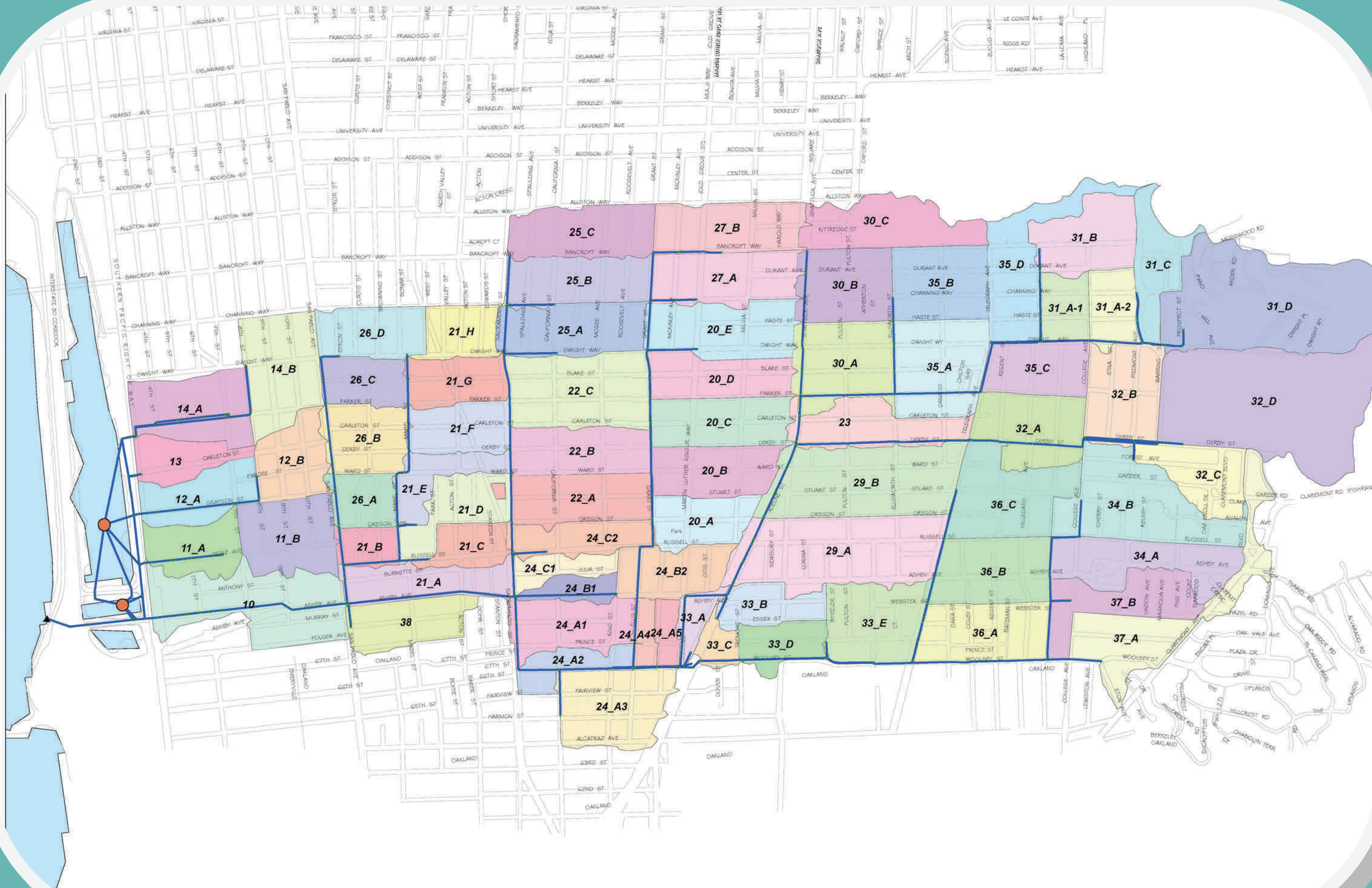

DRAFT POTTER AND CODORNICES WATERSHEDS HYDROLOGY AND HYDRAULCS REPORT

DRAFT ~ July 26, 2011

Consultant Report Prepared by:

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EXECUTIVE SUMMARY

--- To be developed when all review completed ---

1 INTRODUCTION

1.1 Purpose of this Report

The City of Berkeley is in the process of developing a watershed management plan (WMP) for the drainages within its boundaries.

A key component of the assessments and analyses underpinning the WMP is a thorough understanding of the hydrology and hydraulics of the respective drainages. Therefore, the City commissioned a detailed hydrology and hydraulics study for the Potter and Codornices watersheds. These watersheds were selected based on a number of criteria that indicate they are generally representative of the range of conditions that exist within the City. Therefore, detailed study of these two watersheds can provide information and insights that, with proper consideration, can be extrapolated to the remaining watersheds.

This report summarizes the objectives, technical analyses, and results of the hydrology and hydraulics study. It is intended to serve as a guidance document for watershed planning by identifying existing constraints as well as any retrofits and upgrades that maximize benefits to the community as a whole. This report also serves as the primary technical reference for the detailed hydrodynamic models of the two watersheds that were completed as part of the study. These models will be archived, maintained, and updated by the City as integral tools in the overall watershed management program.

1.2 Objectives and Approach of the Hydrology and Hydraulics Study

The objectives of the hydrology and hydraulics study were:

1. Utilize the City's geographic information system (GIS) database of utility infrastructure, land use, and topography to compile a data set to support modeling of the respective watersheds. Where appropriate, identify data gaps and acquire supplementary information.

2. Use the assembled data to construct non-steady state hydrodynamic models of the Codornices and Potter watersheds. The structure of the models must be such that they can be used to assess runoff volumes and rates for a wide range of meteorological inputs including extreme and design storm events and long-term continuous simulations based on historical data.

3. Carry out simulations using the hydrodynamic models to characterize existing conditions in the two watersheds with respect to stormwater conveyance for large design storms. These simulations quantify existing conveyance capacity and identify constraints contributing to observed and predicted localized flooding problems.

4. Use the hydrodynamic models to identify the size and location of infrastructure improvements necessary to meet the 10-year design storm performance criteria

through increased conveyance capacity and evaluate main trunklines or the 25-year design storm. This approach, defined herein as the "traditional retrofit" approach, would rely solely on increasing pipe sizes to reduce flooding risks. The anticipated capital costs for the required traditional retrofits would be compiled as well.

5. Extend the hydrodynamic modeling efforts to assess the potential for meeting stormwater conveyance goals through the inclusion of green infrastructure approaches such as rainwater harvesting (such as rain barrels, cisterns), infiltration (such as permeable pavements), and bio-retention areas (such as rain gardens) that provide important corollary watershed management benefits. Identify the size and location of appropriate green infrastructure improvements and compile anticipated capital costs estimates for their construction.

