This alternative proposes a tide gate at the bridge over Belmont Creek near Marine Parkway with a pump station that discharges into Redwood Shores Lagoon, which is within Redwood City's right-of-way. The tide gate would be raised prior to large storm events and the reach of the creek between the tide gate and US 101 would act as a storage basin. After the storm event, the gate would be opened to release flows into O'Neill and Belmont sloughs. In the event of larger flows, the pump station would be activated to discharge flows that overwhelm the storage capacity of this area. With tidal influence extending to just upstream of Industrial Road, this alternative was developed to reduce or eliminate the downstream control so tidal stage would not factor into the hydraulics of large storm events. See Figure 28 for a layout of this alternative. This alternative was selected for further assessment.

Figure 28. Alternative 8 Layout

4.10 Alternative 9: Floodgates and Pump near Oracle Bridge

This alternative follows the principle of Alternative 8, but on a larger scale. Multiple tide gates and a pump station would be installed along the Oracle Bridge at Belmont Slough. The gates would be closed prior to large storm events to transform the area into a detention basin. In the event of larger storms, the pump station would activate and discharge flows into Belmont Slough to minimize inundation. This alternative would include the area from Alternative 8 as well as additional areas shown on Figure 29. This alternative was not selected for further assessment.

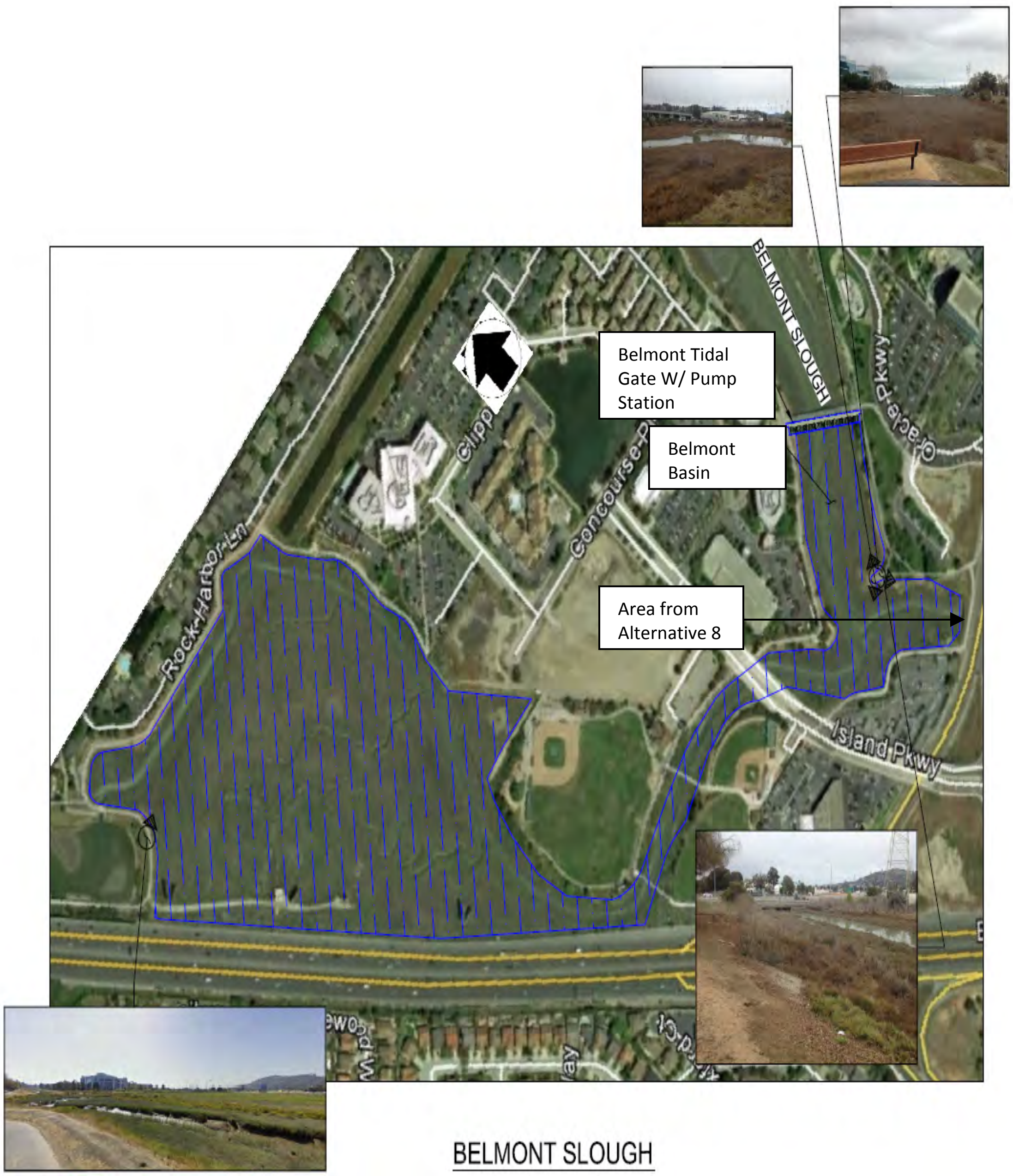

Figure 29. Alternative 9 Layout

4.11 Modified Alternative 6

During the stakeholders meeting, Alternative 6 received the fifth-most amount of votes for assessment through modeling. In order to better understand Alternative 6 and to see what would be needed to reduce flooding during the 10-year storm event, larger box culverts were included in this modified version of Alternative 6. Figure 30 shows the layout for this alternative. Preliminary calculations showed promise in flood reduction, so the Project team found this alternative worth exploring.

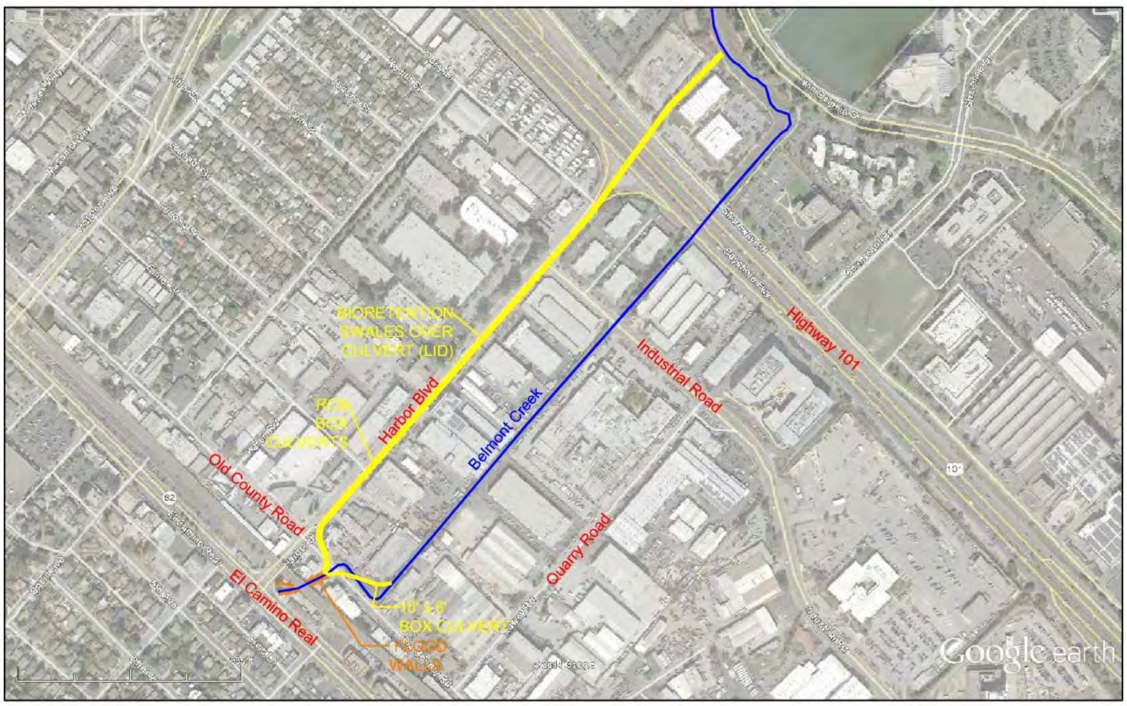

Figure 30. Modified Alternative 6 Layout

Like Alternative 6, this alternative would provide opportunities for LID measures along Harbor Boulevard as roadway excavation would be required for construction. Figure 31 presents potential locations for the installation of bulb-outs wherein bioretention areas could be installed above the proposed RCB along Harbor Boulevard; Figure 32 provides a typical cross section of this alternative.

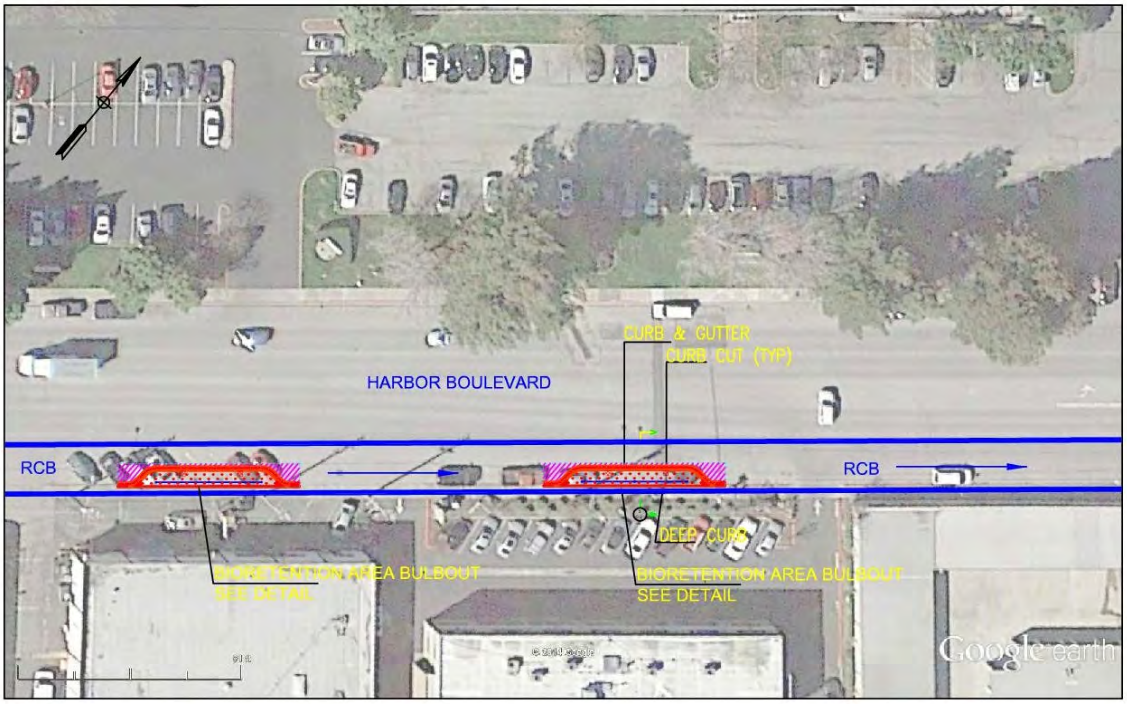

Figure 31. Harbor Blvd with Potential Green Street Improvements

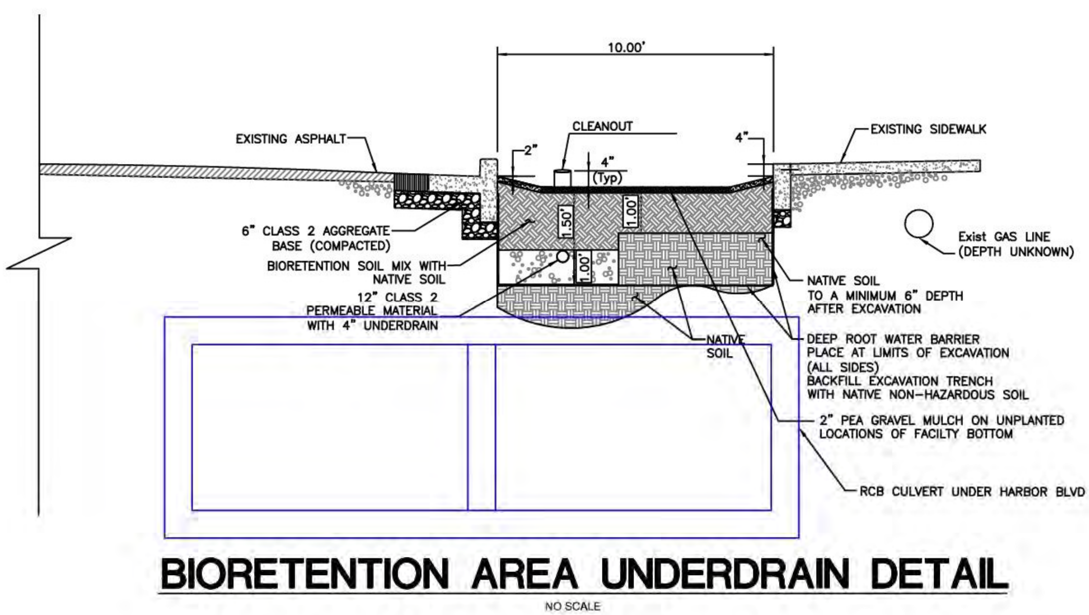

Figure 32. Typical Section of Green Street Improvements on Harbor Blvd

Utilities would require consideration during the design process for this alternative. Preliminary reconnaissance indicates the potential presence of gas, sanitary sewer, and water force mains that may need to be replaced or relocated. Figure 33 shows approximate locations of utilities along Harbor Boulevard that were discovered during the field reconnaissance. If selected for further analysis, utilities would need to be investigated to more precisely analyze constructability and other logistics.

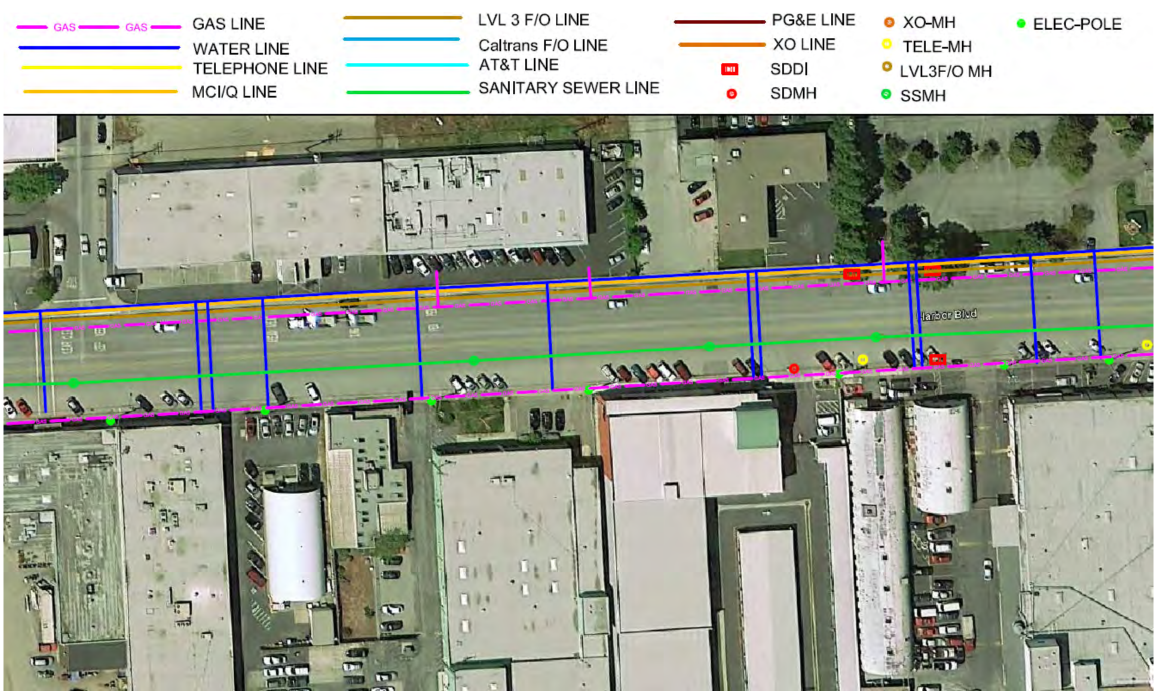

Figure 33. Approximate Utility Locations Based on Field Reconnaissance

5 HYDROLOGIC MODELING

This section of the report describes the hydrologic modeling effort, and Appendix D provides the model and additional model documentation.

5.1 Approach

Section 2.4.1 provides an overview of the hydrologic modeling approach and methodology.

5.2 Watershed Spatial Data

5.2.1 Topography

Where available, the USGS 1/9 arcsecond National Elevation Dataset (NED) was used to acquire elevations within the watershed. In the locations where 1/9 arcsecond data were not available, 1/3 arcsecond NED data were used. A digital elevation model (DEM) was created in ArcMap using these data. The elevations in the watershed are shown in Appendix B.

5.2.2 Flow Paths

Flow paths were delineated based on the DEM, observations in Google Street View, storm drain network layouts provided by the City of Belmont, and locations where open channels are visible.

5.2.3 Soils

Hydrologic soil groups (HSGs) were assigned within the watershed based on data downloaded from the Natural Resource Conservation Service's (NRCS) Web Soil Survey. Most of the watershed is in the HSG D range, but there are some areas of B and C range soils. A large portion of the upper watershed and an area near Twin Pines Park are composed of HSG C soils. The HSG map is included in Appendix B.

5.2.4 Land Use

The land use within the watershed was delineated in ArcMap based on the aerial topography available in ESRI's World Imagery reference layer. The land use map is included in Appendix B.

5.2.5 Mean Annual Precipitation

The mean annual precipitation data were obtained from PRISM Climate Group. The mean average precipitation was calculated for each subbasin, and is included in Appendix B.

5.3 Watershed Delineation

Watershed areas were delineated based on topographic data, storm drain system data provided by the City of Belmont, and storm drain inlets and drainage patterns observed in Google Street View. HEC-GeoHMS was used to delineate the initial watershed areas, which were edited as necessary based on observed drainage patterns that are not accurately represented in the DEM. A total of 18 watershed areas were delineated. The watershed areas, reaches, and junction names are shown in Appendix C. The watershed areas range from approximately 0.9 ac to 34 ac; however, most are similarly sized and between 5.6 ac and 17.9 ac. The HEC-HMS link-node network is shown in Figure 34.

5.4 Infiltration Model

Infiltration was initially modeled using NRCS Technical Release No. 55 (TR-55) *Urban Hydrology for Small Watersheds* (1986) curve numbers, which are based on land use, cover quality, and HSG. However, the TR-55 infiltration did not match the calibration well (see Section 5.8), so the Initial and Constant loss method was used instead. The HEC-HMS inputs for this portion of the model are the initial loss, constant rate, and impervious percent. The initial loss was set to 0.1 in. for all subbasins. The initial values of the constant rate were set based on the weighted average of HSG within the subbasins. The constant rate was changed based on the calibration process (see Section 5.8).

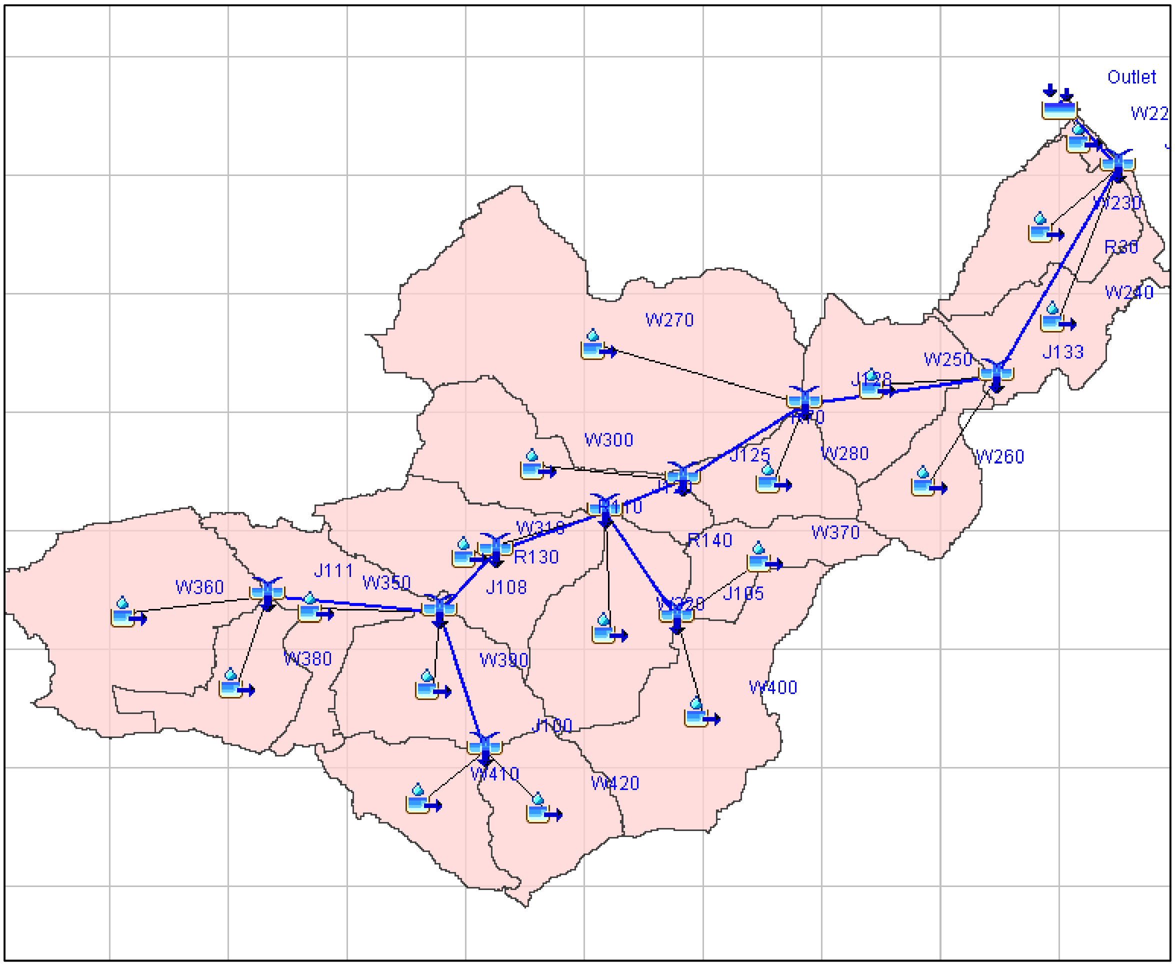

Figure 34. HEC-HMS Link Node Network for the Belmont Creek Watershed

5.5 Subbasin Flow Routing

Flow routing within the subbasins was modeled with the SCS Unit Hydrograph transform method, which requires the lag time as the input, and the lag time was estimated as 0.6 times the time of concentration. The time of concentration was calculated per the NRCS TR-55 graphical method, using the HEC-GeoHMS spreadsheet (see Appendix D). The time of concentration is the sum of the travel times for sheet flow, shallow concentrated flow, and channel flow.