Two aspects of the objectives enumerated above are interrelated and distinguish the study from previous planning efforts. The first of these is the extent of the storm water conveyance networks modeled and scale of the watershed sub-catchments. The decision to include conveyance lines down to 18-inch diameter allows for delineation and simulation of watershed sub-catchments at a small scale (several city blocks) where differences in land use and neighborhood-level variations in stormwater management techniques can be assessed. This is directly related to the second distinguishing aspect of the study, namely using the respective watershed models to assess the potential efficacy of green infrastructure stormwater management approaches to reduce flood risks by controlling the volume and rate of runoff, either limiting or reducing the need for traditional retrofit improvements that are designed simply to move runoff to the Bay as quickly as possible.

1.3 Data Collection

Information collected for the study was obtained from available sources for soils, regional rainfall intensity mapping, and historical tide records. Additional information sources included the Storm Drainage Master Plan prepared by CH2M Hill in 1994, the Alameda County Hydrology Manual, and the City's GIS database.

To augment the data available for the current study, a monitoring network was installed to measure rainfall and flow at critical locations within the watersheds. These efforts included the installation of two flow gauges in the Potter watershed, one at a local runoff scale at the intersection of Channing Way and California Street and one at the near-total watershed scale in the trunkline at Potter Street and 7th Street. Long-term gauging at the near-total watershed scale for Codornices has been ongoing with a flow gauge at Cornell Avenue. Codornices data collection has now been augmented with an upper-watershed flow gauge in the South Fork of Codornices Creek at Codornices Park (above Euclid Avenue).

Detailed tip-bucket rainfall data is available for recent years from a number of locations in the City including rain gauges operated by Balance Hydrologics:

- In the Codornices watershed at Cornell Street,
- just north of the lower portion of the Potter watershed at Bancroft Way and 5th Street, and
- an upper watershed gauge at City Fire Station 7 on Shasta Road just east of Grizzly Peak Boulevard.

2 HYDROLOGIC SETTING

Careful consideration of the specific hydrologic setting is critical in formulating an effective and sustainable watershed management strategy. This applies to the detailed watershed modeling as well, which needs to appropriately represent diverse hydrologic factors such as elevation, slope, land cover, and soils if stormwater runoff volumes and rates are to be simulated successfully.

This section discusses the hydrologic setting of both the Potter and Codornices watersheds, allowing for a direct comparison of the factors that are similar and those that are disparate between the two watersheds. Later sections of this report focus on specific aspects of stormwater conveyance in each watershed.

2.1 Overview of Watersheds and Drainage Patterns in Berkeley

Stormwater runoff within the City of Berkeley drains to San Francisco Bay through ten different watersheds, with each having a distinct outfall point to the Bay. These watersheds include (from south to north): Temescal, Potter, Aquatic Park, Strawberry, Schoolhouse, Gilman, Codornices, Marin, Wildcat, and Cerrito. There are several additional outfalls along the Bay shore, but they handle runoff from small localized drainages or represent circulation connections for the various Aquatic Park lagoons. The outfalls for the southernmost (Temescal) and three northernmost watersheds (Codornices, Marin, and Cerrito) are located outside of the City boundaries.

Berkeley watersheds (except Wildcat) drain in a generally westerly direction, conveying runoff from the Berkeley Hills across the intervening East Bay Plain to the Bay. The Bay shore has been highly altered in the past, with the former extensive beach and marsh environments replaced by fill and armored banks in most cases.

The Potter and Codornices watershed boundaries used in this study are shown in **Figure 2-1.** The Potter watershed is the largest watershed in the City. The watershed as analyzed is a combination of the lands that historically fed Potter and Derby Creeks. Runoff in this watershed is conveyed by pipes which drain to the Bay via the storm drain trunkline that passes along the southern edge of Aquatic Park and under I-80/580.

By contrast the Codornices watershed includes the most extensive open watercourse of any watershed within the city limits. There are numerous (but generally discontinuous) creek culverts at road crossings and under private property. The creek channel largely follows its historic course while inside the city limits. Storm drain infrastructure is generally limited to short runs of pipe carrying lateral inflows to the creek.

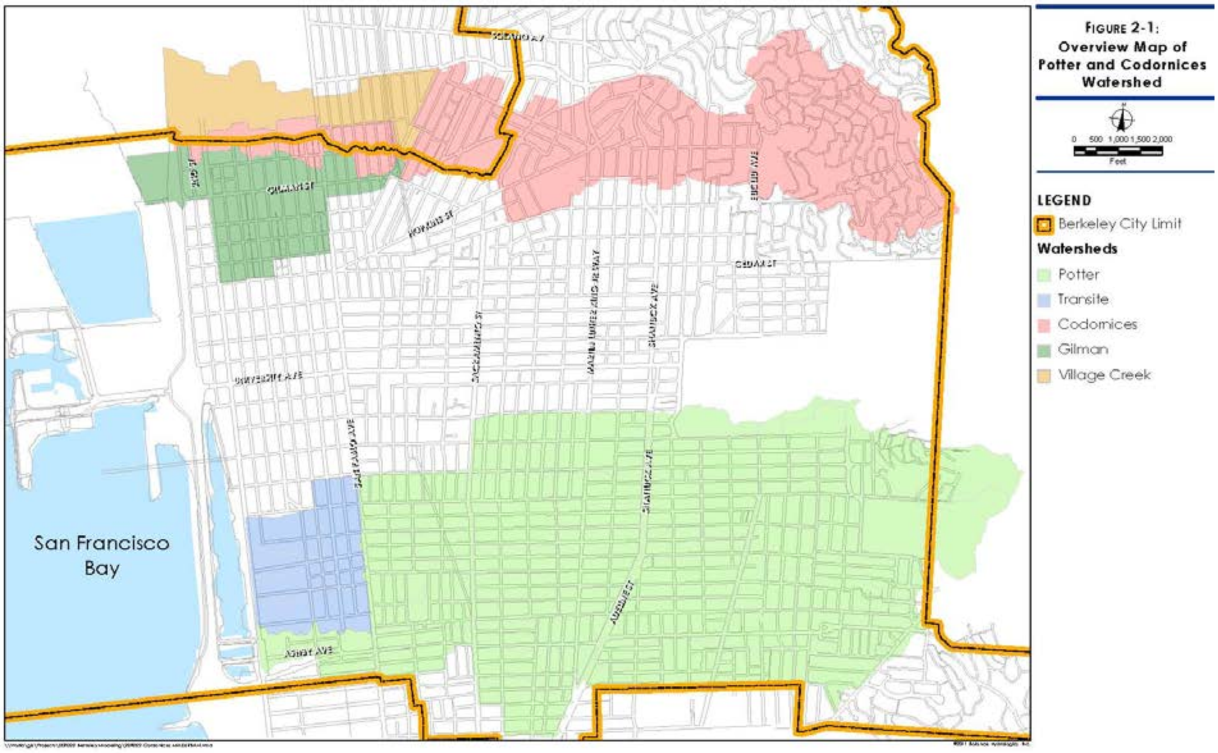


FIGURE 2-1: Overview Map of Potter and Codornices Watershed

2.2 Topography and Land Use

Topography

With a total area of 2,053 acres, the Potter watershed is the largest in the City of Berkeley (see **Table 2-1**). With respect to topography it is most representative of the lower-lying watersheds in the City (Potter plus Schoolhouse and Gilman). This reflects the fact that the headwaters of the watershed are located well below the crest of the Berkeley Hills, with only a small portion extending upslope of the Hayward Fault that marks the upper limit of the East Bay Plain. The maximum elevation is 1,185 feet. Very little of the watershed is located above an elevation of 400 feet (roughly 8 percent), and its centroid is at an elevation of 145 feet. The overall watershed slopes to the southwest in a relatively uniform manner, with steeper grades as one approaches the hills. There is a marked difference in street slopes as east-west trending streets generally have 3 to 4 times the slope of north south trending streets.