5.5.1 Sheet Flow

The sheet flow travel time was calculated using the following equation:

$$T_t = \frac{0.007 L^{4/5} n^{4/5}}{P_2^{1/2} S^{2/5}}$$

Where:

T_t = sheet flow travel time (hr)
L = sheet flow path length (ft)
n = Manning's coefficient of roughness for sheet flow
P_2 = 2-year, 24-hr rainfall depth (in.)
S = slope of flow path (ft/ft)

The 2-year, 24-hr rainfall depth (P_2) was determined to be 2.47 in., per National Oceanic and Atmospheric Association's (NOAA) Atlas 14. The slope was calculated in ArcMap using the elevations at the beginning and end points of the sheet flow path. The sheet flow path ended where the flow path entered a defined channel (such as a gutter, drainage inlet, or ditch) or where the path length reached the limit of 300 ft.

Manning's n values used for the sheet flow were based on Table 816.6A of the *Caltrans Highway Design Manual*. For impervious sheet flow surfaces, 0.015 was used as the n value. For pervious sheet flow surfaces, 0.24 was used as the n value.

5.5.2 Shallow Concentrated Flow

The shallow concentrated flow velocity was determined using Figure 3-1 of TR-55. The equations are as follows (Appendix F of TR-55):

$$Unpaved: \quad V = 16.1345 s^{1/2}$$
$$Paved: \quad V = 20.3282 s^{1/2}$$

Where:

V = average shallow concentrated flow velocity (ft/s)
s = shallow concentrated watercourse slope (ft/ft)

The slope was calculated similarly to the slope of the sheet flow segment. The travel time is the length divided by the velocity calculated with the equation above.

5.5.3 Concentrated Flow

The concentrated flow travel time was estimated by assuming a travel time of 3 ft/s over the path delineated in ArcMap.

5.5.4 Lag Time

The time of concentration was the sum of the three travel time components. The time of concentration calculations are summarized in Appendix D. The lag time was assumed to be 0.6 times the time of concentration.

5.6 Reach Flow Routing

Flow routing in the reaches was modeled using the Muskingum-Cunge method with no loss method. The inputs to this calculation are the length, slope, Manning's n, and channel dimensions. The results are relatively insensitive to the channel dimension variables. The length and slope were calculated based on the HEC-GeoHMS model. Manning's n values were set to 0.04.

5.7 Precipitation Model

The design storm hyetograph was developed per the Santa Clara County *Drainage Manual* (Schaff and Wheeler 2007). The unit rainfall pattern used by Santa Clara County is based on the three-day event that occurred in December 1955, which is considered the storm of record for northern California. According to the *Drainage Manual*, the storm is balanced for use when events with durations of less than 24 hours are critical, which is likely for this site.

The 10-year, 24-hour precipitation depth was calculated using the TDS Regional Equation, as presented by the *Drainage Manual*:

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

Where:

$x_{T,D}$ = precipitation depth for return period of T years and duration of D hours
$A_{T,D}$ and $B_{T,D}$ = coefficients per *Drainage Manual* Tables B-1 and B-2
MAP = mean annual precipitation (in.)

The mean annual precipitation depth was based on spatial data from PRISM. The average of a DEM within the watershed boundaries was used as the mean annual precipitation variable in the equation. This process was used for each subbasin individually, and for the entire watershed as a whole. The resulting hydrographs from each method were similar.

The mean annual precipitation of each subbasin ranged from 19.5 in. at the east end of the watershed to 26.8 in. at the west end of the watershed. The average mean annual precipitation was 23.4 in. over the whole watershed. A map showing the mean annual precipitation average for each subbasin is included in Appendix B.

The resulting 10-year, 24-hour precipitation depths ranged from 3.74 in. to 4.92 in. The watershed overall average precipitation depth was 4.38 in. Per NOAA Atlas 14, the 10-year 24-hour precipitation depth for the watershed is approximately 3.82 in. The depth obtained from the Santa Clara County *Drainage Manual* is approximately 15% greater than the NOAA Atlas 14 depth. The depths from the Santa Clara County *Drainage Manual* were used to be consistent with the hydrologic method.

5.8 Calibration

Calibration was performed on the hydrologic model using time series depth data measured by WRECO in the field during a precipitation event on February 28, 2014. The monitoring methods are described in Section 2.2.2.2. The maximum flow depth reached during the event was approximately 2.0 ft, and the measured flow rate at a depth of approximately 0.85 ft was 37 cfs. Manning's equation was used to calculate the slope of the channel at the monitoring location adjacent to the Novartis facility. The result was a slope of approximately 0.4%, which is reasonable. This confirmed the n values that were used, and allowed WRECO to calculate the flow rate varying with time corresponding to the depth measurements. The resulting peak flow rate is approximately 150 cfs at a depth of 2.0 ft. The hydrograph for the February 28, 2014, event calculated by this process is shown in Figure 35.

The precipitation time series in various locations within the watershed were obtained from Weather Underground weather station data. The HEC-HMS model was run with precipitation from the February 28, 2014, event, and compared with the calculated peak flow rate. An optimization trial in HEC-HMS was used to calculate the constant infiltration rate components within the watershed by reducing or increasing infiltration rates by a constant ratio throughout the watershed. The resulting calculated versus measured flow rates are shown in Figure 35.

The hydrograph used in this calibration was much smaller than a design event or the channel full event, but it is the only data available due to the droughty conditions during the 2013-2014 water year. There are no additional data available for use as a validation run. Using a small hydrograph calibration could result in conservative flows for design events, as infiltration rates determined could be lower during a longer event.

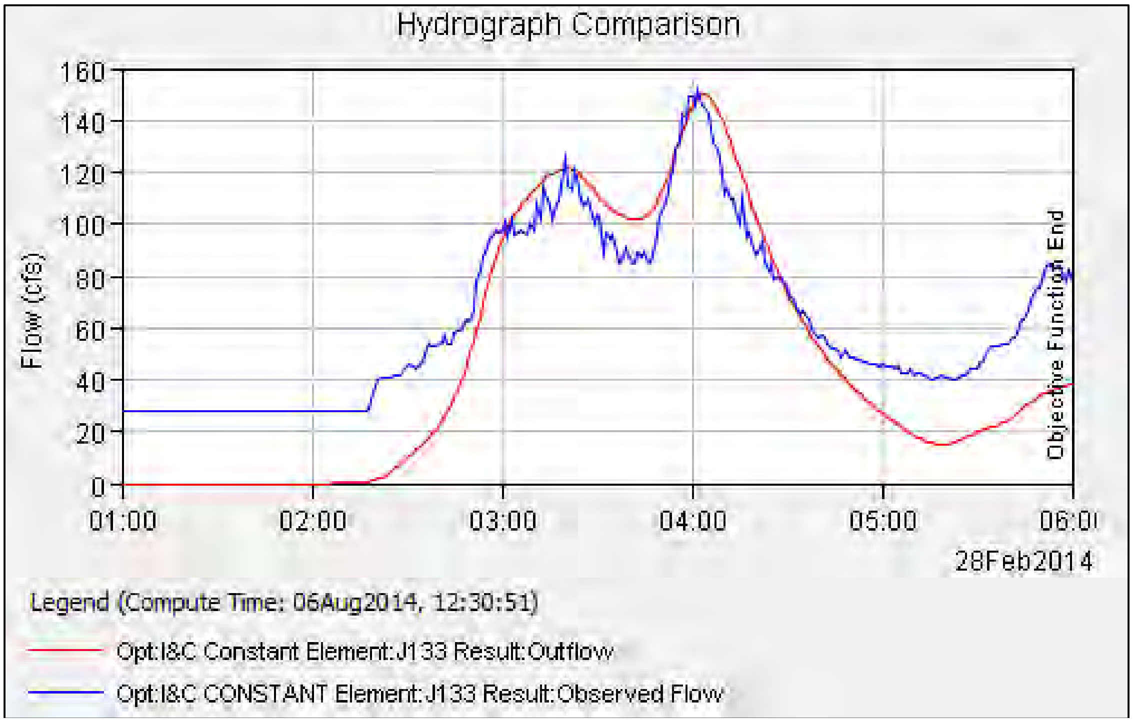

Figure 35. HEC-HMS Calculated Hydrograph versus Field-Measured (Observed Flow) Hydrograph for February 28, 2014, Storm Event

5.9 Results

The results of the HEC-HMS model are hydrographs at locations critical to the hydraulic model, such as upstream of reaches or at confluences. The hyetograph and hydrographs downstream of El Camino Real and at the discharge point to Belmont Slough are included in Figure 36. The 10-year peak flow adjacent to the Novartis property is approximately 1,400 cfs. Flows at El Camino Real and discharge point to Belmont Slough outfall are similar, because peak flow from the additional watershed area between the two is relatively small and occurs sooner than peak flow in the main creek. The peak precipitation in the hyetograph begins at 6:00. The peak at El Camino Real occurs at 6:45 and the peak at the discharge location to Belmont Slough occurs at 6:51.

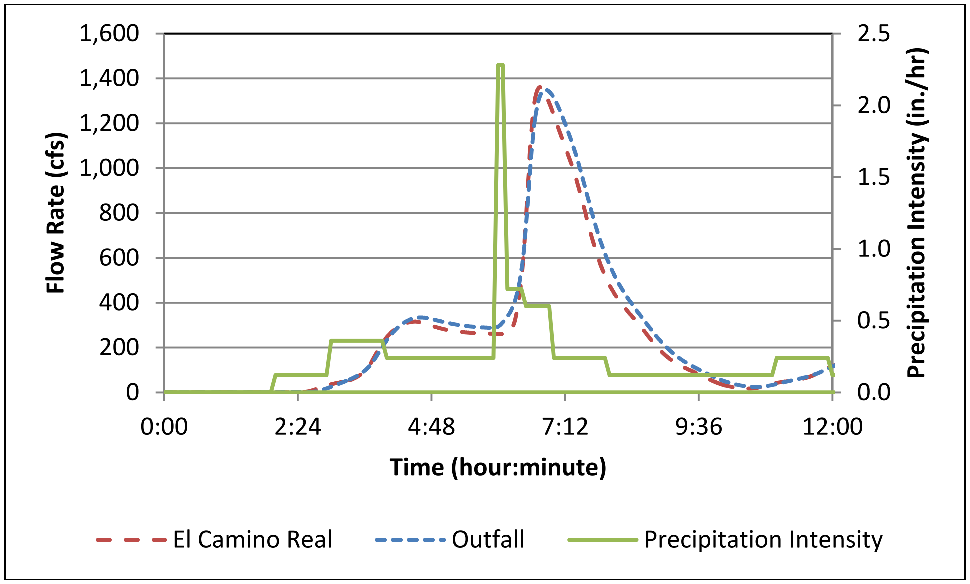

Figure 36. Summary of Hydrology Modeling Results

The flows calculated by this method assume that there are no constrictions or obstructions preventing the entire runoff volume from reaching the outfall once it has reached the channel. Therefore, although the HEC-HMS model calculates 1,400 cfs in Belmont Creek at the Novartis facility, that peak flow may not actually reach that location if it is attenuated upstream. The simulation of attenuation due to constrictions and obstructions is accomplished in the hydraulic model.

Figure 37 shows the peak 10-year flows through the inflow locations within the sub-watersheds for this study.

Figure 37. 10-year Maximum Flows at Inflow Locations

6 HYDRAULIC MODELING

This section of the report describes the hydraulic modeling effort of the existing and proposed conditions, and Appendix E provides the model and additional model documentation.

6.1 Methodology

Section 2.4.2 provides an overview of the hydraulic modeling approach and methodology, and the following sections provide additional detail.

6.2 Existing Condition

6.2.1 Channel Data

Survey data from RSE Inc. were imported into a topographic computer-aided design and drafting (CADD) file where both were integrated to create a surface of the existing condition. This topographic CADD file was then imported into GIS, where cross sections were cut and channel alignments were created so they could be imported into PCSWMM. Channel cross sections downstream of US 101 were cut from the DEM.

Friction and other head losses were represented in the model using the Manning's n value and the Average Loss Coefficient. Manning's n values for most of the channel were set to 0.05. In the larger channels downstream of US 101, 0.045 was used as the n value. The Average Loss Coefficient was typically set to 0.5 to represent expansion and contraction losses that are not otherwise calculated in the model. At the extreme channel bends, the coefficient was increased to 0.9.

6.2.2 Man-made Structures

Bridges and creek crossings were represented in the model as a set of parallel conduits. One conduit represented the geometry of the cross culverts, and the second conduit represented the flow over the bridge deck or other cover. The second conduit would only be used if the water surface elevation would overtop the bridge. Crossings included in the modeling effort include Chula Vista Drive, Caltrain tracks, Harbor Boulevard, Industrial Boulevard, and US 101.

Hydraulic losses at the inlets and outlets of crossings were represented using entry and exit loss coefficients. The entry coefficients were based on the hydraulic smoothness of the transition between the channel upstream and the crossing structure. Values ranged from 0.2 for a smooth transition to 0.7 for an especially abrupt transition. For typical transitions, 0.5 was used. The exit losses for all crossings were set to 1.0.

In addition, the culvert system from upstream of 6th Avenue to downstream of El Camino Real was modeled as a series of conduits. The geometry of these conduits was based on as-built plans. Inlet and outlet coefficients were used to represent the hydraulic losses due to configuration changes where the conduits significantly change shape. In addition, friction losses were represented with Manning's n values. A Manning's n value of 0.015 was used for concrete and 0.024 was used for corrugated metal pipe.

6.2.3 Two-Dimensional Mesh

A simulated 2D mesh was developed for the model in the area bounded by El Camino Real, the discharge point to Belmont Slough, Ralston Boulevard, and Holly Street, and the typical mesh spacing was set to 30 ft. At roads, the spacing was set to 20 ft. The spacing was determined by balancing computational time with the desired resolution of results. Buildings and other obstructions were added as an obstructions layer and the mesh did not go through them. The mesh was connected to the 1D channel model by large orifices at each cross section location. If the water surface in the cross section rose above a specific elevation, flow was split between the channel downstream and the 2D mesh. Runoff could enter a channel from the floodplain through the same orifice.

6.2.4 Downstream Control

The water surface elevation at the downstream end of the model was set to a tidal series, with the peak water surface occurring at the same time as the peak precipitation. The peak water surface was Mean Higher High Water (MHHW), and the Redwood City tidal gauge owned by NOAA was for used tidal data. Figure 37 shows the peak 10-year flows through the inflow locations within the sub-watersheds for this study.

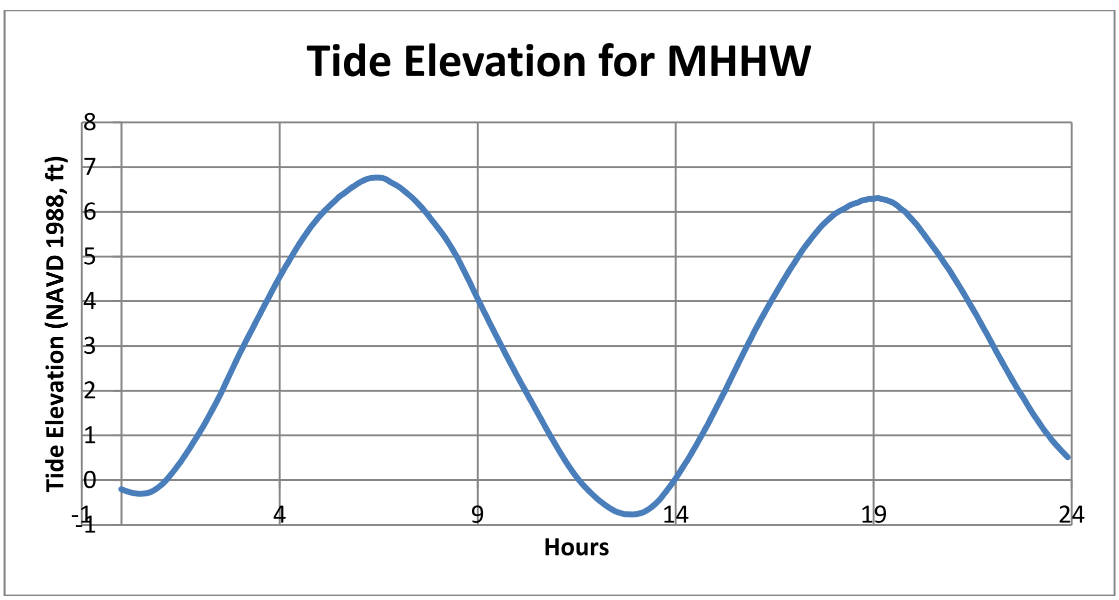

Figure 38. Tidal Curve for MHHW at the Downstream End

WRECO acknowledges the importance of sea level rise (SLR) and the challenges facing critical infrastructure from SLR. The *Committee on Sea Level Rise in California, Oregon, and Washington* estimated SLR in 2030 will be approximately 14.4 cm (5.7 in.) in 2030; 28.0 cm (11.0 in.) in 2050, and 91.9 cm (36.2 in.) in 2100. For this study, the 2050 SLR estimate was used on top of the tidal gauge data for the downstream condition. SLR estimates did not take into consideration any increase in storm duration or intensity due to climate change.

6.3 Proposed Condition

The proposed alternatives were modeled using the existing condition model with modifications to represent the properties of each alternative.

6.3.1 Alternative 1

The changes to Water Dog Lake and dam were made in the HEC-HMS model using a reservoir unit and a stage-storage-discharge relationship. The bathymetry of the lake was assumed to match the adjacent topography, if it continued in a linear fashion below the water surface. This was used in order to estimate lake storage volumes. The resulting modified hydrograph was input to the PCSWMM model.

6.3.2 Alternative 5

Floodplain restoration was modeled in PCSWMM using a storage unit and adding a weir connection between the channel and the storage unit. A pipe was added from the flow line of the assumed basin and the channel flow line downstream. Flow could transfer in both directions between the channel and the basin over the weir, which represented the channel bank. The weir height was set to the 2-year flow elevation.

6.3.3 Alternative 6

Additional conduits were added to the model to carry overflow from Old County Road down Harbor Boulevard. The ground elevations above the bypass were taken from the DEM. The bypass was assumed to cross US 101 at Harbor Boulevard and join the channel downstream of the existing US 101 crossing. The existing crossing was left in the model.

6.3.4 Alternative 7

Floodwalls were represented by disconnecting the 1D and 2D models in the affected reach. The height of the floodwalls was determined by taking the difference between maximum water surface elevation and the elevation of the adjacent ground.

Culvert improvements at Old County Road, Industrial Road, and US 101 were represented by modifying the conduits. Old County Road was changed from an elliptical culvert to a double 10-ft by 6-ft RCB. An additional barrel was added at the existing Industrial Road and US 101 crossings. The entrance loss coefficients were reduced to

0.2, which assumed that the new configuration would have a well-designed smooth transition. The additional barrels assumed that there would area available for installation, although such an area is currently unavailable.

6.3.5 Alternative 8

The floodgate was represented by a new conduit with a tide gate to prevent backflow. The floodgate was set to have a flow line approximately 1 ft above the existing channel flow line. The pump was represented as a pump link in PCSWMM, with a flow rate of 40 cfs.

6.3.6 Modified Alternative 6

The reconfigured Old County Road crossing and junction with the bypass were represented as a series of box culverts with high inlet losses. Although the inlet losses are high, they still represent a well-designed smooth transition.

6.4 Results

6.4.1 Existing Condition

6.4.1.1 2D Model Results: El Camino to Outfall

The existing condition hydraulic model indicated widespread flooding downstream of El Camino Real. Flows escaped the confines of the banks in several locations, and flowed in a generally northerly and northeasterly direction toward US 101, where it ponded until it passed through the US 101 cross culvert. Some of the flood flow continued to the east into the drainage system that discharges near Holly Street. The extra runoff floods the adjacent drainage system as well, which does not receive runoff from its watershed in this event. The maximum extents of the floodplain are shown in Figure 39. The maximum flooding occurs approximately 3 hours after the peak precipitation in the model.