The Codornices watershed is much smaller in size, having a total area of 796 acres at the point where it leaves the City at the I-80 freeway.[1] The Codornices watershed is markedly different in topography, being much more representative of the higher-

[1] The watershed is considerably larger at the outlet to the Bay in the City of Albany, totaling 1,065 acres. This reflects the fact that the Village Creek watershed joins the Codornices downstream of the Berkeley city limit.

elevation, steeper slope watersheds (including Strawberry, Marin, and Cerrito). The maximum elevation is 1,330 feet and roughly 44 percent of the watershed is located above 400 feet. The watershed centroid is located at an elevation of 250 feet.

Table 2-1. Topographic Characteristics for the Potter and Codornices Watersheds

	POTTER	CODORNICES
Total Area *(acres)*	2,053	796
Maximum Elevation *(feet)*	1,185	1,332
Elevation of Centroid *(feet)*	145	250
Area above 200 feet *(acres)*	736 (36%)	516 (65%)
Area above 400 feet *(acres)*	159 (8%)	354 (44%)

Land Cover and Drainage Pathways

Table 2-2 summarizes key hydrologic metrics for land cover and the drainage pathways in the two watersheds. The Potter watershed, with much less land area in the steep Berkeley Hills and large areas of commercial and industrial land use has a relatively high impervious cover of 55 percent. In this respect the watershed is, again, more similar to the Schoolhouse and Gilman watersheds. With a larger portion of its area in the steeper hills and a relatively high percentage of park space, the Codornices watershed has a significantly lower impervious cover of 35 percent.

Table 2-2. Land Use and Drainage Pathway Characteristics for the Study Watersheds

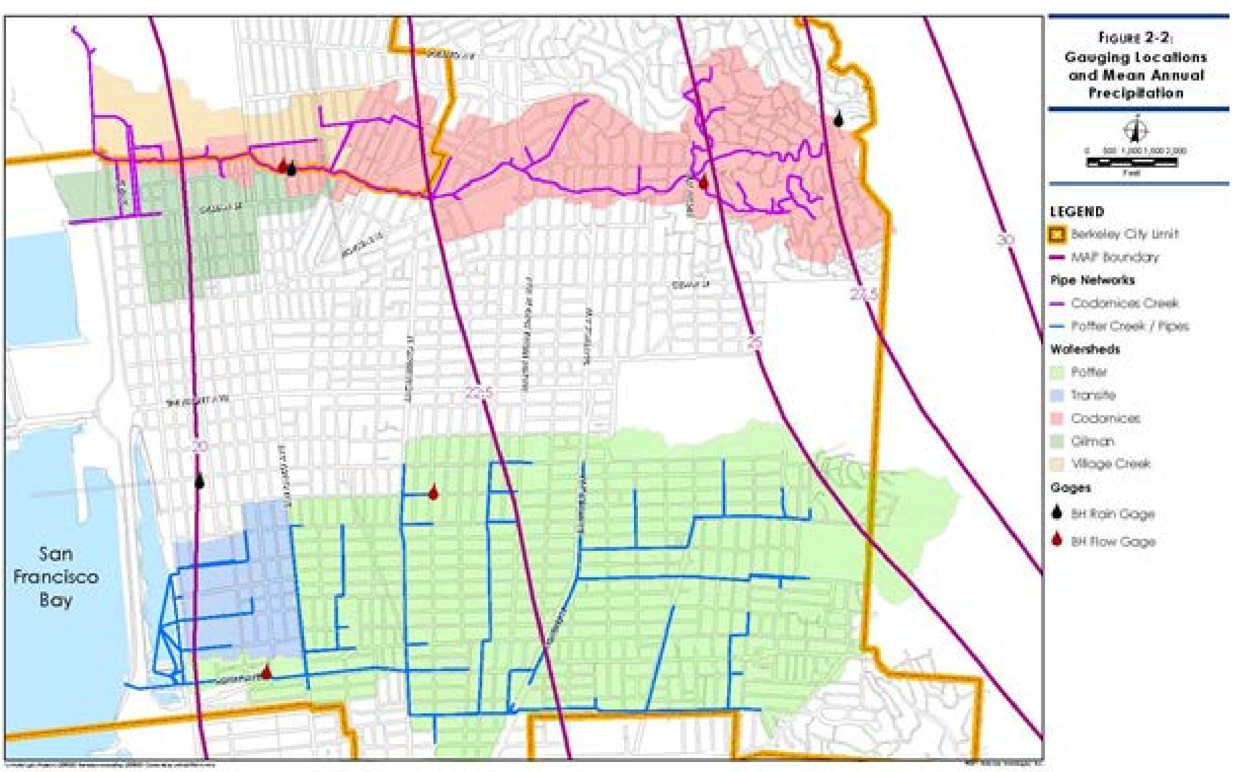
	POTTER	CODORNICES
Impervious Cover *(%)*	55	34
Mean Annual Precipitation *(inches)*	22.4	24.0
Longest Flow Path *(feet)*	20,175	20,525
Flow Path from Centroid *(feet)*	13,810	15,115

The nature of the drainage pathways in the two watersheds includes other distinguishing hydrologic characteristics. One of these is watershed shape. The Potter watershed is roughly rectangular in shape, with a much higher percentage of the drainage area located near its outfall into the Bay. The Codornices watershed has a distinctly different overall shape, with a larger headwaters area tapering down to a narrower watershed corridor across the East Bay Plain, typical of natural creek channels crossing alluvial fan landforms. One measure of this is the fact that the longest flow path in the Potter watershed (20,175 feet) is nearly the same as that for the Codornices watershed (20,525 feet) even though its watershed area is over 2.5 times as large.

2.3 Climate Characteristics

The area encompassing the study watersheds is located in the Mediterranean climate zone typical of coastal, central California. This climate zone is characterized by cool, wet winters and dry summers tempered, in this case, by their proximity to San Francisco Bay and by the occurrence of coastal fog, especially in late spring and summer.

Average rainfall conditions are the statistical mean of rainfall totals that show a wide range of values strongly influenced by global weather patterns such as the El Niño Southern Oscillation and prolonged periods of drought. Additionally, the location of the watersheds adjacent to and/or on the western slopes of the Berkeley Hills strongly influences storm event and annual rainfall totals. The elevations and aspects typical of the Berkeley Hills produce orographic (mountain-induced) precipitation that can be markedly higher than the rainfall that is measured along the edges of San Francisco Bay only several miles to the west. Maps prepared by the Alameda County Flood Control and Water Conservation District show a that mean annual precipitation (MAP) ranges from a low of 20 inches along the Bay shore to a high of approximately 28 inches along the crest of the Berkeley Hills as shown in **Figure 2-2** (ACPWA, 2003).