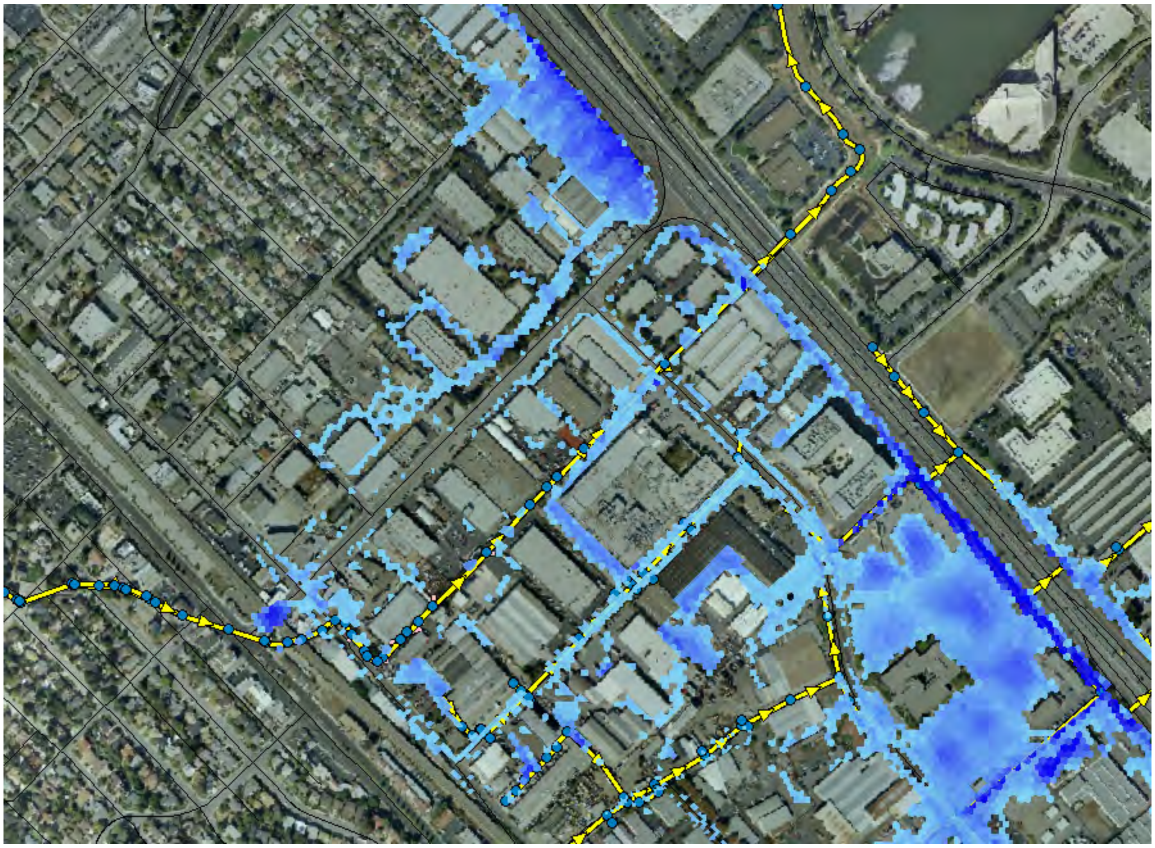

Figure 39. Existing Condition: 10-year Maximum Floodplain, Flooding at 9:00 in Model

The first location where flow escapes the banks is adjacent to the Novartis facility. Shortly after the channel is overtopped adjacent to Novartis, the channel immediately upstream of US 101 and at the 90-degree bend near Old County Road begins flooding. Overbank flow from Old County Road is small and does not travel far from the channel. However, the spill flows adjacent to Novartis and upstream of US 101 continue to disperse across the area (see Figure 40).

Approximately 20 minutes after peak precipitation occurs, flooding increases dramatically. Spill flow upstream and downstream of the Old County Road crossing rapidly enters the Harbor Boulevard undercrossing of the Caltrain tracks, and spreads to the south, parallel to Caltrain. It inundates the intersection of Old County Road and Harbor Boulevard, and begins flowing down Quarry Road about 30 minutes after the peak precipitation (see Figure 41).

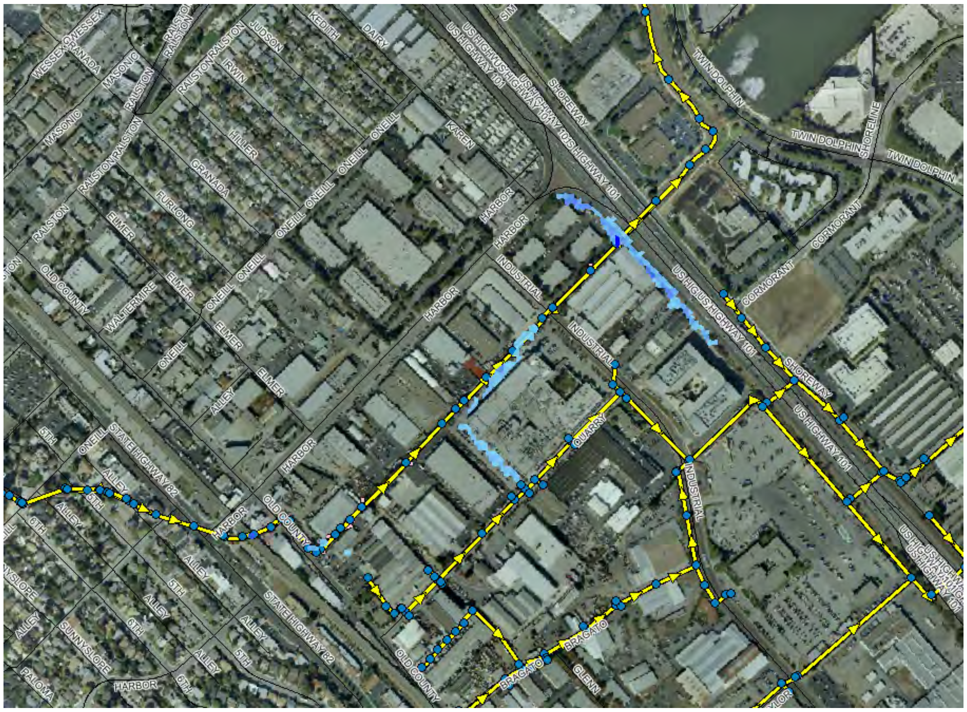

Figure 40. Flooding at 6:10 in the Model

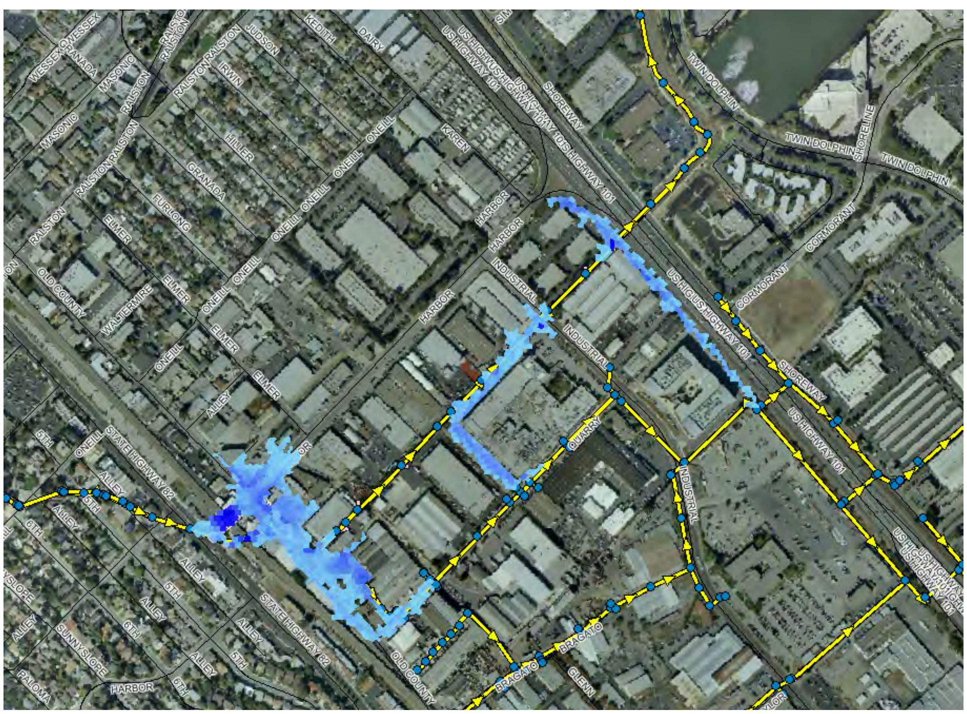

Figure 41. Flooding at 6:30 in the Model

The same flooding patterns continue, with flood flows spreading to the north of Harbor Boulevard and flowing down Quarry Street. At approximately 50 minutes after the peak precipitation, flooding along Quarry Street reaches Industrial Road and flows south along Industrial Road. At this point, flooding along Harbor Boulevard near Old County Road spreads toward Bragato Road.

Figure 42. Flooding at 6:50 in the Model

The flooding continues to spread out to the north and to the east. Flood waters reach US 101 near Karen Road and enter the parking lot of the PG&E building approximately 1 hour after peak rainfall. By 2 hours after peak rainfall, the flood wave begins to recede at the upstream end (near the Caltrain tracks), and has spread out to O'Neill Street and Dairy Street in the north and past Taylor Way to the south. Flood waters border US 101 from Ralston Avenue to Holly Street.

A majority of flooding clears from the Harbor Boulevard/Belmont Creek vicinity by 4 hours after the peak precipitation though the Novartis property is still inundated until the end of the model (see Figure 43). Areas that remain inundated at the end of the model are shown in Figure 44. These areas likely have local drainage systems that are not represented in the model, so flooding indicated at these locations may be conservative when the surrounding areas begin to recede.

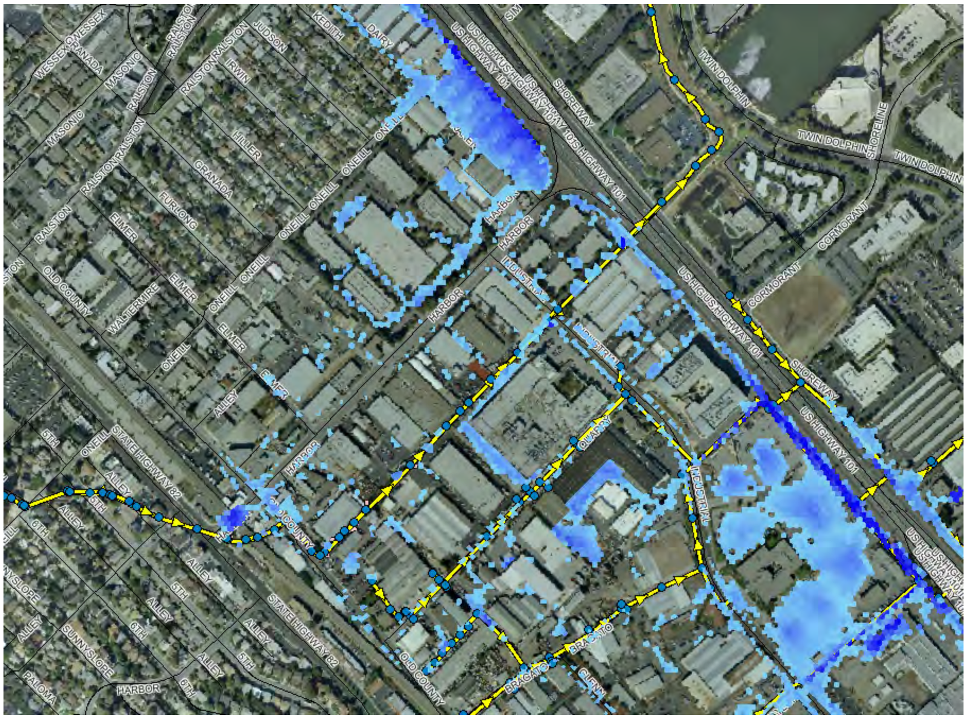

Figure 43. Flooding at 10:00 in the Model

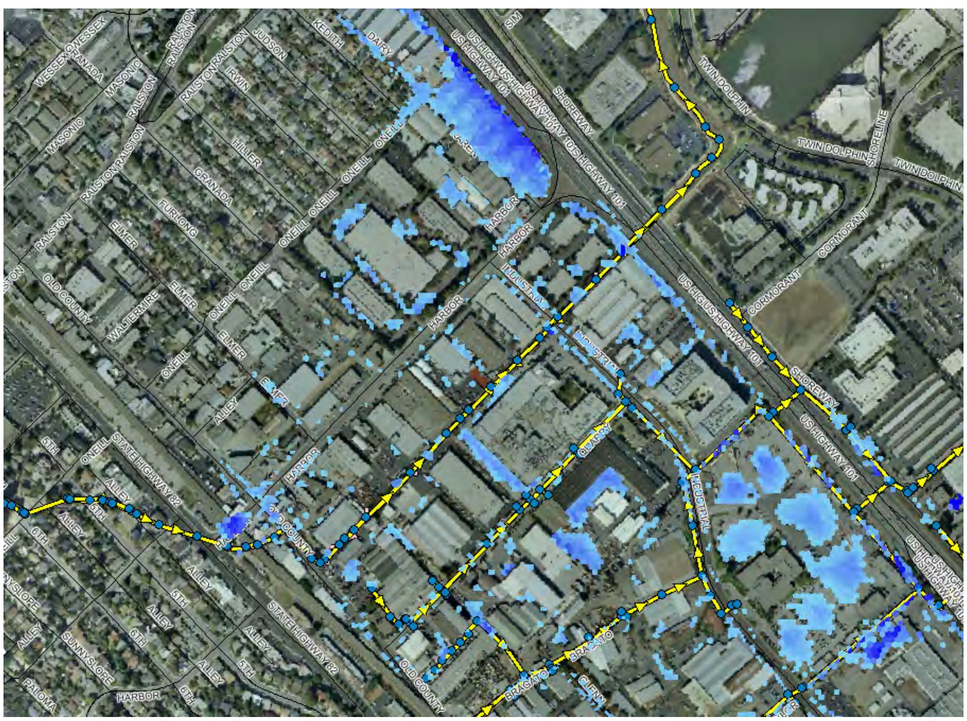

Figure 44. Flooding at the End of the Model

Approximately 950 cfs of water flows downstream through the culvert system under El Camino Real, and approximately half is lost due to spill flow. From El Camino Real to upstream of Old County Road, the peak spill flow is approximately 400 cfs. Some of the lost flow reenters the channel locally, and by the time the channel passes through the 90-degree bend, the flow in the channel has increased from 425 cfs to approximately 525 cfs.

The peak flow out of the channel adjacent to the Novartis facility is approximately 200 cfs. The peak flow is 420 cfs through the Industrial Road crossing.

The peak overflow upstream of the US 101 crossing is approximately 90 cfs, most of which continues south along US 101. The peak flow through the crossing is approximately 410 cfs.

The peak flow at the downstream end of the model, at Ralston Avenue, is approximately 430 cfs.

6.4.1.2 Sea Level Rise Impacts to the Existing Condition

SLR was applied to this existing condition to simulate the potential impacts to the flood water surface elevations. Figure 45 shows that while SLR will increase the elevation of flood waters, the impact is insignificant, especially in the upstream portions.

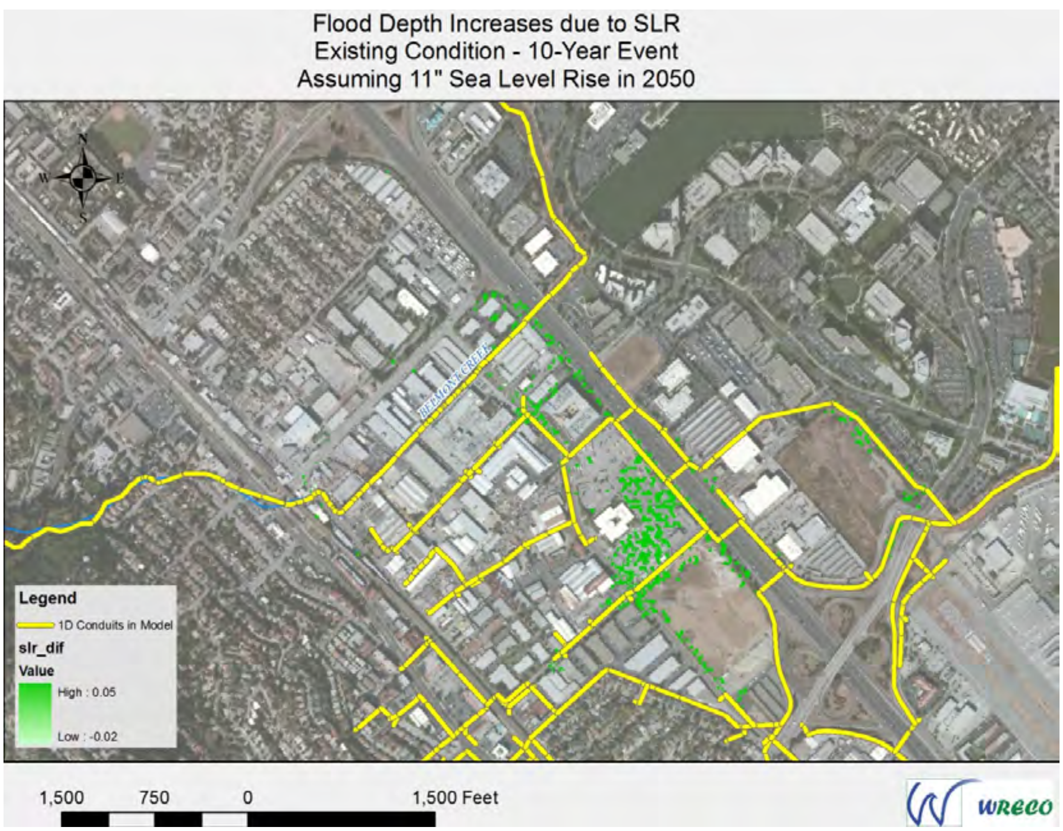

Figure 45. Sea Level Rise Impacts to the Existing Condition

The peak flooding extents are shown in Figure 46 and Figure 47 with and without SLR influence, respectively.