Long-term meteorological data from the National Weather Service Berkeley gauge (COOP Station 040693, period of 1919 to 2006) show a MAP of 23.6 inches at an elevation of roughly 300 feet. This can be considered typical of the uppermost reaches of the Potter watershed and generally representative of an overall average for the Codornices watershed. A more representative value for the Potter watershed is an area-weighted average of 22.4 inches, scaled off the rainfall mapping prepared by Alameda County Flood Control (see **Table 2-2**). The Codornices watershed has an area-weighted mean annual precipitation of 24.0 inches.

For the purposes of flood analyses, storm event precipitation totals are more important than annual averages. The watersheds in the City can experience relatively large single event rainfall totals. For example, the maximum daily rainfall recorded at the Berkeley gauge was 6.98 inches on January 4, 1982. This compares to a predicted 100-year, 24-hour rainfall total of 5.7 inches per the calculation framework presented in the Alameda County Hydrology and Hydraulics Manual.[2] The modeling presented in this report is based on a 6-hour "balanced" storm distribution with 10-year peak rainfall intensities for the highest 15-minute time step ranging from 1.80 inches/hour for a MAP of 20 inches to 2.26 inches/hour for MAP of 26 inches (see **Table 2-3**). Storm totals for the 10-year, 6-hour design storm range from 1.8 inches to 2.3 inches for the same range of mean annual precipitation.

Table 2-3. Predicted 10-year and 100-year Storm Total Depths

MEAN ANNUAL PRECIPITATION (inches)	STORM EVENTS 10-YEAR, 6-HOUR (inches)	STORM EVENTS 25-YEAR, 6-HOUR (inches)	STORM EVENTS 100-YEAR, 24-HOUR (inches)
20	1.81	2.17	4.95
22	1.96	2.36	5.36
24	2.12	2.54	5.78
26	2.27	2.73	6.20

[2] The term "100-year storm" is used to describe the storm event that has a 100-year return period, with return period defined as the inverse of the annual probability that the event occurs. Many experts in the field have advocated abandoning this terminology since it is often mistakenly interpreted as implying that this event will only happen once in any 100-year period. In reality this event has a 1-percent (1/100) chance of occurring in any given year, even if such a storm occurred the year before. Similarly the "10-year flood" has a 10-percent (1/10) chance of occurring in any given year.

2.4 Soil Characteristics

Soils in the two study watersheds are predominately clay loams and/or urban fill and are almost exclusively characterized by low infiltration capacity. In fact, the published soil survey information for the area (NRCS 2009a, and NRCS 2009b) places all soil groups into either hydrologic soil group (HSG) C or D where designations have been made.[3] Distribution and properties of the major soil types are summarized in **Table 2-4**.

The Potter watershed is dominated by three major soil types that combined underlie roughly 90 percent its area. One of these three is the Urban land – Tierra complex, classified in HSG D, which covers approximately 60 percent of the watershed area. The other two soil types are the Urban land – Clear Lake complex and Urban land. Neither of the latter two is assigned an HSG rating in the soil survey, but both are characterized by low maximum and minimum infiltration rates. Overall, the soil data indicates that the entire watershed is prone to high volumes and rates of runoff.

The Codornices watershed has a somewhat more differentiated variation in soil types with the three most-widely distributed types encompassing roughly 75 percent of the watershed. Given the less developed nature of this watershed, it is not surprising to note a more limited distribution of "urban land" soils. The most common soil type in the watershed is the Xerorthents – Millsholm complex underlying approximately one-third of the area, but not assigned an HSG rating. Roughly one-quarter of the area is in the Urban land – Tierra complex (HSG D) with another eighth in the Xerorthents – Los Osos complex (also not given an HGS rating). Soil properties for the unrated soils show that they would likely be classified as HSG C or D if classifications were assigned.

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[3] The Natural Resource Conservation Service (NRCS) hydrologic soil groups divide all soil types into one of four categories on the basis of potential to produce runoff, ranging from group A to Group D. Group A soils have the lowest runoff potential and typically have high infiltration rates. Group D soils have the highest runoff potential and typically have low infiltration rates and/or are shallow.

Table 2-4. Hydrologic Properties of Soils in the Potter and Codornices Watersheds

SOIL TYPE	WATERSHED COVERAGE (%)	HYDROLOGIC SOIL GROUP	INFILTRATION RATE (inches/hour)	
			Maximum	Minimum
Potter Watershed				
Urban land – Tierra (2 to 5 %	60	D	1.00	0.06
Urban land – Clear Lake	18	n.a.	0.20	0.06
Urban land	12	n.a.	0.10	0.01
Codornices Watershed				
Xerorthents – Millsholm	36	n.a.	1.00	0.10
Urban land – Tierra (2 to 5 %	27	D	1.00	0.06
Xerorthents – Los Osos	12	n.a.	0.40	0.06

2.5 Tidal Conditions Impacting Conveyance

Both of the study watersheds experience tidal effects at and near their outlets to the Bay, with high tide levels and storm surge effects often directly impacting hydraulic conveyance capacity and leading to increased potential for localized flooding.

Statistics from the long-term tidal gauging station operated by the National Oceanic and Atmospheric Administration at the Golden Gate are generally applicable to the shore area in the City of Berkeley and are summarized in **Table 2-5**. Mean higher high water is approximately 0.0 feet in the Berkeley datum, while mean tide level is -2.6 feet.[4] As noted, maximum tide levels can be much higher due to a number of factors most often associated with storm surge effects. For example, the highest observed tide for the reference data set was 2.8 feet recorded in January of 1983.

[4] Unless otherwise noted, all elevations referenced in this report are in the City of Berkeley datum, which is the North American Vertical Datum of 1988 + 5.9 feet (NAVD + 5.9).

Table 2-5. Tidal Datums for the City of Berkeley

TIDAL DATUM	ELEVATION (feet, City of Berkeley datum)
Mean Higher High Water	0.02
Mean High Water	-0.59
Mean Tide Level	-2.64
Mean Low Water	-4.69
Mean Lower Low Water	-5.82
North American Vertical Datum	-5.88
Highest Observed Tide (01/27/1983)	2.84

3 HYDRODYNAMIC MODELING TECHNICAL APPROACH

3.1 Model Platform Considerations

A number of considerations were involved in the selection of the modeling software for the study. Primary among these was the City of Berkeley's interest in developing watershed models that had the greatest flexibility for planning and design purposes for a wide range of potential uses. These uses include green infrastructure and detention storage modeling that called for a platform capable of non-steady state hydrodynamic simulations. The need to represent both channel and pipe hydraulics at a detailed scale was another consideration as was the additional ongoing interest in capabilities for continuous simulation modeling of long-term precipitation records in addition to single, discrete storm events. Low ongoing model maintenance costs were another factor, so that the final models could be archived by the City and updated with ease as needed.

Based on these considerations, the primary modeling tool selected was the U.S. Environmental Protection Agency's Storm Water Management Model (SWMM). This model met all of the criteria cited above, although it is worth noting that there are limitations in the rainfall-runoff modeling capabilities (see below). All modeling for the highly urbanized Potter watershed was carried out using SWMM through the Mike Urban software platform by DHI Software and Environment. However, final modeling of the Codornices watershed was completed using the XP-Storm platform from XP Software. This model was chosen for its integrated ability to model 2-D overland flow as well as using the SWMM engine to simulate channel/pipe flow, an important consideration given the flooding issues known to exist in the lowermost part of the watershed.

Each watershed was modeled separately to reduce complexity and run times.

3.2 Design Storms

A number of potential design storm types and durations were assessed in the initial phases of the study. These included storm events with durations from three to 24 hours with several different rainfall distributions. The result of this review was the selection of a 6-hour duration for the base design storm. This storm duration provides a reasonable balance between longer shorter duration events, where peak intensities might be higher and longer duration events, which by their nature produce a higher runoff volume that may be a consideration in detention or storage assessments. The 6-hour event also provides continuity with the 1994 CH2M Hill Report and the Alameda County Flood Control and Water Conservation District that also use a 6-hour duration design storm.

Noting that a major objective of the study is identification of locations with conveyance restrictions and that time lag values are likely to be small along lateral lines, a design rainfall distribution that embeds short duration rainfall intensities was also considered to be essential. Therefore, a "balanced" rainfall distribution was selected that includes 15-minute intervals for the hyetograph and preserves rainfall intensity probabilities for all durations around a peak centered at the 15-minute interval ending at 3 hours after the start of rain (see **Table 3-1** for intensities and depths for this design storm for 24 inch mean annual precipitation areas). This is quite conservative rainfall distribution, as it assumes that the 10-year 6-hour storm also includes the 10-year 15-minute peak, the 10-year 30-minute peak, etc.

Table 3-1. 10-year Design Storm Rainfall Distribution for 24 inch Mean Annual Precipitation Areas

TIME INTERVAL	**RAINFALL DEPTH** (inches)	**INTENSITY** (in/hr)
15-minute	0.53	2.11
30-minute	0.72	1.43
60-minute	0.97	0.97
2-hour	1.31	0.65
4-hour	1.77	0.44
6-hour	2.11	0.35

3.3 Hydrologic Modeling (Rainfall-Runoff)

A number of hydrologic variables were needed as input values to the hydrodynamic models. A full tabulation of these values is included in the attached appendices for the various model runs. Nonetheless, a brief description of the most important variables is warranted.

Catchments

The overall area of each watershed was subdivided into catchments (sub-watershed) for modeling purposes. Catchments were generally selected to best simulate lateral inflows along the drainage pathways, with additional consideration given to maintaining a relatively uniform distribution in catchment size to avoid scale effects in the rainfall-runoff modeling. Catchment boundaries were digitized In GIS using the City's comprehensive LiDAR-derived[5] 18-inch contour base supplemented by field investigations where drainage pathways were ambiguous. Impervious area for each catchment was assessed using the City's color orthophoto base in GIS. Flow path length and slope were also calculated using the LiDAR topo base. Specific aspects of the catchment delineations are discussed in the Section 4 and 5 of this report for the Potter and Codornices watersheds respectively.

The Potter watershed model includes catchments that drain directly to Aquatic Park and tie into the Potter trunk storm drain line via the so-called "Transite" pipe. These

[5] LiDAR is the acronym for Light Detection and Ranging.

catchments were included in the modeling to allow for an assessment of how much runoff enters the lagoons from the Transite pipe and to provide a modeling base to simulate alternative configurations for the pipe that would eliminate or greatly reduce runoff to Aquatic Park.

Analogously, the Codornices model includes additional catchments to simulate the complex flow patterns downstream of San Pablo Avenue. These catchments include those for Village Creek in the City of Albany (to account for tailwater effects in the reach through the City of Albany to the Bay and to allow simulation of the Codornices-Village bypass structure) and those for the Gilman Street watershed (to allow for modeling of Codornices overflows to the south).

Rainfall-Runoff Transform Function

Design storm rainfall is converted to runoff in the hydrologic model through use of a rainfall-runoff transform function. In all cases, the rainfall-runoff transform used was the SWMM runoff function. This function is discussed in detail in a number of SWMM reference documents (e.g. U.S. Environmental Protection Agency, 2010). Although the XP-Storm platform allows for numerous other transform functions to be used, the SWMM runoff function was selected for continuity with the modeling done on the SWMM platform.

Initial Abstraction and Infiltration

Rainfall losses were represented using depression storage for initial abstraction and Horton's equation for infiltration. Soil properties were extracted from soil mapping prepared in GIS using NRCS soil survey data (NRCS 2009a and b) based on area weighted averaging. Decay rates for infiltration rate (rate at which the soil infiltration capability decreases during a storm) were uniformly set at 2 inches per hour per hour. Depression storage was applied to 75 percent of directly connected impervious areas.

Time Lag

Time lag is not specifically an input variable in the SWMM runoff function. Rather, catchment width is used. The modeling in this study calculated catchment width as the catchment area in square feet divided by the length of the longest representative flow path within the catchment in feet.

3.4 Hydraulic Modeling (Runoff Routing)

As discussed previously all hydraulic modeling, irrespective of software platform, is based on the SWMM engine. Tables of pertinent input hydraulic variables are also included in the attached appendices with discussion of specific variable considerations below.

Storm Drain Pipe Hydraulics

Pipe geometry information (including such variables as length, size, shape, etc.) was exported from the City's GIS database and then imported into the respective watershed model. Similar export and import routines were used to load node information (rim elevation, invert elevation, etc.) into the models. Where node rim elevation data was in obvious conflict with the LiDAR topo base or missing, an appropriate estimate was made using the pertinent contour information from the LiDAR. Invert information was found to be missing for many nodes, particularly for "YT" junctions, which generally represent pipe junctions that do not have an associated manhole access. Where invert information was missing, estimates were made by extrapolating downstream pipe slope information and/or assuming a constant slope for pipe segments between nodes with known invert elevations.

Almost all pipe segments within the study watersheds date from the first half of the last century. Therefore, pipe roughness values can reasonably be expected to reflect decades of wear and abrasion. On this basis a Manning's roughness of 0.015 was used for all concrete pipe sections. This was increased to 0.025 in the few cases where the GIS database indicated corrugated metal pipe exists.

Node and pipe link names for the Potter watershed were taken directly from the City's GIS database, which uses a numbering system taken from the 1994 CH2M Hill Report. The nomenclature system identifies the type of node and associates each pipe with its respective upstream and downstream nodes as described in **Table 3-2**. This numbering convention includes creek culverts as well.

Table 3-2. Identification Numbering Convention, City of Berkeley

INFRASTRUCTURE TYPE	NUMBERING
Manhole	000-099
Catch basin	100-299
Cross inlets/outlets	300-699
Inlets and outlets	700-799
Wyes and tees (YTs)	800-899
Structures owned by others	900-999

Open Channel Hydraulics

Open creek channels (primarily found in the Codornices watershed) were modeled using the open channel functionality in SWMM. Channel cross-section information was developed from a variety of sources including previously surveyed sections assembled as part of the City's creek setback ordinance work, previous hydraulic studies of the lower reaches of the creek and from the LiDAR topo base where needed.