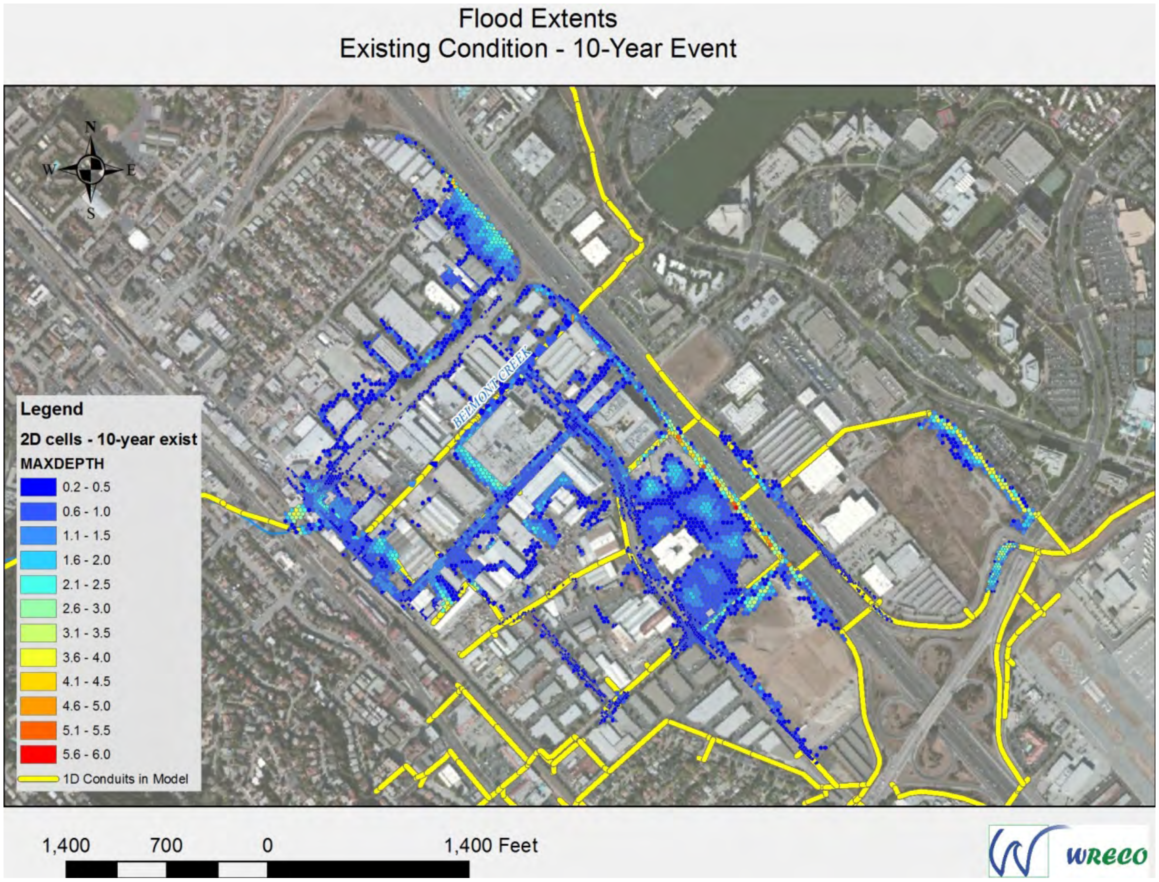

Figure 46. Peak 10-year Flooding Extents in the Existing Condition

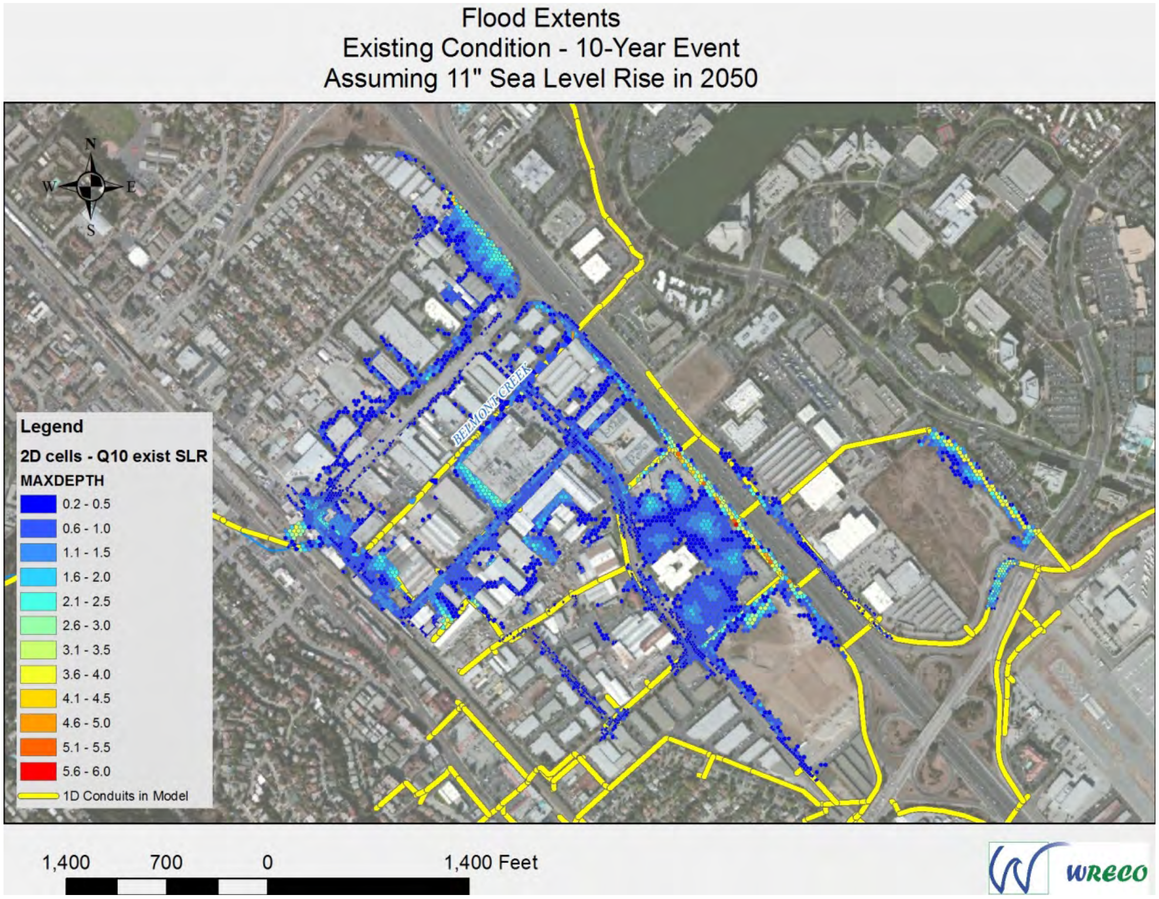

Figure 47. Peak 10-year Flooding Extents in the Existing Condition with Sea Level Rise

6.4.1.3 1D Model Results: Upstream of El Camino Real

In addition to the flooding between El Camino Real and US 101, where the hydraulic model is focused, backwater was observed in some locations in the system upstream.

At the upstream end of the culvert system through Belmont that starts at the intersection of 6th Avenue and O'Neill Avenue, the peak flow rate is approximately 1,060 cfs. However, upstream of this location, the peak flow rate is approximately 1,300 cfs, indicating that 240 cfs is attenuated by backwater behind the culvert system. The channel profile is shown in Figure 48.

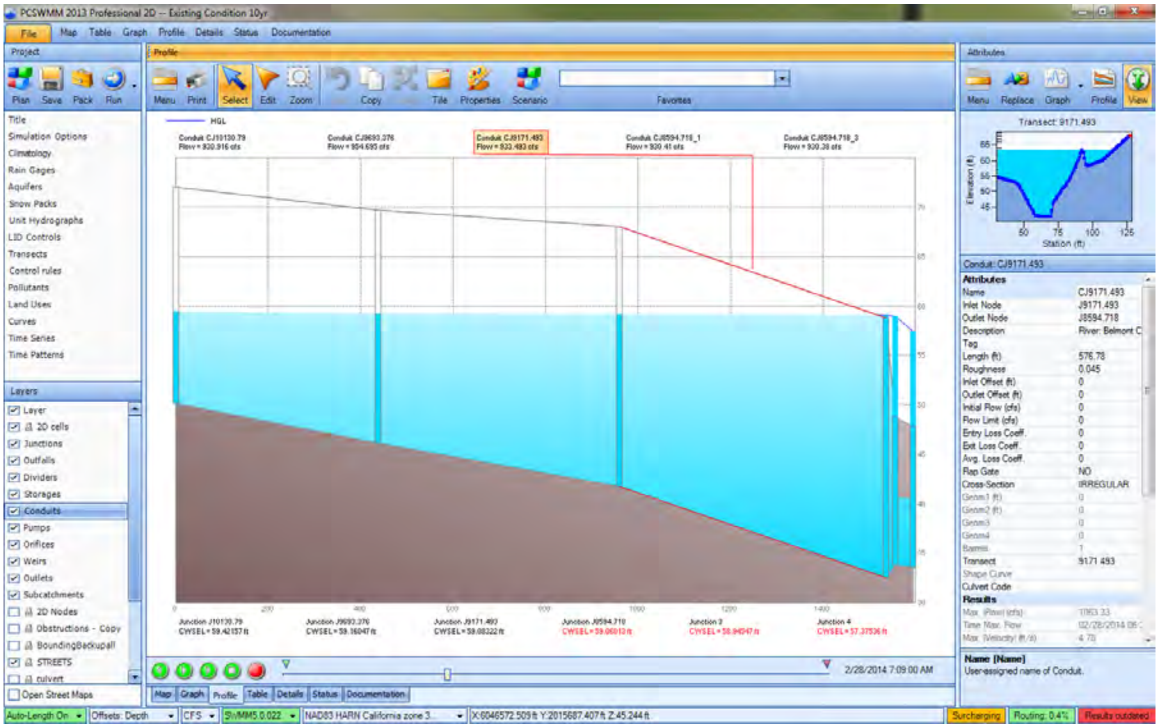

Figure 48. Culvert Backing Up within the Channel Profile

This portion of the model is 1D, as it is located upstream of the primary area of interest. Flows that may overtop the roadway in this location remain within the conduits in the model. Those flows may actually overtop the roadway instead of entering the culvert, and make their way downhill toward El Camino Real. Along the system, the water surface also has the potential to be above ground, and any flows that escape in this manner would also continue to El Camino Real. Any escape flows following this route were not included in this model and should be further evaluated. They may enter the 2D model boundary sooner than the model indicated, resulting in greater peak flows than shown.

6.4.2 Alternatives

The modeled alternatives 1, 5, 6, 7, and 8 all had an insignificant impact on the flooding between El Camino Real and US 101. Alternatives that include modifying the underground system upstream of the backup (e.g. Alternative 1 and Alternative 5) would likely have a minimal impact, because the peak flow was already attenuated by the backwater effect.

Due to downstream constrictions and the distance to the lower watershed, Alternative 1 (modifying Water Dog Lake) would have a minimal impact on the extent of inundation. Alternative 5 (floodplain restoration at Twin Pines Park) would provide minimal flood reduction benefits, because the box culvert system downstream of Twin Pines Park currently acts as a constriction for upstream flows. However, Twin Pines Park currently acts as a temporary storage basin. Figure 49 shows the 10-year flooding impacts downstream of El Camino Real with Alternative 5, which is similar to the existing condition. Figure 50 shows that the reduction in flooding from the existing condition to the 10-year storm for Alternative 5 is minimal, with the water surface elevation throughout a majority of the inundated area decreasing by 0.01 to 0.02 ft.

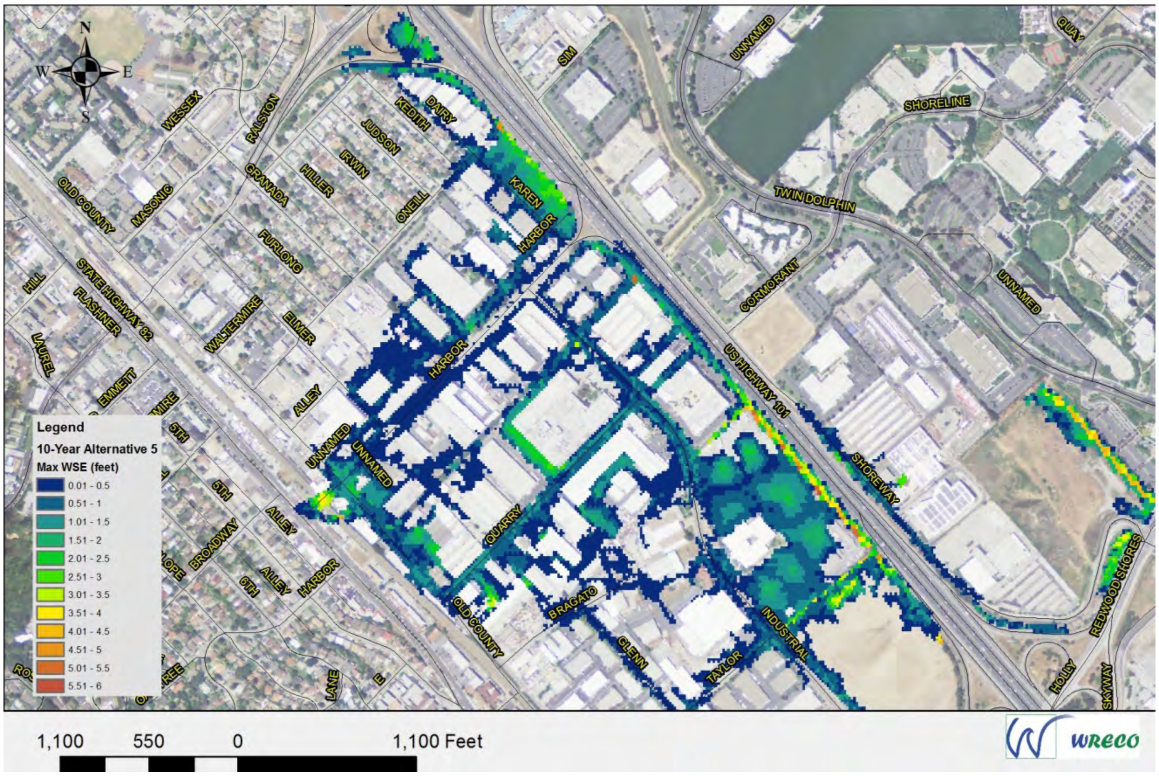

Figure 49. 10-Year Storm Flooding with Alternative 5

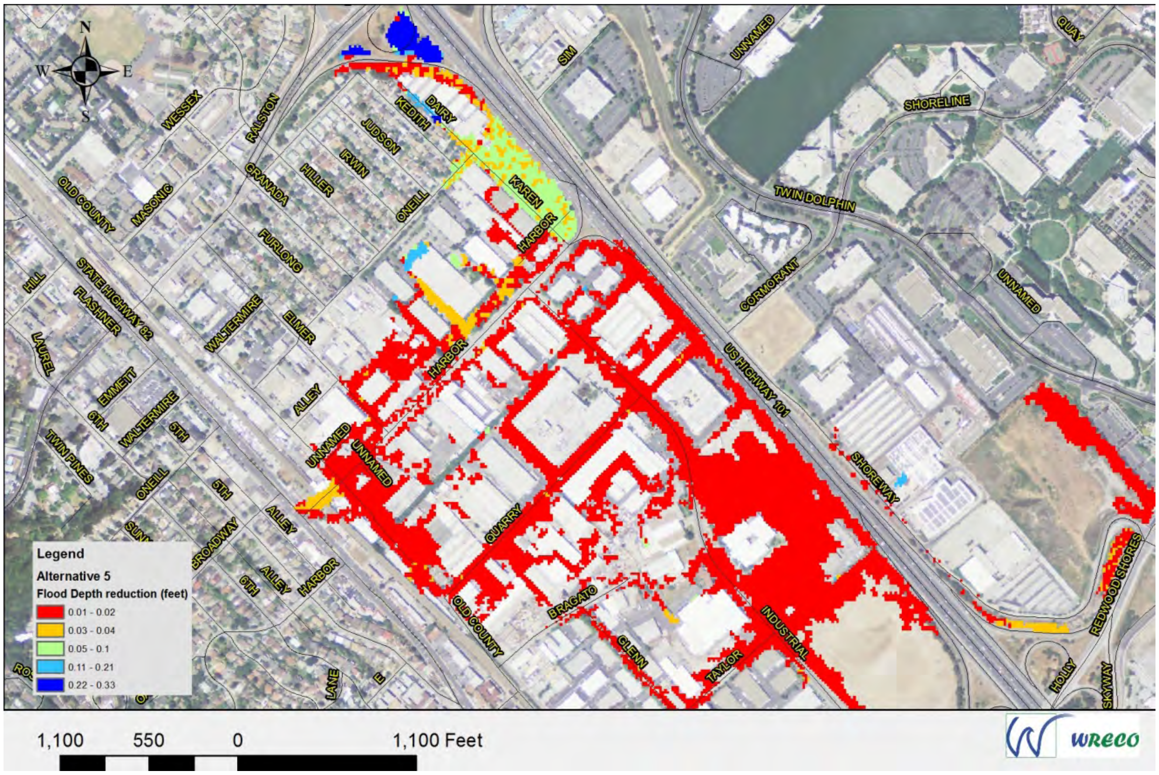

Figure 50. 10-Year Storm Flooding Reduction from Existing Condition with Alternative 5

Although Alternative 5 may present minimal benefits to flood reduction, this alternative provides an opportunity to improve water quality and riparian habitat through the establishment of a floodplain. This alternative could also act as a regional example of the ideal floodplain with a bankfull channel and overbank benches for higher flows. The floodplain areas would retain sediment through a depositional process, thereby reducing bank erosion and sediment contributions to downstream reaches.

Culvert improvements to Industrial Road and US 101 would have impacts on flood reduction in the current system, as those culverts convey received flows. Approximately half of the peak flow that gets to El Camino Real is lost by the time Belmont Creek passes through the 90 degree bend, due to the undersized channel and creek crossings. Additional flow is also lost near the Novartis facility due to the undersized, constructed channel. Therefore, by the time the flood wave reaches Industrial Road and US 101, the flows in the channel are less than half of the peak flow observed at El Camino Real, and less than one third of the peak flow observed farther upstream.

Improving the cross culvert under Old County Road would reduce the backwater caused by the hydraulic constriction. However, the additional flows conveyed by an improved culvert would spill over the banks of Belmont Creek downstream.

In Alternative 6 (parallel bypass along Harbor Boulevard), the addition of parallel overflow pipes would not significantly reduce the flow to the creek channel unless they are significantly oversized. Installing a double 6-ft by 10-ft RCB would reduce flow to Belmont Creek and avoid flooding, but backwater would spill over at El Camino Real and the Harbor Boulevard undercrossing. See Figure 51 for flooding during the 10-year storm event with Alternative 6, and Figure 52 for 10-year flooding reduction from the Existing Condition for Alternative 6.

With the improved road crossings, functional floodwalls would be too tall to be practical, because the channel is so undersized. In addition, as the water surface gets within the channel confined between flood walls gets higher, additional local runoff would need to be stored before discharging into the channel.

Alternative 8 (tide gate at Marine Parkway), which proposes a floodgate and pump station, would not have a significant benefit. The constructed channel downstream of Old County Road is significantly undersized, such that flows would overtop the banks upstream of backwater effects created by the tide gate. Therefore, the water surface elevation within the creek adjacent to the Novartis facility and farther upstream is independent of the water surface elevation downstream, and any improvements to the channel capacity downstream would not improve the current flooding conditions. By the same logic, Alternative 9 would also not have a significant flood reduction benefit.

Figure 51. 10-Year Flooding with Alternative 6

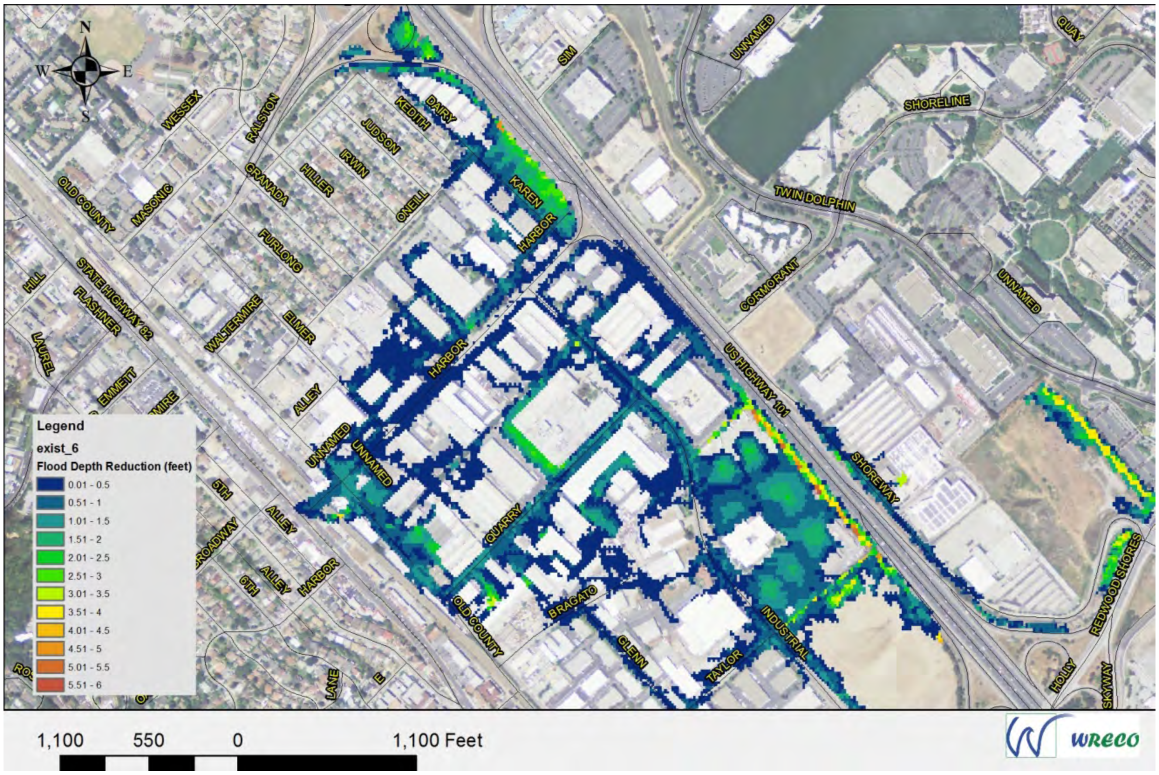

Figure 52. 10-Year Storm Flood Reduction from Existing Condition with Alternative 6

The models of the voted-upon alternatives show that there would still be inundation during the 10-year storm event. Analyzing the options have shown constrictions in the channel, culverts and basins being undersized, or a combination of the two. WRECO then analyzed a separate alternative that takes a combination of a few of the selected ones and enhanced them to the extent to be able to convey the 10-year storm event.

6.4.3 Modified Alternative 6

Because none of the alternatives provided a significant flooding reduction during the modeled 10-year event, combinations of alternatives were studied to find a feasible solution to convey the 10-year storm event. The resulting solution includes:

- A new Old County Road crossing that leads to a split between the bypass and Belmont Creek.
- RCBs from Old County Road diagonally to Harbor Boulevard, then along Harbor Boulevard across US 101 and discharging at Belmont Slough. See Figure 53 for the sizes of culverts, number of barrels, and layout of this alternative. These culverts were modified to avoid crossing private property.
- New crossing of US 101 for the bypass.
- RCB from Old County Road to the straightened segment of Belmont Creek to straighten the existing 90-degree bend to two, 45-degree bends.