Roughness values were adjusted to account for bed and bank conditions as well as the type and extent of vegetation along the open channel reaches. Typical roughness values ranged from 0.05 in moderately vegetated reaches to 0.10 in selected reaches with dense vegetation.

Ponding and Storage

As discussed in ensuing sections of this report, there are numerous locations where the modeling predicts overflow due to inadequate conveyance capacity. The SWMM model allows for overflows to be treated as lost to the model or to be held as locally ponded runoff that drains back into the model when total flow falls back below storm drain pipe or creek capacity. The latter approach was used in the modeling for this study so that overall runoff volume is conserved. Ponding is occurs at the nodes where overflow is predicted and the ponded water is assumed to flow back into the network at the same node. This is a generalization that does not account for the potential for alternative overland flow paths (typically in streets) that could divert overflows to other inlets. Future 2-dimensional overland flow modeling can be used to resolve this issue in more detail.

Being a true hydrodynamic modeling engine, SWMM does account for storage within the pipes and channels used in the models. However, specific storage nodes were used where appropriate. Examples in this regard include the lagoons at Aquatic Park in the Potter models and storage pipes used to simulate green infrastructure improvements.

Tailwater Considerations

As discussed previously, the lower reaches of both study watersheds are subject to tidal influences. To account for this, all modeling in this study used a constant tailwater elevation of 0.0 feet. This is equivalent to mean higher high water and is somewhat more conservative (by approximately 0.6 feet) than the mean high water standard typically applied in floodplain mapping work by the Federal Emergency Management Agency. Conveyance and capacity in the lowermost part of the watersheds would be expected to be greater for lower tidal conditions and substantially greater at low tides. Nonetheless, the use of mean higher high water was deemed appropriate as an initial accounting of potential future sea level rise over the near-term.

3.5 Design Criteria

The City of Berkeley uses the design criteria recommended in the 1994 CH2M Hill Report. This includes a 10-year design standard for watershed areas of less than 1,000 acres and a 25-year standard for those of greater size, with no allowance for freeboard (e.g. surcharging to ground level allowed). In the City, only the Potter and Strawberry watersheds have infrastructure serving areas greater than 1,000 acres.

3.6 Green Infrastructure Modeling Considerations

A range of potential green infrastructure (GI) elements was evaluated as part of the Potter watershed model development process. These elements included storage systems of various scales (rain barrels, cisterns), porous pavement options for streets (porous asphalt, pavers), and biofiltration features (such as rain gardens, planter strips).

The initial modeling results showed that the key factor in green infrastructure selection with respect to stormwater runoff for large events is storage. The soil and terrain characteristics within the City limits were not deemed conducive for infiltration type best management practices (BMPs) that relied on deep percolation of runoff as a primary control mechanism. Therefore, even in the case of porous pavement options and biofiltration features, any infiltrated runoff was assumed to eventually drain to the existing creek, creek culvert, or storm drain pipeline.[6] In such a case, the effectiveness of these treatment measures is predicated by the available storage for the infiltrated runoff. Thus they function, from a peak flow control perspective, analogously to simple storage elements such as cisterns.

Based on this observation, modeling of green infrastructure components was shifted to generic representations of storage volume in catchments within each watershed. The volume in this case can be considered as the sum of the various storage elements that may be included in any catchment. Detailed design of the elements is beyond the scope of this study, but the storage volumes that have been used were selected based on a preliminary assessment of constraints that included factors such as street width and slope, right-of-way availability (e.g. without mature trees) and potential utility conflicts (e.g. water and sewer laterals).

[6] The effectiveness of infiltration based BMPs at controlling peak stormwater flows would be expected to be markedly improved in areas with suitable (high infiltration rate) soils and low water tables.

4 POTTER WATERSHED MODELING AND RESULTS

In general four main models were run for each of the study watersheds. These include the following:

1. **Existing Conditions Model**. A baseline model of the existing conditions in each watershed was created to identify and quantify capacity limitations.

2. **Traditional Retrofit Model.** The term "traditional" retrofit refers to the hydrodynamic model created to identify and quantify conveyance capacity improvements only. This represents the traditional, but now significantly disfavored, "capture and convey" approach to stormwater management and does not include any green infrastructure improvements.

3. **Green Infrastructure Retrofit Model.** A series of models used to assess the amount of green infrastructure that would be necessary to eliminate the need for all, or a significant portion, of the conveyance capacity enhancements identified in the traditional retrofit model. The final green infrastructure model includes the preferred maximum utilization of GI storage for peak flow control.

4. **Special Considerations Model.** For both watersheds additional modeling was carried out to evaluate other peak flow controls (in addition to green infrastructure) that can be used to address localized flooding issues.

4.1 Existing Conditions Hydrodynamic Model

Model Framework and Special Considerations

The Potter watershed was divided into a total of 74 catchments for hydrologic modeling purposes. The model routes stormwater runoff through a network that includes 488 storm drain pipes with a total length of roughly 80,500 feet. The catchments delineations for the Potter watershed model are illustrated in **Figure A-1** and summarized in **Table A-1** in **Appendix A**.

In most respects the Potter storm drain network is conventional, though there are many now non-standard pipe shapes (e.g. horseshoe, egg) and sizes that reflect the age of the infrastructure in most locations. The longest flow path through the modeled network originates near Panoramic Way just southeast of Memorial Stadium. The overall watershed is divided into two major areas (east and west) along the storm drain branch that runs south along Shattuck-Adeline to the Ashby BART Station. Catchments to the east are steeper and are generally a mix of residential and open space areas with the larger storm drain pipes running east-west (Dwight Way, Derby Street, Woolsey Street). Catchments to the west are much gentler in slope and include a mix of residential, commercial, light industrial land uses with branch storm drain lines running north to south (Grant Street and Ellis Street, Sacramento Street and San Pablo Avenue) to meet the trunkline line along Woolsey Street or Ashby Avenue. As mentioned earlier there are only a few open channel segments, essentially all of which are located in the most upstream reaches of the watershed and are not included in the modeling.

The notable complexity in the Potter watershed is the interflow that occurs with the Aquatic Park lagoons. The lower end of the Potter trunk line conveys water to the Bay just north of the Ashby Avenue / I-80 interchange. However, the trunk pipe also includes a cross-connection to the Model Yacht Basin (MYB), which is itself connected by pipes to the Main Lagoon (ML). This cross-connection is one of the primary means for circulating Bay water into and out of the two lagoons during non-storm periods. During storm periods, runoff from the trunk line flows into the MYB whenever the hydraulic grade line in the Potter trunk line is higher than the water surface in the MYB. After the storm peak passes, flow reverses and the MYB drains out to the Bay via the Potter trunk line. In combination, the two lagoons provide a very large detention capacity for excess stormwater flows in the lowermost portions of the watershed. In fact, substantial overflows of the trunkline occur west of the Union Pacific Railroad (UPRR) tracks resulting in overland flows to the lagoons, especially during high tides, which create a high tailwater condition and raise hydraulic gradelines in the trunkline pipes.

Additional flow enters the lagoon from the “Transite” storm drain line that was installed to intercept low flows from a number of storm drains serving a 178-acre almost completely industrial area stretching from Heinz Avenue north to roughly Dwight Way. The connections to the Transite line include weir structures to direct low flows to the Potter trunkline near the intersection of Shellmound Street and Bolivar Drive, with high flows allowed to pass to the MYB or ML.

Existing Pipe Capacities, Conveyance Limitations, and Localized Flooding

The complete existing conditions modeling results for the 10-year design storm in the Potter watershed are included in **Appendix B**, with the modeled pipes and location of predicted network overflow shown in **Figure B-1**. The modeling results also include the 25-year storm routing for the main trunkline. The model output reveals a number of issues that merit consideration, including the following:

1. Storm drain pipe capacities. The modeling shows many of the storm drain pipes are under capacity for the 10-year design storm. A total of 271 pipes are predicted to flow at or beyond full capacity, with the average pipe at roughly 150% of full flow. Under capacity pipes are widespread throughout the watershed, but tend to be located along the lower slope north-south streets.

2. Predicted flooding. The modeling predicts that a large volume of stormwater would either be unable to enter the pipe network or would overflow at catch basins and manholes. Total overflow volume is estimated to be on the order of 36 acre-feet or roughly 15 percent of the total runoff of 236 acre-feet. Particularly high risks of overflow are noted along Woolsey just west of the Ashby BART Station where several major lines combine and land slope decreases west of Adeline Street.

3. Tailwater flooding. Significant overflow issues are predicted along the north-south branch lines located along Ellis Street and along San Pablo Avenue. In

large part these overflows are caused by the high hydraulic grade lines in the trunkline line (in Woolsey Street and Ashby Avenue respectively).

4. Flow to Aquatic Park. The lowermost reaches of the pipe network west of the UPRR tracks are markedly under capacity due to their size, very shallow slope and susceptibility to tidal backwater. This is reflected in the very large volume of stormwater predicted to flow to the Aquatic Park lagoons, either directly as piped flow through inter-connections or as overflow in the local vicinity. The modeling indicates this volume could be as high as 68 acre-feet or nearly 29 percent of the total runoff.

5. Peak flow rates. The model predicts that the maximum peak flow out to The Bay to be approximately 445 cfs due to various constrictions and inter-connections to Aquatic Park.

4.2 Traditional Retrofit Hydrodynamic Model

Model Framework and Special Considerations

The traditional retrofit model was developed by systematically increasing pipe sizes where additional conveyance needs had been identified. Standard pipe sizes were used, with round reinforced concrete pipe as the preferred section except where cover issues were identified or a box pipe sections was needed for capacity reasons. The trunkline and Transite pipe connections to Aquatic Park were not changed, although pipe capacity was increase such that overland flows to the lagoons were generally eliminated.

Retrofit Requirements and Projected Costs

The results of the traditional retrofit modeling are included in **Appendix C**. The associated map for this scenario has not been included as all pipes shown would have sufficient capacity to avoid overflows. Key points to note include:

1. Pipe capacities. **Table C-1** summarizes the required pipe sizes for this scenario and shows that very large pipes would be needed in many locations, especially on the trunkline line in Ashby Avenue, Potter Street, and west through Aquatic Park. For example, the trunkline line west of San Pablo Avenue is currently 9-foot egg-shaped pipe. For a traditional retrofit this would have to be replaced with 10-foot X 10-foot box pipe to convey the full 10-year discharge.

2. Predicted flooding. Only a very small amount of residual flooding is predicted, on the order of 2 acre-feet. Complete elimination of flooding could be accomplished with even larger pipes, but was not modeled as the marginal cost of eliminating the last overflow was deemed prohibitively high.

3. Tailwater flooding. Although the trunkline retrofit pipe sizes would be very large, they would have sufficient capacity to reduce tailwater flooding in the north-

south storm drain branches, reducing the need for, or size of, retrofits in those lines.

4. Flow diversion to Aquatic Park. Even with the markedly increased capacity, water surface elevations in Aquatic Park with respect to high tide are such that there would still be stormwater diversion to the lagoons. However, the total predicted diversion volume to the lagoons would be reduced by somewhat more than half to roughly 30 acre-feet.

5. Peak flow rates. The increased conveyance and reduced transient detention effects due to overflow and diversion to the Aquatic Park lagoons would markedly increase peak discharge at the Bay outfall to 1,110 cfs.

Estimated capital costs for a full traditional retrofit of the Potter watershed network are also included in **Appendix C**. Unit cost data for pipe replacement was estimated for the densely urbanized setting and adjusted for full capital cost requirements. The preliminary estimate for full retrofit is on the order of $55 million and entails the replacement of approximately 35,000 linear feet of pipe.

Several distinct disadvantages are associated with of the traditional retrofit approach in addition to the very high capital costs. These include the fact that there are few auxiliary benefits, such as enhanced water quality, associated with upsizing pipes. Equally of concern is the sequencing of implementation, as increases in conveyance capacity cannot legitimately be made in one location if they result in higher peak flows (and more overflow risk) elsewhere in the City.

4.3 Modeling of Potential Green Infrastructure Retrofits

Green Infrastructure Scenarios and Limitations

The green infrastructure modeling was developed from the existing conditions model. Sections of under capacity pipe were identified and then an assessment was made whether there was enough public-right-of way of suitable characteristics in the respective catchments to provide sufficient stormwater storage within a generalized green infrastructure implementation. If so, representative storage elements were added to the model, which was then re-run to assess the need for further incremental storage additions. Detailed siting of specific features was beyond the scope of the study, but efforts were made to model green infrastructure storage only where physical factors such as street slope, depth of downstream storm drain lines, etc. indicate that such installations would be practical.

Green Infrastructure Retrofit for Flood Control and Projected Costs

The modeling demonstrated that appropriate implementation of green infrastructure in the public right-of-way can have a significant impact on the need for storm drain retrofits. As with most methods that detain and release stormwater over a period of time, green infrastructure can reduce peak flows both by metering runoff that is handled directly and, importantly, by affecting the timing of flow such that different storm drain pipelines in the network are not contributing peak rates at the same time. The latter reason, in particular, explains why the modeling found the highest efficiency in peak flow reduction using GI was in the upper reaches of the watershed, particularly east of Shattuck-Adeline.

The required amount of storage to completely obviate the need for increasing pipe capacity is quite high, as is to be expected in a watershed such as Potter that is subject to extensive network overflows in its existing condition. However, the hydrodynamic modeling shows that selective placement of green infrastructure storage can cost-effectively reduce the expenditures needed by allowing smaller diameter storm drain pipes to be installed. For example, installation of approximately 50 green infrastructure projects could reduce the need to upsize existing storm drain pipes so that $33 million in retrofit costs could be avoided. However, a green infrastructure implementation on this order would be roughly the same cost as at traditional network retrofit, while providing substantial additional benefits for stormwater runoff quality and reduction of trash flows to the Bay.

Another important consideration is that green infrastructure storage can be installed incrementally almost wherever desired in the watershed without creating adverse flooding impacts elsewhere.

4.4 Additional Modeling of Outfall Routing Options

The modeling results indicated tailwater conditions in the lower trunkline were a significant influence on drainage conveyance. Thus, additional options for stormwater routing were examined. These included means of reducing or preventing stormwater diversions to the lagoons through relocation of the Transite pipe, construction of pump stations, or the construction of a pressure pipe segment to convey runoff through the park.