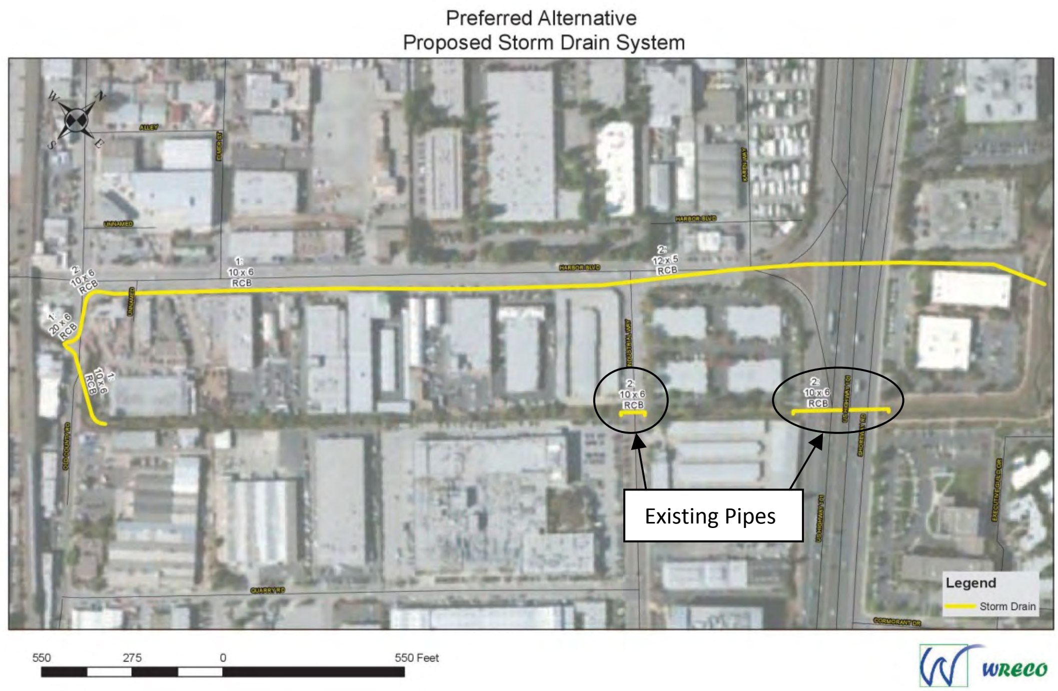

Figure 53. Layout for Modified Alternative 6

Implementing this combination of improvements minimized the extent of flooding during the simulated 10-year storm event. The analysis did not include any flows that may spill onto roadways farther upstream in the watershed and reach El Camino Real via overland routes.

WRECO has also analyzed this option with creek widening within the reach of Belmont Creek along the Novartis facility. This option would reduce the number of parking spaces at the Novartis facility and create a 40-ft floodplain for hydraulic conveyance and water quality benefits. For this additional option to work, a portion of the creek by Old County Road would need to be dredged, as that area is a hydraulic constriction that caused overbank flows in the models. However, the improvements to flood water surface elevations associated with a floodplain along the Novartis facility are negligible.

A particular challenging aspect to this improvement would be right-of-way acquisition to accommodate channel widening. Although Novartis has shown good faith in analyzing and considering widening adjacent to their facility, they do not own the parcel. This site is currently leased by Novartis, and it would be up to the land and property owner to negotiate any property acquisition or easements. With right-of-way acquisition and a lack of significant flood reduction, widening the channel in lower Belmont Creek may not be cost-effective for the anticipated result.

7 RECOMMENDATIONS AND CONCLUSIONS

7.1 Preferred Alternative

The modeling effort suggests that the selected alternatives may not have a significant benefit during high-intensity storm events, but would have an impact on more frequent storm events. Models suggest that the root cause of flooding is insufficient carrying capacity in the constructed channel in the lower watershed, rather than tidal influence or a lack of storage. The reach between Old County Road and US 101 is undersized along the cross section to convey the 3.1-sq-mi watershed. With regard to flood reduction, the most advantageous improvement would be channel widening in this reach. However, right-of-way constraints and property acquisition would make widening along the entire reach infeasible.

The alternatives analysis indicated that only Modified Alternative 6 would convey the 10-year storm event without inundation. Therefore, the preferred alternative, as described below, is Modified Alternative 6 with elements from Alternative 7 and Alternative 5 added to address the undersized constructed channel and sedimentation issues, respectively.

Modified Alternative 6 proposes an upsized bypass culvert along Harbor Boulevard to receive high flows during storm events from where Belmont Creek daylights at Old County Road. The bypass culvert would discharge into Belmont Creek just downstream of US 101. Additional culverts would be constructed at Industrial Road and US 101 to increase conveyance capacity, and LID measures would be installed along Harbor Boulevard (Figure 31 and Figure 32).

Moreover, floodplain restoration and the construction of a low-flow channel in Twin Pines Park, as described in Alternative 5, would address sedimentation and aggradation issues in downstream reaches (Figure 54); the preferred alternative does not include the storage basin identified in Alternative 5 (Figure 55).

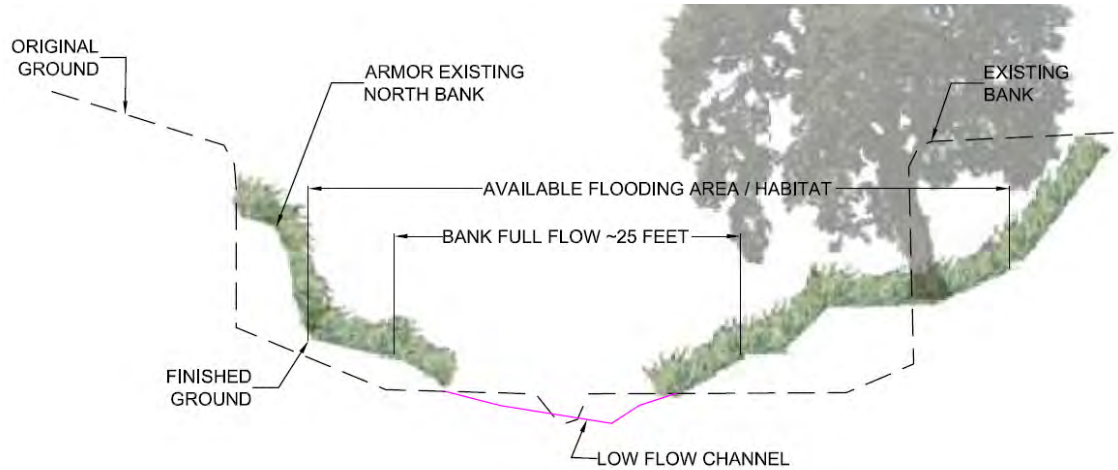

Figure 54. Floodplain Restoration, Typical Section

BELMONT CREEK WITHIN TWIN PINES PARK

Figure 55. Conceptual Layout of Floodplain Restoration in Twin Pines Park

Incorporating these structural components of a natural stream into this reach of Belmont Creek would enhance beneficial fluvial processes, such as conveying flood flows, improving sediment transportation as well as riparian and aquatic habitat, and enhancing water quality through the maintenance of dissolved oxygen levels, moderation of pH, turbidity, and nutrient loading, and reduction of biochemical oxygen demand (RWQCB 2003).

Incorporating LID measures and components of stream restoration with infrastructural improvements would mitigate potential water quality and riparian impacts associated with such a conveyance and constitute an improvement to the watershed. In this way, the preferred alternative is a complete and potentially self-mitigating project, while satisfying the criteria of attenuating inundation, enhancing water quality, and improving riparian and aquatic habitat. WRECO recommends pursuing this alternative through further analysis.

Please note that this alternative is conceptual, and subsequent studies should analyze it in more detail to assess benefits, as well as logistical and constructability issues, such as property acquisition, utility relocation, and cost.

7.2 Cost Estimate for Preferred Alternative

A preliminary cost estimate was developed for the Modified Alternative 6. This estimate is based on recent construction costs and may change in the future. Because it is preliminary, a contingency of 30% was added to the cost to account for the conceptual stage of the plan. Table 10 shows the breakdown of potential bid line items for this alternative.

Table 10. Preliminary Cost of Modified Alternative 6

Item Description	Unit of Measure	Estimated Quantity	Unit Price	Item Total
Roadway Excavation	CY	25,000	$100.00	$2,500,000
Irrigation Facilities	SQYD	2,000	$20.00	$40,000
Underdrain Cleanout	EACH	15	$500.00	$7,500
Underdrain	LF	2,700	$30.00	$81,000
Concrete (Box Culvert)	CY	4,500	$900.00	$4,050,000
Steel (Box Culvert)	TON	500	$2,000.00	$1,000,000
Bioretention Soil	CY	500	$75.00	$37,500
Permeable Rock	CY	70	$100.00	$7,000
Plant Establishment	LS	1	$10,000.00	$10,000
Temporary Construction Site BMPs	LS	1	$50,000.00	$50,000
Liner Plants	EA	400	$5.00	$2,000
Remove Concrete	LF	900	$100.00	$90,000
Traffic Control	LS	1	$80,000.00	$80,000
Mobilization	LS	1	$150,000.00	$150,000
Floodwalls	SQFT	12,000	$25.00	$300,000
Temporary Shoring	LS	1	$500,000.00	$500,000
Alternative 5 Mitigation	LS	1	$1,990,000.00	$1,990,000
SUBTOTAL				$10,895,000
CONTINGENCY (30%)				$3,268,500
TOTAL				$14,170,000

CY = Cubic Yard
SQYD = Square Yard
LF = Linear Foot
LS = Lump Sum
SQFT = Square Feet

7.3 Policy Prescriptions

Currently, bank armoring and channel modifications are prevalent within the watershed. Reach B1 and B2 flow through a residential area within the City of Belmont, with numerous land owners flanking either side of Belmont Creek. Lacking a unifying policy governing creekside development, bank armoring and channel modifications have been constructed piecemeal by land owners without any coordination. As such, numerous erosion control methods have been implemented throughout the watershed with varying degrees of success. Two methods that could guide future efforts include a comprehensive creekside development policy and establishment of a special assessment district.

7.3.1 Creekside Development Policy

A development policy for Belmont Creek would unify and coordinate construction efforts within and along the creek channel, provide approved revetment methods, guide the efforts of the special assessment district as described below, and ensure that the conservation of Belmont Creek is consistent with goals and visions of the various stakeholders. By establishing creekside development policies, water quality, hydrologic, biological, recreational, and aesthetic goals for Belmont Creek may be systematically pursued and enforced.

7.3.2 Special Assessment District

Assessment districts are financing mechanisms that enable cities, counties, and communities to designate specific areas as a district, which collects funds from residents within the district to finance public improvements for the district. Assessment districts are often created in rural or suburban communities with little resources to finance public improvements. For example, an assessment district could be established in a flood zone to finance flood control improvements for those residing in the flood zone.

For the purposes of this Project, a Belmont Creek special assessment district would include all real estate parcels adjacent to the creek channel or within flood zones associated with Belmont Creek. The Belmont Creek special assessment district would collect funds from those within the district to implement improvements consistent with the Creekside Development Policy. These improvements would benefit all within the district through reduced flooding, bank erosion prevention or repair, sediment removal, or creek restoration.

7.4 Photographic Documentation

Once the design plans are approved for the construction of the preferred alternative, it is recommended that long-term photo vantage points be established to provide monitoring and documentation of geomorphic changes in Belmont Creek. Photo points should be established upstream and downstream of as well as along the proposed project, and marked with a global positioning system (GPS) unit. The location and heading of the photo vantage point should be recorded, included on a map, and geo-referenced to provide consistent photo documentation.

7.5 Required Permits

Work on Belmont Creek would likely require coordination with the City of Belmont, Caltrans, the U.S. Army Corps of Engineers, the California Department of Fish and Wildlife (CDFW), and the RWQCB. Work on the Quarry Road and Taylor Way open channels may require coordination with the CDFW. Work on ditches adjacent to US 101 will require coordination with CDFW and Caltrans (BKF Engineers 1998).

8 REFERENCES

BioAssessment Services. (2007). *Biological Assessment of Belmont Creek and Comparison with Existing San Mateo County Data*.

BKF Engineers. (1998). *Harbor Industrial Area Storm Drain Report*.

California Wetlands Monitoring Workgroup (CWMW). EcoAtlas. <www.ecoatlas.org> (Last accessed: February 14, 2014).

Caltrans. (2011). *Bridge Inspection Report. Bridge Inspection Records Information System* (BIRIS).

Caltrans. (2012). *Caltrans Highway Design Manual,* Sixth Edition.

City of Belmont. (2004). Staff Report: Creek Maintenance and Easements Study Session.

Committee on Sea Level Rise in California, Oregon, and Washington. (2012). Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future. <http://www.nap.edu/catalog.php?record_id=13389>

Computational Hydraulics International. Spatial Decision Support for United States Environmental Protection Agency Storm Water Management Model (PCSWMM). < http://www.chiwater.com/Software/PCSWMM/index.asp>

Dewing, R.E. (1977). *Heritage of the Wooded Hills: A Belmont History*.

EOA, Inc. (2007). *Unified Stream Assessment in Six Watersheds in San Mateo County, California.* San Mateo Countywide Water Pollution Prevention Program.

EOA, Inc. and Kinnetic Laboratories Incorporated. *Joint Stormwater Agency Project to Study Urban Sources of Mercury, PCBs and Organochlorine Pesticides.*

FEMA. (2012). *Flood Insurance Study, San Mateo County, California and Incorporated Areas.* Numbers 06081CV001A and 06081CV001B.

Graymer, R.W., Moring, B.C., Saucedo, G.J., Wentworth, C.M., Brabb, E.E., and Knudsen, K.L. 2006. Geologic Map of the San Francisco Bay Region. USGS, California Geological Survey.

Kinnetic Laboratories Incorporated. *Belmont Creek Watershed Monitoring Report.*

Kondolf, G. Mathias. (1997). "Hungry Water: Effects of Dams and Gravel Mining on River Channels." *Environmental Management.* Vol. 21, No. 4: pp. 533-551

Lee, Joseph. (1865). "Ralston Hall and Its Grounds, San Mateo County." Retrieved from <http://www.museuma.com/joseph-lee/ralston-hall-and-its-grounds-san-mateo-county.html>

McKee, L.J. and Gilbreath, A.N., Hunt, J.A., Greenfield, B.K. Pollutants of concern (POC) loads monitoring data, water year (WY) 2011. Regional Monitoring Program for

Water Quality in San Francisco Bay (RMP), Small Tributaries Loading Strategy (STLS).

National Oceanic and Atmospheric Administration. Tides and Currents for Redwood City, CA. <http://tidesandcurrents.noaa.gov/waterlevels.html?id=9414523&units=standard&bdate=20140102&edate=20140104&timezone=GMT&datum=MLLW&interval=6&action=data> (Last accessed: March 26, 2014).

National Oceanic and Atmospheric Administration. Hydrometeorological Design Studies Center. NOAA ATLAS 14 Point Precipitation Frequency Estimates. <http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca>

Natural Resources Conservation Service. “Urban Hydrology for Small Watersheds.” *Technical Release No. 55.*

PRISM Climate Group, Oregon State University. (2004). <http://prism.oregonstate.edu>

Rosenberg, M. (February 15, 2011). *San Carlos businesses sue Belmont, San Mateo County for flood damage.* <http://www.mercurynews.com/ci_17397465> (Last accessed: February 25, 2014).

Rosgen, Dave. 1994. *Applied River Morphology*.

Regional Water Quality Control Board, San Francisco Bay Region. 2003. *Technical Reference Circular W.D. 02 - #1*. A Primer on Stream and River Protection for the Regulator and Program Manager.

San Mateo Countywide Stormwater Pollution Prevention Program (STOPPP). (2002). *Characterization of Imperviousness and Creek Channel Modifications for 17 Watersheds in San Mateo County.*

STOPP. *Trash Assessment in Six Watersheds.*

Schaaf and Wheeler. (2010). *Belmont Creek Hydraulic Analysis.*

Schaaf and Wheeler. (2007). *Santa Clara County Drainage Manual*.

United States Army Corps of Engineers Hydrologic Engineering Center. Hydrologic Modeling System (HEC-HMS). <http://www.hec.usace.army.mil/software/hec-hms/>

United States Army Corps of Engineers Hydrologic Engineering Center. Geospatial Hydrologic Modeling Extension (HEC-GeoHMS). <http://www.hec.usace.army.mil/software/hec-geohms/>

United States Department of Agriculture. Natural Resources Conservation Service. *Web Soil Survey*. <http://websoilsurvey.nrcs.usda.gov/app/> (Last Accessed: 2014).

United States Environmental Protection Agency. *Watershed Assessment, Tracking & Environmental Results.*

<http://iaspub.epa.gov/waters10/watershed_characterization.control?p_comid=2804731> (Last accessed: February 4th, 2014)

United States Geological Survey. 1899. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

United States Geological Survey. 1915. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

United States Geological Survey. 1939. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

United States Geological Survey. 1947. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

United States Geological Survey. 1956. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

United States Geological Survey. 1993. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

United States Geological Survey. 2012. California, San Mateo Quadrangle. <http://geonames.usgs.gov/apex/f?p=262:1:2495888065712> (Last accessed: February 26, 2014).

William Lettis & Associates and San Francisco Estuary Institute. (2007). Creek and Watershed Map of San Mateo County and Vicinity.

WRECO. (2012). *Dredging Assessment of Novartis Pharmaceuticals Corporation Site.* Technical Study Memorandum.

Appendix A Photo Documentation

Photo 5. Taken approximately 2,000 ft upstream of the Live Oak culvert at the low flow outfall of Water Dog Lake and taken from the prior photo location facing up the right slope toward the spillway pipes

There is little established vegetation along the top of the right bank. Erosion has been occurring at the down slope side of the spillway pipe. Water has begun to scour the lefts side (facing downstream) of the spillway pipe and formed a channel down the bank to the main channel.

Photo 6. Taken approximately 2,000 ft upstream of the Live Oak culvert at the low flow outfall of Water Dog Lake facing upstream, left bank, and downstream, respectively

The outfall structure has no velocity dissipation and the slope is relatively steep within the active channel. The channel is incised and undermining both the left and right bank with signs of prior bank failure. The banks are nearly vertical with no supporting vegetation along the bank slopes immediately adjacent to the active channel.

Photo 7. Taken approximately 1,760 ft upstream of the Live Oak culvert facing upstream and the right bank

The pipe is of Water Dog Lake spillway outfall. There is a concrete basin area and platform built as velocity dissipation. The right bank is partially supported with a geotextile material with rocks placed over it. Some of the geotextile is exposed. Upstream of the spillway outfall, the channel shows signs of becoming incised.

Photo 8. Taken approximately 1,500 ft upstream of the Live Oak culvert facing upstream

The channel is not incised, but there is erosion along the right bank. There is a floodplain area along the left bank that has moderate vegetation. Some debris is present including a rusted car and old pipes.

Photo 9. Taken approximately 1,320 ft upstream of the Live Oak culvert facing upstream and the right bank
The bankfull channel is not incised. There is a large floodplain area, compared to that of the downstream conditions, available. The area is lightly vegetated with mature trees present. There are large branches crossing the low-flow channel.

Photo 10. Taken approximately 1,250 ft upstream of the Live Oak culvert facing upstream
The channel and floodplain area open up at this location. The bankfull channel is not incised here and there is a sizable, available floodplain area. There are a number of young downed trees and vegetation debris present. The floodplain area is well vegetated.

Photo 11. Taken approximately 1,200 ft upstream of the Live Oak culvert facing upstream
The channel is incised and is undermining the right bank. Aggradation is occurring and forming a small scale point bar along the left bank. Trash and debris was present at this location and largely consisted of metal debris and rusted car parts.

Photo 12. Taken approximately 900 ft upstream of the Live Oak culvert facing upstream
The channel is less incised at this location than 100 ft downstream. There are a number of natural barriers formed at this location from large fallen branches, smaller branches, fallen leaves, and rocks of various graded rock sizes. Unnatural debris and trash is present at the blockage including a shopping cart and a tire.

Photo 13. Taken approximately 870 ft upstream of the Live Oak culvert facing upstream

The geometry of the native channel at this location has a more sustainable shape. The channel is slightly incised, but has a greater floodplain area created by aggradation at the point bar along the right bank of the meander. The right bank is well vegetated while the left bank is approaching vertical. There are a number of fallen branches and young trees spanning the flood water level of the channel.

Photo 14. Taken approximately 840 ft upstream of the Live Oak culvert facing the left bank and upstream, respectively

The channel is incised and the right bank is nearly vertical. The channel is undermining portions of the left bank. Downstream of the undermined location, there is evidence of prior left bank failure.

Photo 15. Taken approximately 520 ft upstream of the Live Oak culvert facing upstream
The native channel significantly incised at this location. The channel is approximately 5-6 feet deep and approximately three feet wide. There is also little meandering immediately upstream and downstream of this location.

Photo 16. Taken approximately 520 ft upstream of the Live Oak culvert facing the right bank and upstream, respectively
The channel is incised and the right bank is nearly vertical. The right bank appears to consist of a stratified rock layer. Sedimentation (aggradation) is occurring along the left bank.

Photo 17. Taken approximately 400 ft upstream of the Live Oak culvert facing upstream

The native channel has a natural obstruction of fallen branches and other vegetated debris. The obstruction slows some of the flow velocities of the channel. Aggradation is present upstream of the debris.

Photo 18. Taken approximately 300 ft upstream of the Live Oak culvert facing upstream.

The channel is incised approximately 4 to 5 ft with nearly vertical banks. Vegetation above the vertical banks is well established.


Photo 19. Taken at the upstream end of the Live Oak culvert facing downstream and upstream, respectively
The native channel upstream of the Live Oak culvert is heavily vegetated along the slope. The channel is not heavily incised.


Photo 20. Taken approximately 3,200 ft upstream of the culvert crossing at Chula Vista Dr. at the downstream side of Maywood Drive facing downstream
The channel is incised and a pool has formed at the outfall. The right bank is partially lined with sacked concrete.

Photo 21. Taken approximately 2,520 ft upstream of the culvert crossing at Chula Vista Drive facing the right bank from the left bank and upstream, respectively
The channel is incised and has reached a rock formation. Some private drain pipes outfall along the top of the bank without any velocity dissipation.

Photo 22. Taken approximately 2,470 ft upstream of the culvert crossing at Chula Vista Drive facing upstream
Rock barriers have been placed along the left bank. There are signs of scour at the downstream end of the rocks and channel has undermined the rocks. Upstream of the rocks, sacked concrete and a concrete apron have been placed. The concrete and sacked concrete are also being undermined. The right bank is nearly vertical.

Photo 23. Taken approximately 2,440 ft upstream of the culvert crossing at Chula Vista Drive facing upstream and the right bank
The channel has undermined the right bank to a height of approximately 6 ft.

Photo 24. Taken approximately 1,900 ft upstream of the culvert crossing at Chula Vista Drive facing the right bank

The right bank shows evidence of erosion and large tarps have been anchored to the slope. A segment has been hardened with concrete. The concrete segment has a vertical wall, a small platform and a concrete apron. The channel has undermined the concrete apron and the platform area. The concrete section has scour at the upstream side of the wall/platform. Concrete debris is also present within the low-flow channel.

Photo 25. Taken from approximately 1,860 ft upstream of the culvert crossing at Chula Vista Drive facing the left bank beginning upstream of the cut bank and moving downstream, respectively

The left bank has been hardened with shotcrete. The channel is undermining the hardened segment. Aggradation is occurring along the point bar.

Photo 26. Taken approximately 1,720 ft upstream of the culvert crossing at Chula Vista Drive facing upstream
The channel is narrow at this location due to the concrete walls constructed as part of the footing for the bridge supporting a private driveway. At the downstream end of the concrete walls, there are signs of scour around the extents of the concrete barrier.

Photo 27. Taken approximately 1,630 ft upstream of the culvert crossing at Chula Vista Drive facing the downstream, downstream right bank, and upstream right bank, respectively
The channel is incised with a nearly vertical right bank. A portion of the right bank has been hardened with sacked concrete, a partial concrete wall, and concrete apron. The channel is undermining the concrete apron and wall. Upstream of the bank hardening, the channel has undermined the vegetation and created a nearly vertical right bank. The undermining under the vegetation is over 6 ft in height.

Photo 28. Taken approximately 1,320 ft upstream of the culvert crossing at Chula Vista Drive facing the left bank starting upstream and proceeding downstream (left to right)
The left bank has been hardened. The channel is undermining the concrete bottom of the vertical wall. At the upstream side of the wall, sacked concrete has been placed at a culvert outfall. Concrete that was placed at the bottom of the sacked concrete is now being undermined by the channel. There is also some scouring at the downstream side of the sacked concrete.

Photo 29. Taken from approximately 1,180 ft to 1,120 ft upstream, respectively, of the culvert crossing at Chula Vista Drive facing the right bank
The channel is incised with a nearly vertical right bank. The channel is also undermining along the right bank. Concrete blocks and rock have been added at some of the undermined locations, which hardened the right bank.

Photo 30. Taken approximately 1,100 ft upstream of the culvert crossing at Chula Vista Drive facing upstream
The bottom of the channel has been hardened with concrete. Scour is occurring where the concrete ends on the downstream side.

Photo 31. Taken approximately 700 ft upstream of the culvert crossing at Chula Vista Drive facing upstream and downstream, respectively
The channel is incised. The right bank has been hardened with poured concrete. The channel has continued to incise and is undermining the poured concrete segment.

Photo 32. Taken approximately 870 ft upstream of the culvert crossing at Chula Vista Drive facing the left bank at the toe and top of bank, respectively
There is evidence of bank failure. Fabric has been placed along the slope with chain link fencing fastened over the top.

Photo 33. Taken approximately 700 ft upstream of the culvert crossing at Chula Vista Drive facing the upstream and the, respectively
The channel is incised. The right bank has been hardened with poured concrete and concrete debris with poured concrete. The channel is incised and is undermining the right bank below the poured concrete segments. There are also signs of high flows undermining the right bank above the poured concrete segments creating overhangs.

Photo 34. Taken approximately 600 ft upstream of the culvert crossing at Chula Vista Drive facing upstream and the left bank, respectively
The channel is incised. The left bank is nearly vertical and has been hardened with bricks. The channel is beginning to undermine the left bank at the downstream end of the hardened area.

Photo 35. Taken approximately 530 ft upstream of the culvert crossing at Chula Vista Drive facing the right bank and right bank/upstream, respectively
The channel is incised. The right bank is nearly vertical and has been hardened with bricks. The channel has continued to incise and is beginning to undermine the right bank at the downstream end of the hardened area.

Photo 36. Taken approximately 420 ft upstream of the culvert crossing at Chula Vista Drive facing downstream

The channel is incised and undermining along the cut bank. The left bank at the culvert consists of a vertical wall. The right bank at the beginning of the culvert consists of brick and poured concrete. The channel has been hardened along both banks and is evidence of the area having a history of scour and erosion problems.

Photo 37. Taken approximately 300 ft upstream of the culvert crossing at Chula Vista Drive facing downstream

The channel is incised. The left bank has been hardened with poured concrete. There is evidence of a small concrete step upstream of the next meander curve. The downstream cut bank has also been hardened with concrete.

Photo 38. Taken at the upstream end of the culvert crossing under Chula Vista Drive facing upstream, downstream, and the right bank at the culvert, respectively
The channel is incised and undermining along the cut bank. The left bank at the culvert consists of a vertical wall. The right bank at the beginning of the culvert consists of brick and poured concrete. The channel has been hardened along both banks and there is evidence of the area having a history of scour and erosion problems.

Photo 39. Taken downstream of the culvert outfall located east of 1301 Ralston Avenue, facing upstream, the left bank, and downstream, respectively
The channel is incised and undermining the banks below existing root systems. There is an approximately 3-ft drop from the flowline invert of the culvert and the flowline of the natural bottom of the channel. The bank slopes are very steep and nearing vertical.

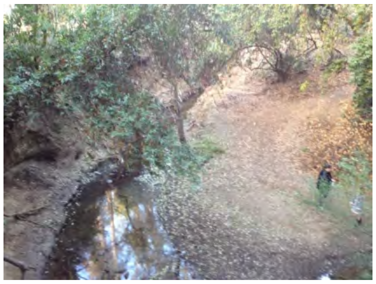

Photo 40. Taken at the meander immediately downstream of the pedestrian bridge in Twin Pines Park facing upstream, the left bank and downstream, respectively
At this location, the drop from the top of the right bank to the water surface level is approximately 20 ft. The right bank is very steep, prohibiting the establishment of vegetation and there is evidence of bank scour along the cut bank.


Photo 41. Taken upstream of 6th Avenue near the play structures in Twin Pines Park facing upstream and facing the left bank, respectively
The channel is incised. Sack-crete and a concrete apron are present along the left bank (shown on the right in the picture above). The channel is undermining the concrete apron along the cut bank and continuing to incise.


Photo 42. Taken in Belmont Creek facing downstream, approximately behind address 1298 O'Neill Avenue and at a tributary outfall at the top of the right bank, respectively
The channel is incised. Along the right bank, rock gabion baskets have been used to stabilize the bank. The channel has also begun undermining the gabion baskets. The alignment downstream of this location is largely straight. Upstream of this location, the creek begins to have some meanders.

Photo 43. Taken upstream of 6th Avenue adjacent to 1000 O'Neill Avenue, facing upstream
The channel is incised. There are signs of bank scour and failure along the right bank (facing downstream) shown on the left in the picture above. The channel is undermining the right bank below existing root ball systems.

Photo 44. Taken between immediately southwest of 6th Avenue facing upstream, at ta bend in the channel, and downstream at the reinforced concrete structure under 6th Avenue, respectively
The creek is incised with steep side slopes. As can be seen in the center photo, the left bank has been hardened in an attempt to stabilize the bank

Photo 45. Taken from the culvert under the Caltrain tracks facing upstream and from El Camino Real facing downstream, respectively
Both banks along this segment are lined with rock gabions. There is one outfall (seen in the photo on the left) that is suspended above the bottom of the channel with no velocity dissipation measures in place. There is a shallow ponded area forming below the outfall pipe where erosion has occurred.


Photo 46. Taken between Old County Road and the Caltrain tracks facing upstream and downstream, respectively
The creek has a straight alignment. There are three segments where rip-rap and/or sack-crete were installed along with poured concrete. The channel has steep side slopes and the channel is incised.


Photo 47. Taken at the downstream end of the culvert crossing Old County Road
At the downstream end of the culvert, the channel has been hardened with concrete via shotcrete, sack-crete, and framed, tiered sections. At the downstream end of the concrete wall, there is a concrete drop structure.

Photo 48. Taken from the 90 degree bend in the creek at Old County Road facing upstream, downstream at the bend, and downstream past the bend (northeast), respectively
The creek has a fairly straight alignment. Pieces of concrete blocks and slabs have been added at the outer (southern) bank of the creek.

Photo 49. Taken approximately 250 feet northeast of Old County Road facing upstream and downstream, respectively
The channel shows signs of erosion and scour along the southeastern bank at the downstream crossing of the Industrial Road bridge.

Photo 50. Taken from the Novartis property southwest and upstream of Industrial Road
This photo was taken at high tide, which was recorded to be approximately 9.2 feet at Redwood City at the time the photo was taken (NOAA Tides & Currents database). The geometry of the channel largely consists of a straight alignment between Industrial Road and Old County Road, at which point the creek makes an approximately 90 degree turn to run parallel along Old County Road.

Photo 51. Taken northeast of Industrial Road facing east toward Industrial Road
The channel shows signs of erosion and scour along the southeastern bank at the downstream crossing of the Industrial Road bridge.

Photo 52. Taken southwest of US 101 facing US 101: Inundated Cross Culvert under US 101

During a high tide event on January 3rd, 2014, the cross culvert crossing under US 101 was observed to be inundated with tidal water reaching the crown of the culvert. This photo was taken at high tide, which was recorded to be approximately 9.2 feet at Redwood City at the time the photo was taken (NOAA Tides & Currents database).

Appendix B Watershed Spatial Data

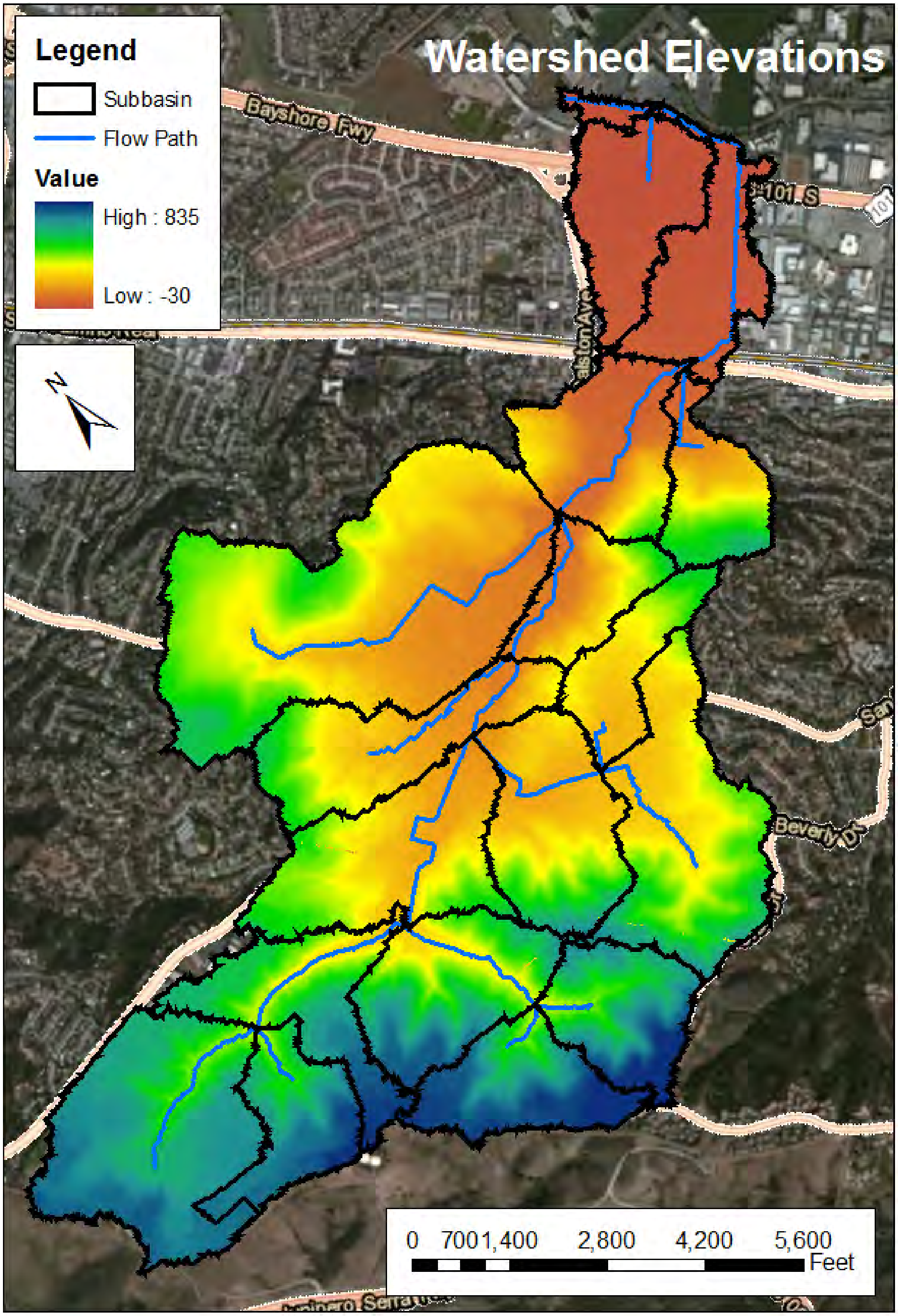

Figure 56. Elevations within the Belmont Creek Watershed

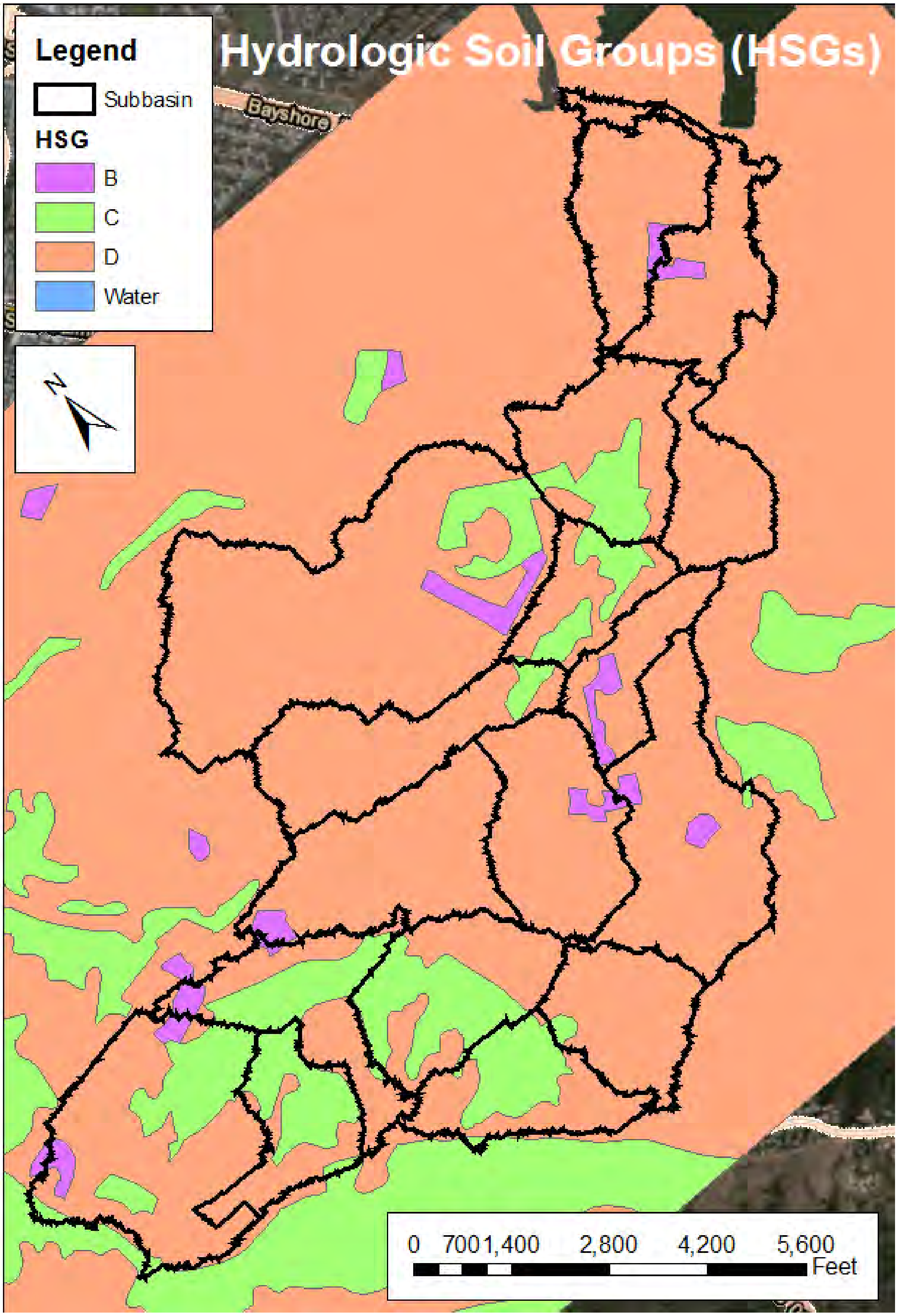

Figure 57. HSGs within the Belmont Creek Watershed

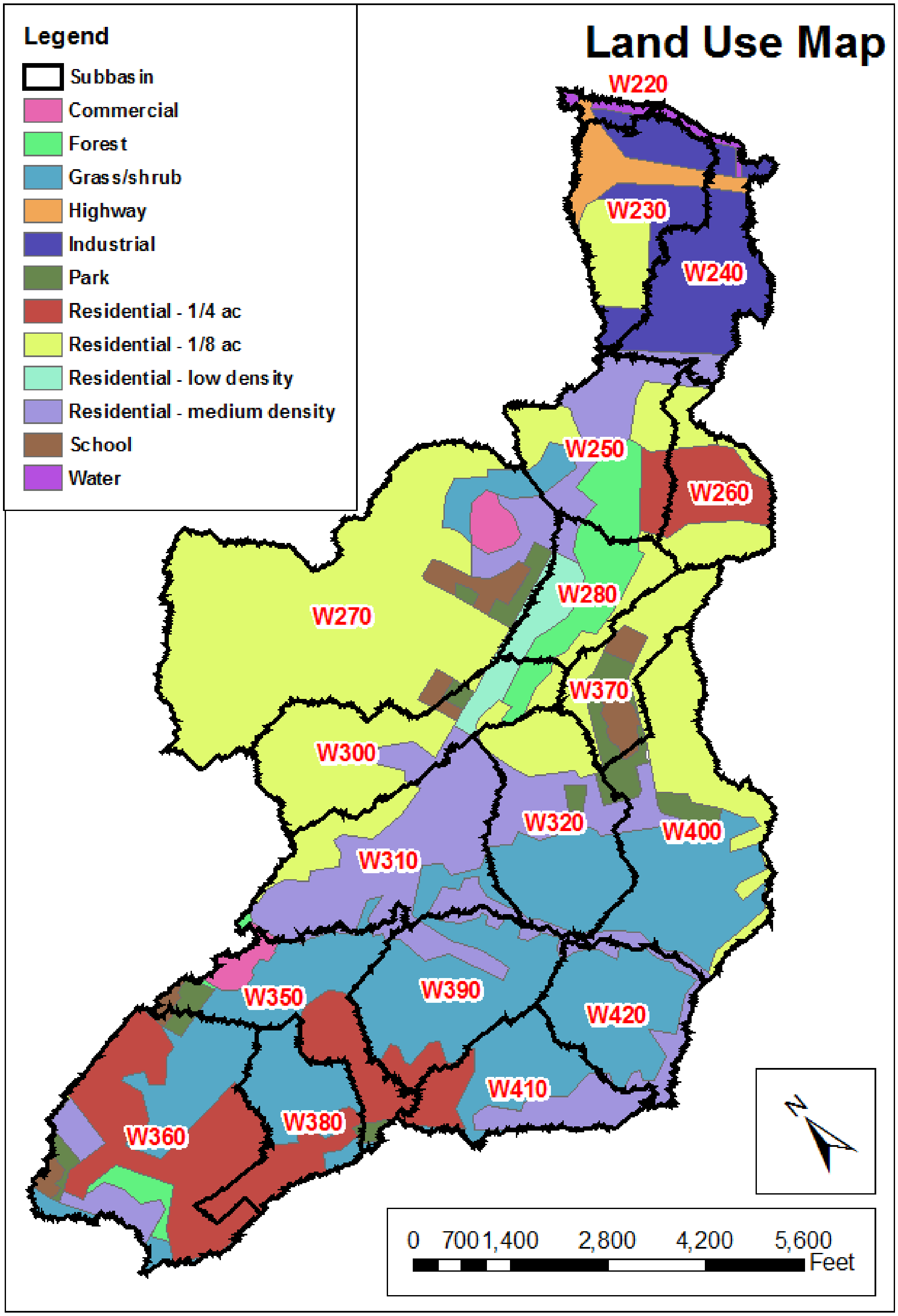

Figure 58. Land Use within the Belmont Creek Watershed

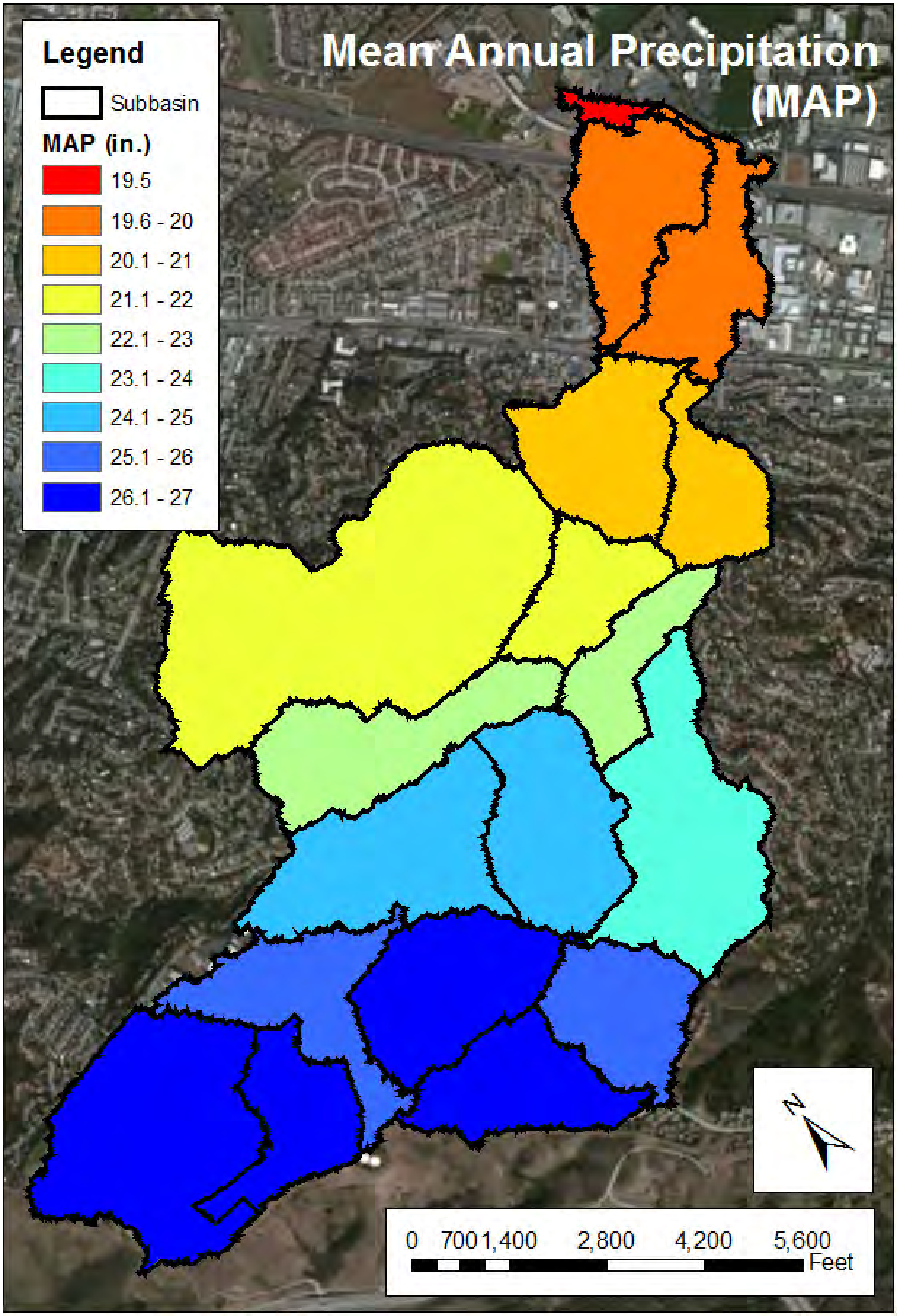

Figure 59. Mean Annual Precipitation within the Belmont Creek Watershed

Appendix C Watershed Map

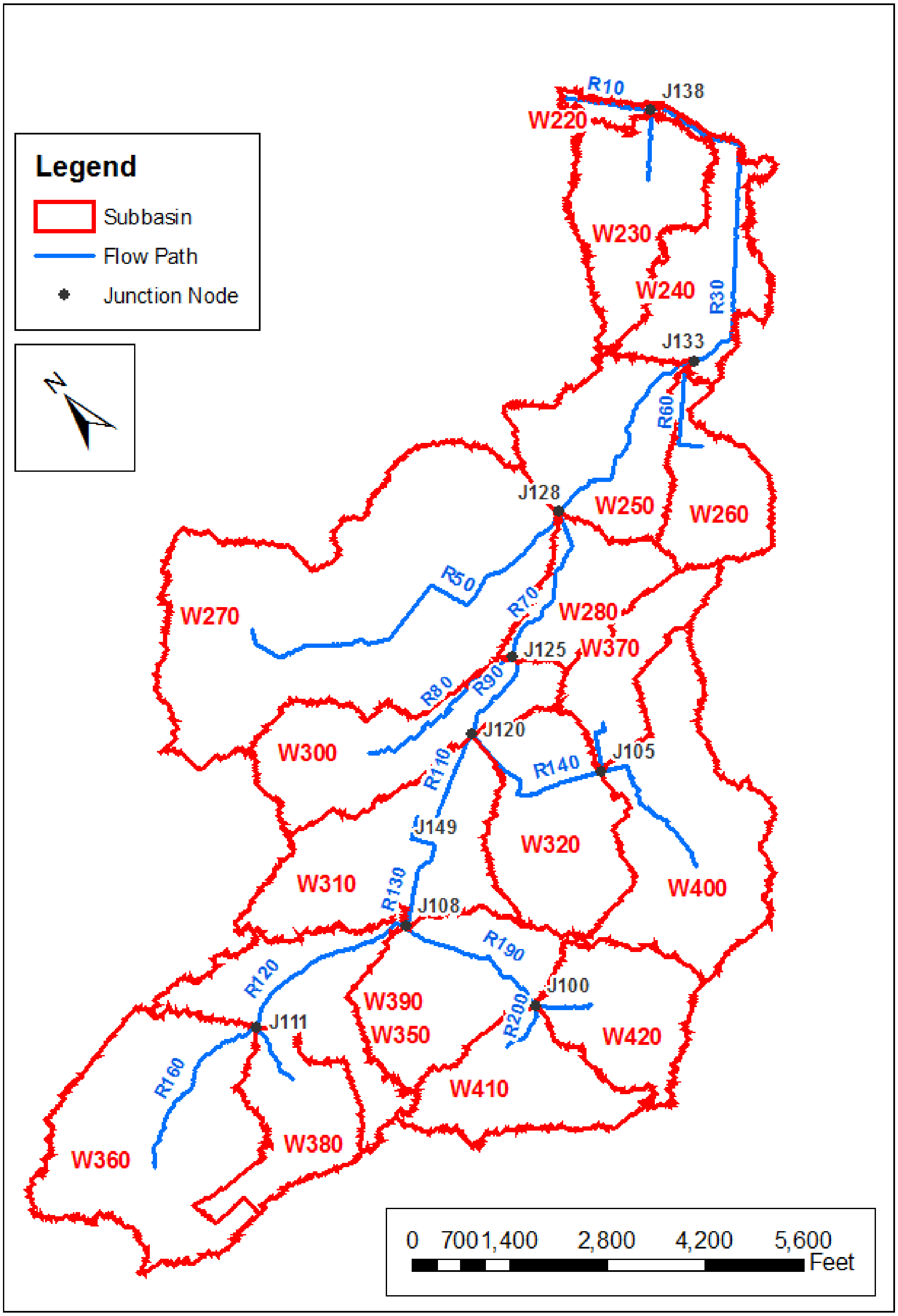

Figure 60. Belmont Creek Watershed in the Hydrologic Model

Appendix D Calculations and HEC-HMS Simulation Results

Basin Model "BelmontCreek1"

Subbasin	Area	SCS CN	SCS Lag
W360	0.30194		40.255
W380	0.11414		59.897
W410	0.13250		27.637
W420	0.13013		32.605
W390	0.17681		15.520
W350	0.15529		22.846
W400	0.25955		19.004
W370	0.0936064		14.475
W310	0.22135		20.986
W320	0.18382		16.737
W300	0.17773		15.758
W270	0.56575		32.547
W280	0.10040		13.527
W250	0.16380		16.254
W260	0.10267		17.291
W230	0.15086		15.759
W240	0.14455		29.036
W220	0.0148225		5.7813

Current as of 26 August 2014 at 19:08:27

Figure 61. Basin Model SCS Lag Summary

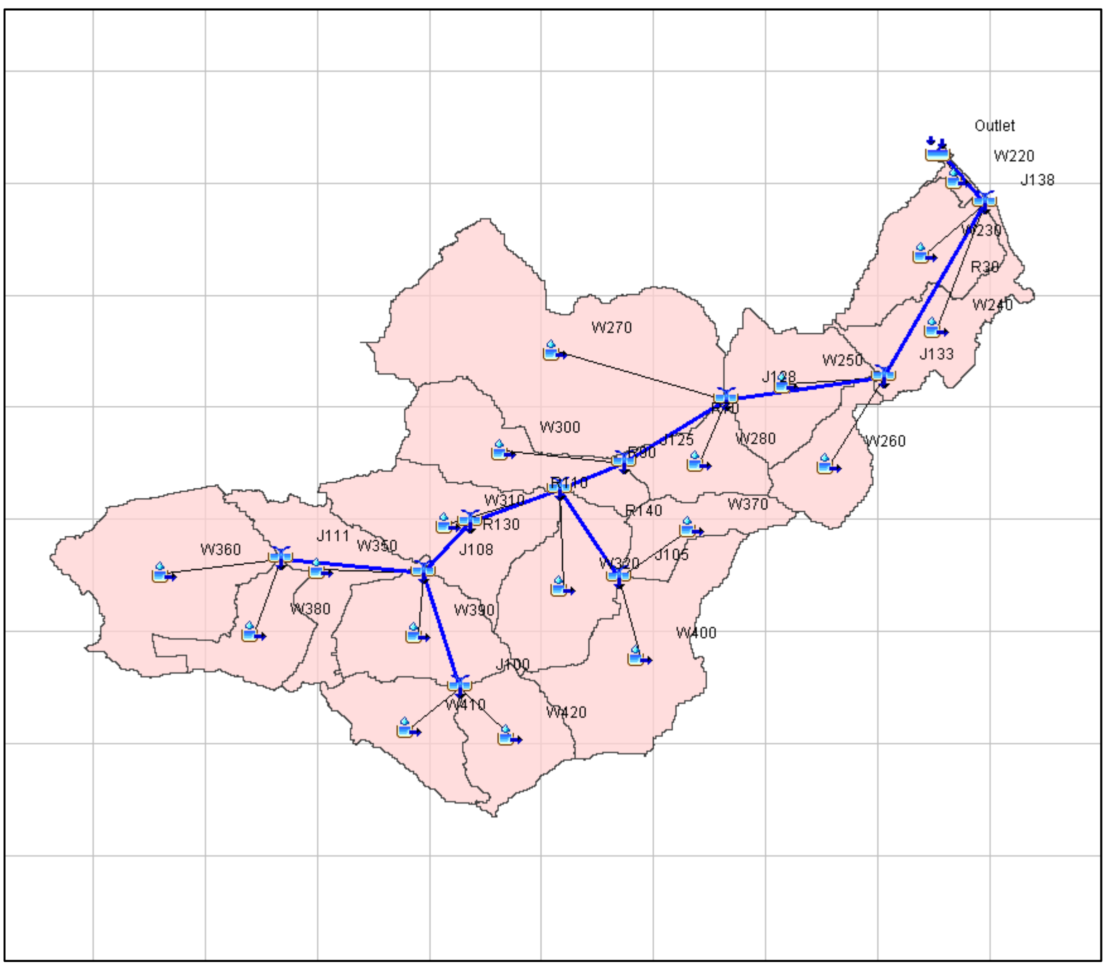

Figure 62. Basin Model

Project: BelmontCreek1 Simulation Run: 10-yr

Start of Run: 03Jan2008, 20:00 Basin Model: BelmontCreek1
End of Run: 04Jan2008, 20:00 Meteorologic Model: 2008 Jan 4 10-yr
Compute Time: 26Aug2014, 12:08:30 Control Specifications: 2008 Jan 4

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
W360	0.3019400	71.4	04Jan2008, 11:16	2.82
W380	0.1141400	23.7	04Jan2008, 11:26	2.64
J111	0.4160800	94.8	04Jan2008, 11:17	2.77
R120	0.4160800	94.8	04Jan2008, 11:21	2.77
W410	0.1325000	34.0	04Jan2008, 11:06	2.76
W420	0.1301300	35.9	04Jan2008, 11:10	3.17
J100	0.2626300	69.7	04Jan2008, 11:08	2.96
R190	0.2626300	69.6	04Jan2008, 11:12	2.96
W390	0.1768100	55.3	04Jan2008, 13:30	2.70
W350	0.1552900	41.0	04Jan2008, 11:02	2.64
J108	1.0108100	239.0	04Jan2008, 11:02	2.79
R130	1.0108100	238.9	04Jan2008, 11:05	2.79
J149	1.0108100	238.9	04Jan2008, 11:05	2.79
R110	1.0108100	238.8	04Jan2008, 11:08	2.79
W400	0.2595500	82.1	04Jan2008, 13:32	3.05
W370	0.0936064	31.1	04Jan2008, 13:29	2.82
J105	0.3531564	112.5	04Jan2008, 13:31	2.99
R140	0.3531564	112.4	04Jan2008, 13:35	2.99
W310	0.2213500	68.9	04Jan2008, 11:00	3.11
W320	0.1838200	61.3	04Jan2008, 13:30	3.05
J120	1.7691364	469.2	04Jan2008, 11:02	2.90
R90	1.7691364	468.9	04Jan2008, 11:04	2.90
W300	0.1777300	61.6	04Jan2008, 13:30	3.11
J125	1.9468664	521.1	04Jan2008, 11:02	2.92
R70	1.9468664	520.0	04Jan2008, 11:07	2.92
W270	0.5657500	150.9	04Jan2008, 11:10	3.05
W280	0.1004000	34.3	04Jan2008, 13:28	2.82
J128	2.6130164	691.7	04Jan2008, 11:07	2.94
R40	2.6130164	690.5	04Jan2008, 11:12	2.94
W250	0.1638000	53.6	04Jan2008, 13:30	2.94
W260	0.1026700	34.8	04Jan2008, 13:30	3.17
J133	2.8794864	751.1	04Jan2008, 11:10	2.95
R30	2.8794864	746.7	04Jan2008, 11:20	2.95
W230	0.1508600	51.5	04Jan2008, 13:30	3.05
W240	0.1445500	39.6	04Jan2008, 10:15	2.67
J138	3.1748964	808.5	04Jan2008, 11:19	2.94
R10	3.1748964	786.4	04Jan2008, 11:22	2.93
W220	0.0148225	7.3	04Jan2008, 13:23	3.21
Outlet	3.1897189	789.5	04Jan2008, 11:22	2.93

Figure 63. Result Summary, 10-Year Storm Event

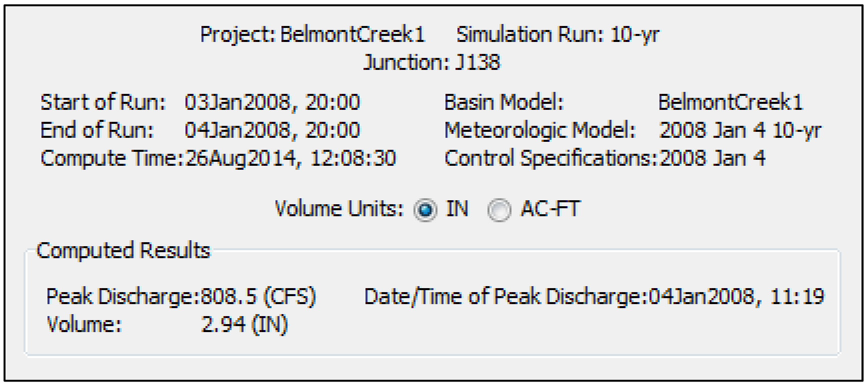

Figure 64. Result Summary J138, 10-Year Storm Event

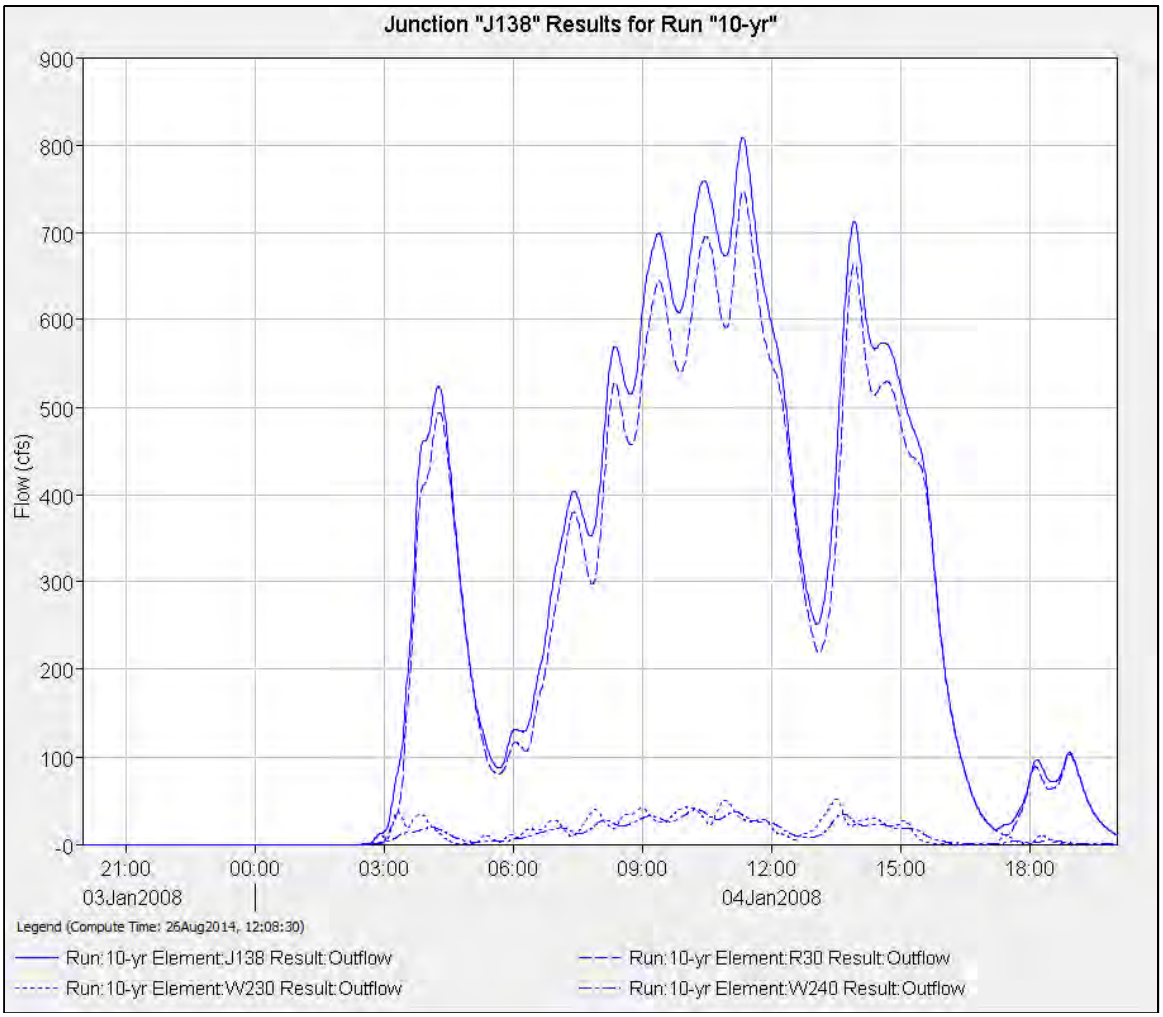

Figure 65. Outflow Hydrograph J138, 10-Year Storm Event

Project: BelmontCreek1 Simulation Run: SCC 25-yr

Start of Run: 30Dec1899, 00:00 Basin Model: BelmontCreek1
End of Run: 30Dec1899, 23:55 Meteorologic Model: 1899 Dec 30 025-yr
Compute Time: 26Aug2014, 12:08:38 Control Specifications: 1899 Dec 30

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
W360	0.3019400	158.2	30Dec1899, 06:50	2.05
W380	0.1141400	46.8	30Dec1899, 07:14	1.66
J111	0.4160800	198.7	30Dec1899, 06:55	1.95
R120	0.4160800	198.7	30Dec1899, 06:58	1.95
W410	0.1325000	83.6	30Dec1899, 06:34	1.93
W420	0.1301300	81.0	30Dec1899, 06:40	3.05
J100	0.2626300	163.1	30Dec1899, 06:36	2.48
R190	0.2626300	163.0	30Dec1899, 06:40	2.48
W390	0.1768100	153.7	30Dec1899, 06:18	1.80
W350	0.1552900	106.6	30Dec1899, 06:27	1.67
J108	1.0108100	503.8	30Dec1899, 06:38	2.02
R130	1.0108100	503.7	30Dec1899, 06:40	2.02
J149	1.0108100	503.7	30Dec1899, 06:40	2.02
R110	1.0108100	503.6	30Dec1899, 06:43	2.02
W400	0.2595500	210.0	30Dec1899, 06:22	2.68
W370	0.0936064	86.0	30Dec1899, 06:16	2.06
J105	0.3531564	289.2	30Dec1899, 06:20	2.51
R140	0.3531564	288.7	30Dec1899, 06:24	2.51
W310	0.2213500	171.0	30Dec1899, 06:25	2.86
W320	0.1838200	159.8	30Dec1899, 06:19	2.68
J120	1.7691364	1024.9	30Dec1899, 06:27	2.29
R90	1.7691364	1024.1	30Dec1899, 06:29	2.29
W300	0.1777300	161.1	30Dec1899, 06:18	2.87
J125	1.9468664	1152.1	30Dec1899, 06:27	2.34
R70	1.9468664	1149.5	30Dec1899, 06:31	2.34
W270	0.5657500	345.3	30Dec1899, 06:40	2.67
W280	0.1004000	96.1	30Dec1899, 06:15	2.06
J128	2.6130164	1532.2	30Dec1899, 06:32	2.40
R40	2.6130164	1529.5	30Dec1899, 06:37	2.40
W250	0.1638000	142.7	30Dec1899, 06:19	2.32
W260	0.1026700	89.0	30Dec1899, 06:20	3.06
J133	2.8794864	1685.3	30Dec1899, 06:35	2.42
R30	2.8794864	1673.6	30Dec1899, 06:44	2.41
W230	0.1508600	135.8	30Dec1899, 06:18	2.68
W240	0.1445500	94.4	30Dec1899, 06:36	2.86
J138	3.1748964	1836.4	30Dec1899, 06:43	2.45
R10	3.1748964	1690.5	30Dec1899, 06:49	2.43
W220	0.0148225	22.4	30Dec1899, 06:07	3.86
Outlet	3.1897189	1696.7	30Dec1899, 06:49	2.44

Figure 66. Result Summary, 25-Year Storm Event

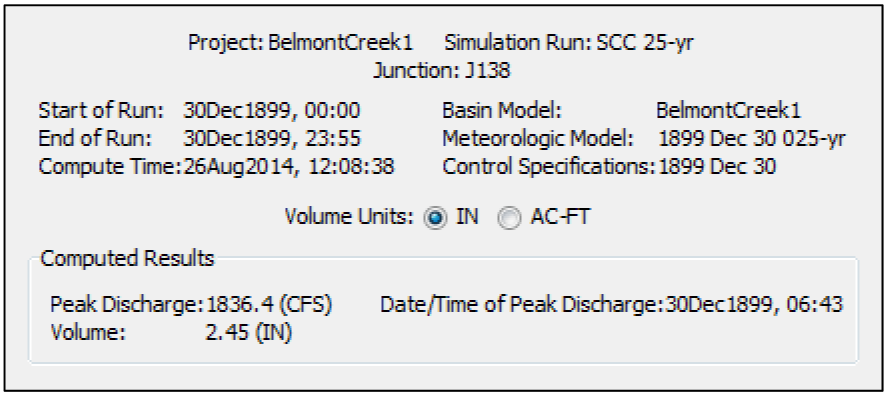

Figure 67. Result Summary J138, 25-Year Storm Event

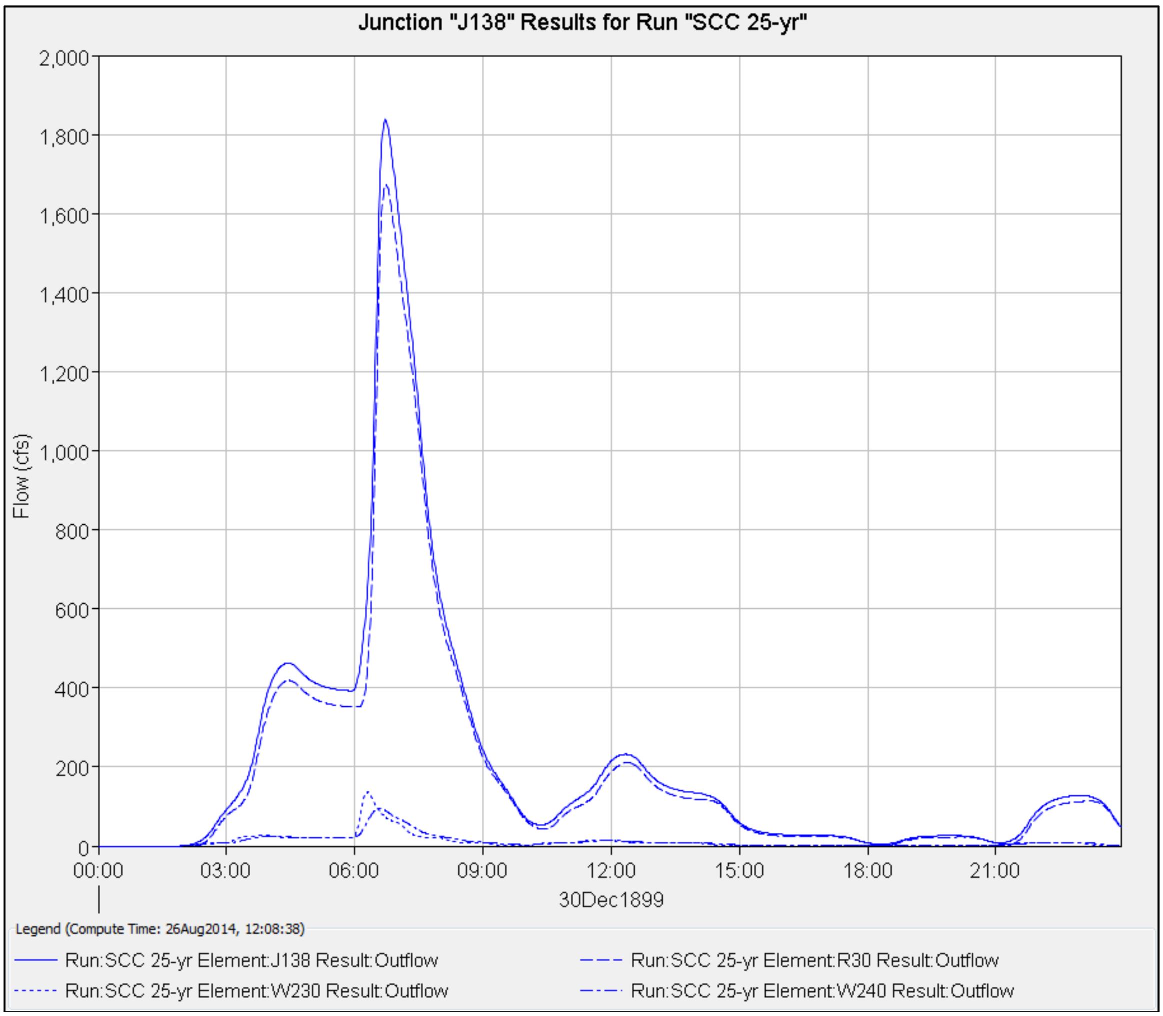

Figure 68. Outflow Hydrograph J138, 25-Year Storm Event

Project: BelmontCreek1 Simulation Run: SCC 100-yr

Start of Run: 30Dec1899, 00:00 Basin Model: BelmontCreek1
End of Run: 30Dec1899, 23:55 Meteorologic Model: 1899 Dec 30 100-yr
Compute Time: 26Aug2014, 12:08:44 Control Specifications: 1899 Dec 30

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
W360	0.3019400	203.1	30Dec1899, 06:50	3.05
W380	0.1141400	61.2	30Dec1899, 07:14	2.61
J111	0.4160800	256.5	30Dec1899, 06:55	2.93
R120	0.4160800	256.5	30Dec1899, 06:58	2.92
W410	0.1325000	106.9	30Dec1899, 06:34	2.89
W420	0.1301300	102.2	30Dec1899, 06:40	4.19
J100	0.2626300	207.2	30Dec1899, 06:36	3.53
R190	0.2626300	207.2	30Dec1899, 06:40	3.53
W390	0.1768100	195.1	30Dec1899, 06:18	2.76
W350	0.1552900	136.5	30Dec1899, 06:27	2.63
J108	1.0108100	650.2	30Dec1899, 06:38	3.01
R130	1.0108100	650.1	30Dec1899, 06:40	3.01
J149	1.0108100	650.1	30Dec1899, 06:40	3.01
R110	1.0108100	650.0	30Dec1899, 06:42	3.00
W400	0.2595500	264.7	30Dec1899, 06:22	3.82
W370	0.0936064	108.8	30Dec1899, 06:16	3.06
J105	0.3531564	365.0	30Dec1899, 06:20	3.62
R140	0.3531564	364.6	30Dec1899, 06:23	3.62
W310	0.2213500	215.4	30Dec1899, 06:25	4.01
W320	0.1838200	201.2	30Dec1899, 06:19	3.82
J120	1.7691364	1314.2	30Dec1899, 06:27	3.34
R90	1.7691364	1313.2	30Dec1899, 06:28	3.34
W300	0.1777300	202.5	30Dec1899, 06:18	4.01
J125	1.9468664	1476.1	30Dec1899, 06:27	3.40
R70	1.9468664	1472.7	30Dec1899, 06:30	3.39
W270	0.5657500	437.6	30Dec1899, 06:40	3.81
W280	0.1004000	121.4	30Dec1899, 06:15	3.06
J128	2.6130164	1957.0	30Dec1899, 06:32	3.47
R40	2.6130164	1954.2	30Dec1899, 06:36	3.47
W250	0.1638000	180.2	30Dec1899, 06:19	3.44
W260	0.1026700	111.7	30Dec1899, 06:20	4.20
J133	2.8794864	2154.7	30Dec1899, 06:35	3.49
R30	2.8794864	2140.8	30Dec1899, 06:42	3.48
W230	0.1508600	170.9	30Dec1899, 06:18	3.82
W240	0.1445500	119.2	30Dec1899, 06:36	4.00
J138	3.1748964	2351.5	30Dec1899, 06:42	3.52
R10	3.1748964	2176.3	30Dec1899, 06:47	3.50
W220	0.0148225	27.8	30Dec1899, 06:07	5.06
Outlet	3.1897189	2184.1	30Dec1899, 06:47	3.51

Figure 69. Result Summary, 100-Year Storm Event

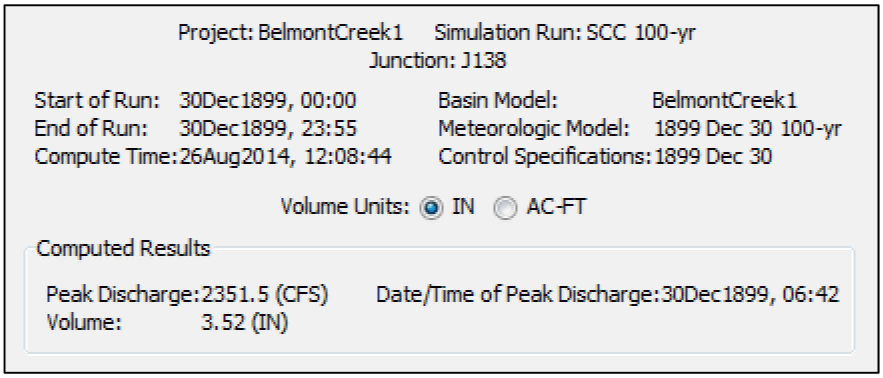

Figure 70. Result Summary J138, 100-Year Storm Event

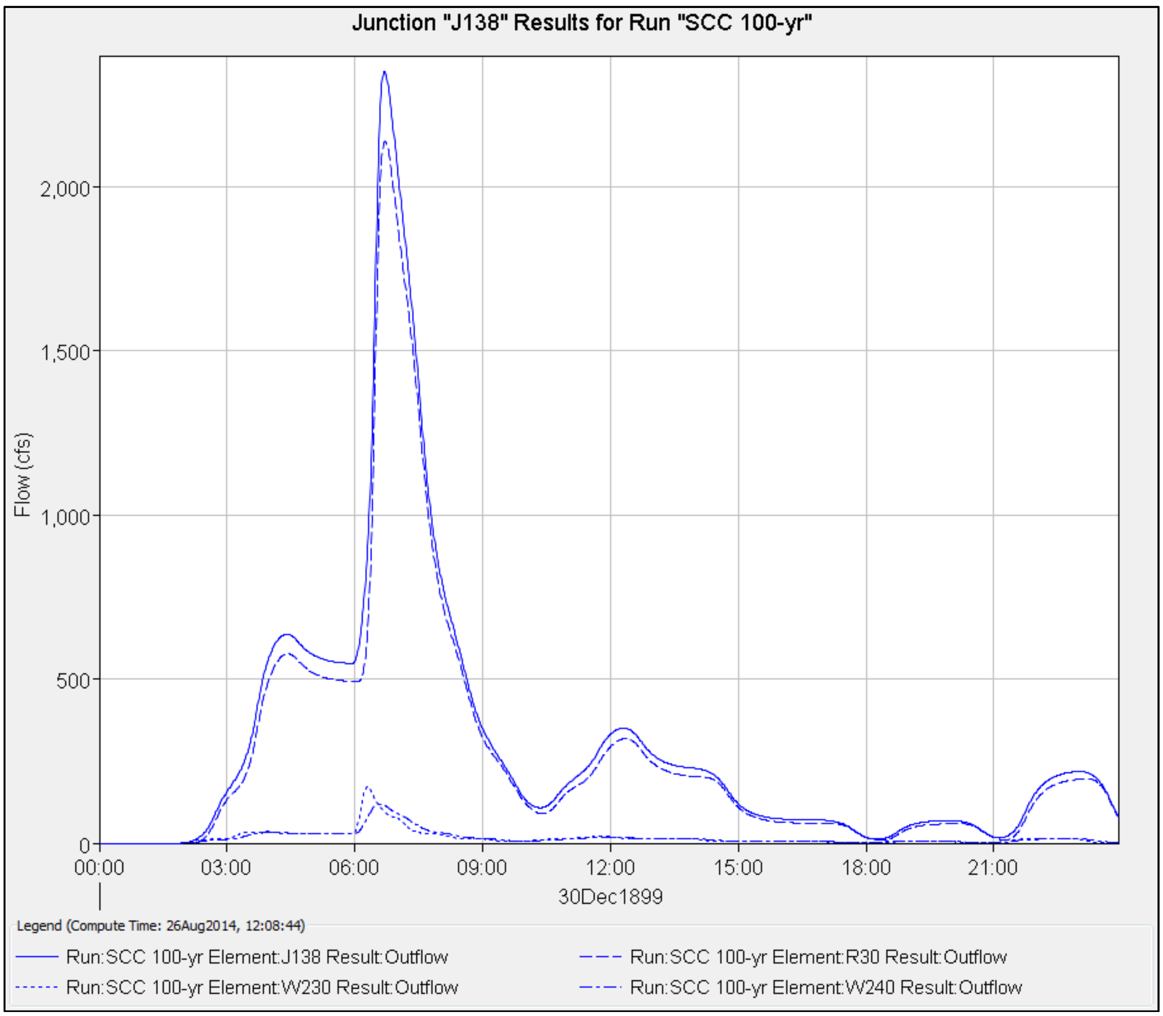

Figure 71. Outflow Hydrograph J138, 100-Year Storm Event

Appendix E PCSWMM Simulation Results

Floodplain Expansion+FW-Q10 Simulation

Existing Condition Belmont Creek
Revised nodes to remove duplicate storage in channels 07/14/2014
10-year storm against tidal curve w/MHHW

****
NOTE: The summary statistics displayed in this report are based on results found at every computational time step, not just on results from each reporting time step.
****

****
Analysis Options
****
Flow Units .... CFS
Process Models:
  Rainfall/Runoff .... NO
  Snowmelt .... NO
  Groundwater .... NO
  Flow Routing .... YES
  Ponding Allowed .... YES
  Water Quality .... NO
Flow Routing Method ...... DYNWAVE
Starting Date FEB-28-2014 00:00:00
Ending Date FEB-28-2014 23:55:00
Antecedent Dry Days 0.0
Report Time Step 00:01:00
Routing Time Step .... 0.50 sec

**************************	Volume	Volume
Flow Routing Continuity	acre-feet	10^6 gal
**************************	---------	---------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	270.742	88.225
External Outflow	265.513	86.521
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.001	0.000
Final Stored Volume	5.213	1.699
Continuity Error (%)	0.006	

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CFS	Max. Flow CFS	Total Volume 10^6 gal
J1655.601	91.15	147.92	980.59	86.832
System	91.15	147.92	980.59	86.832

Floodplain Expansion_1D Simulation

Existing Condition Belmont Creek
Revised nodes to remove duplicate storage in channels 07/14/2014
10-year storm against tidal curve w/MHHW

****
NOTE: The summary statistics displayed in this report are based on results found at every computational time step, not just on results from each reporting time step.
****

****
Analysis Options
****
Flow Units .... CFS
Process Models:
 Rainfall/Runoff .... NO
 Snowmelt .... NO
 Groundwater .... NO
 Flow Routing .... YES
 Ponding Allowed .... YES
 Water Quality .... NO
Flow Routing Method ...... DYNWAVE
Starting Date FEB-28-2014 00:00:00
Ending Date FEB-28-2014 23:55:00
Antecedent Dry Days 0.0
Report Time Step 00:01:00
Routing Time Step .... 0.50 sec

**************************	Volume	Volume
Flow Routing Continuity	acre-feet	10^6 gal
**************************	---------	---------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	282.104	91.928
External Outflow	275.099	89.645
Internal Outflow	0.021	0.007
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	4.387	1.430
Continuity Error (%)	0.921	

FP_Expansion_1D Simulation cont.

```
***********************
Outfall Loading Summary
***********************
 -----------------------------------------------------------
                Flow      Avg.      Max.      Total
                Freq.     Flow      Flow      Volume
Outfall Node        Pcnt.      CFS       CFS    10^6 gal
-----------------------------------------------------------
J1655.601          91.15    153.34    923.55      90.011
O_2                 0.00      0.00      0.00       0.000
z1012               0.00      0.00      0.00       0.000
-----------------------------------------------------------
System             30.38    153.34    923.55      90.011
```

Solution+floodplain

Solution Belmont Creek
Bypass down Belmont Creek, various walls, Y junction at Old County
Revised nodes to remove duplicate storage in channels 07/14/2014

****
NOTE: The summary statistics displayed in this report are based on results found at every computational time step, not just on results from each reporting time step.
****

****
Analysis Options
****
Flow Units .... CFS
Process Models:
  Rainfall/Runoff .... NO
  Snowmelt .... NO
  Groundwater .... NO
  Flow Routing .... YES
  Ponding Allowed .... YES
  Water Quality .... NO
Flow Routing Method ...... DYNWAVE
Starting Date FEB-28-2014 00:00:00
Ending Date FEB-28-2014 23:55:00
Antecedent Dry Days 0.0
Report Time Step 00:01:00
Routing Time Step .... 1.00 sec

************************** Flow Routing Continuity **************************	Volume acre-feet	Volume 10^6 gal
	---------	---------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	282.152	91.944
External Outflow	276.961	90.252
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	5.002	1.630
Continuity Error (%)	0.067	

Solution+floodplain cont.

```
***********************
Outfall Loading Summary
***********************

-----------------------------------------------------------
                Flow      Avg.     Max.      Total
                Freq.     Flow     Flow     Volume
Outfall Node        Pcnt.      CFS      CFS   10^6 gal
-----------------------------------------------------------
J1655.601          94.27    188.98   981.42     90.634
-----------------------------------------------------------
System             94.27    188.98   981.42     90.634
```

Solution+floodplain+SLR

Solution Belmont Creek
Bypass down Belmont Creek, various walls, Y junction at Old County
Revised nodes to remove duplicate storage in channels 07/14/2014

****
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
****

****
Analysis Options
****
Flow Units .... CFS
Process Models:
  Rainfall/Runoff .... NO
  Snowmelt .... NO
  Groundwater .... NO
  Flow Routing .... YES
  Ponding Allowed .... YES
  Water Quality .... NO
Flow Routing Method ...... DYNWAVE
Starting Date FEB-28-2014 00:00:00
Ending Date FEB-28-2014 23:55:00
Antecedent Dry Days 0.0
Report Time Step 00:01:00
Routing Time Step .... 1.00 sec

************************	Volume	Volume
Flow Routing Continuity	acre-feet	10^6 gal
************************	---------	---------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	284.493	92.706
External Outflow	279.222	90.989
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	5.003	1.630
Continuity Error (%)	0.094	

Solution+floodplain+SLR cont.

```
**********************
Outfall Loading Summary
**********************

-----------------------------------------------------------
                 Flow      Avg.     Max.      Total
                 Freq.     Flow     Flow      Volume
Outfall Node        Pcnt.      CFS      CFS   10^6 gal
-----------------------------------------------------------
J1655.601          94.27    190.64   987.25     92.134
-----------------------------------------------------------
System             94.27    190.64   987.25     92.134
```