The various options examined are summarized in **Table D-1** in **Appendix D**. The preferred option identified was construction of a new 8-foot diameter pressure pipe to the Bay from a new weir/trash rack structure at the western end of Potter Street. The weir structure would divert flows to the lagoons only in the infrequent case when the capacity of the pressure pipe was exceeded, generally only for storms approaching a 10-year magnitude. The existing lower Potter trunkline would be left in place and used as circulation enhancement for the lagoons. The existing Transite pipe would be replaced with a new storm drain line following the UPRR right-of-way to the weir/trash rack structure.

These outlet modifications would allow increased flow rates to be discharged to the Bay (compared to existing conditions) without diversion to Aquatic Park, making conveyance enhancements in the upper watershed possible. A primary candidate in this regard would be the trunkline line along Potter Street and Ashby Avenue. The modeling confirms that such a system would almost completely eliminate stormwater diversions to Aquatic Park if a suitable number of green infrastructure installations were constructed in the upper watershed. Model output files and network maps for the latter configuration of pressure pipe, weir with trash rack, relocated Transite line, selective pipe upsizing and green infrastructure are included in **Appendix D**.

Representative hydrographs of peak flow at several locations in the Potter watershed for the primary options models are shown in **Figures 4-1** to **4-3**.

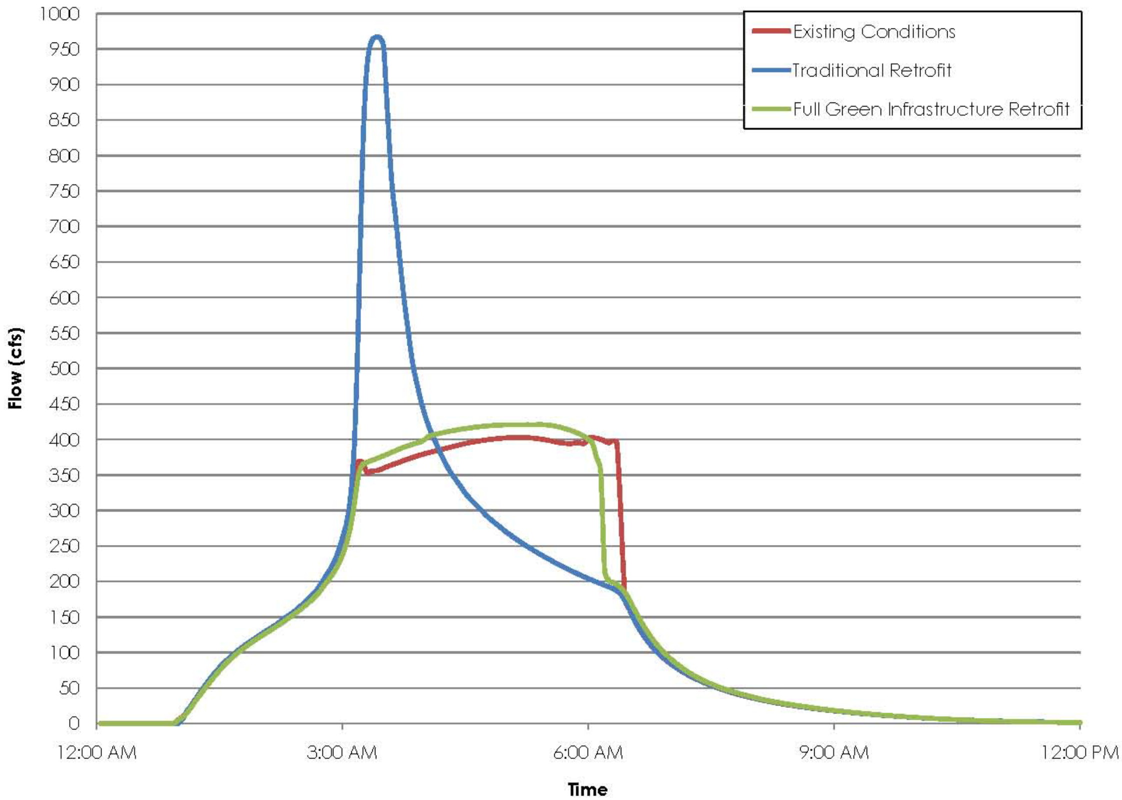

Figure 4-1: Potter Watershed Design Storm Hydrograph, Main Trunk downstream of King Street.

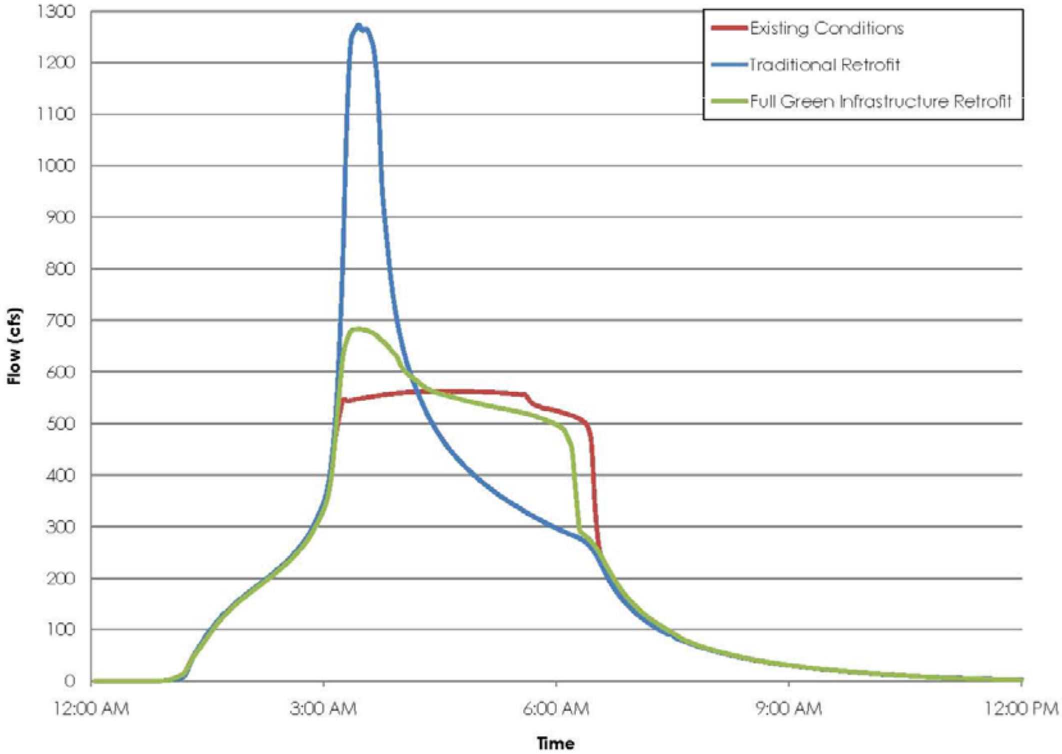

Figure 4-2: Potter Watershed Design Storm Hydrograph, Main Trunk downstream of San Pablo Avenue.

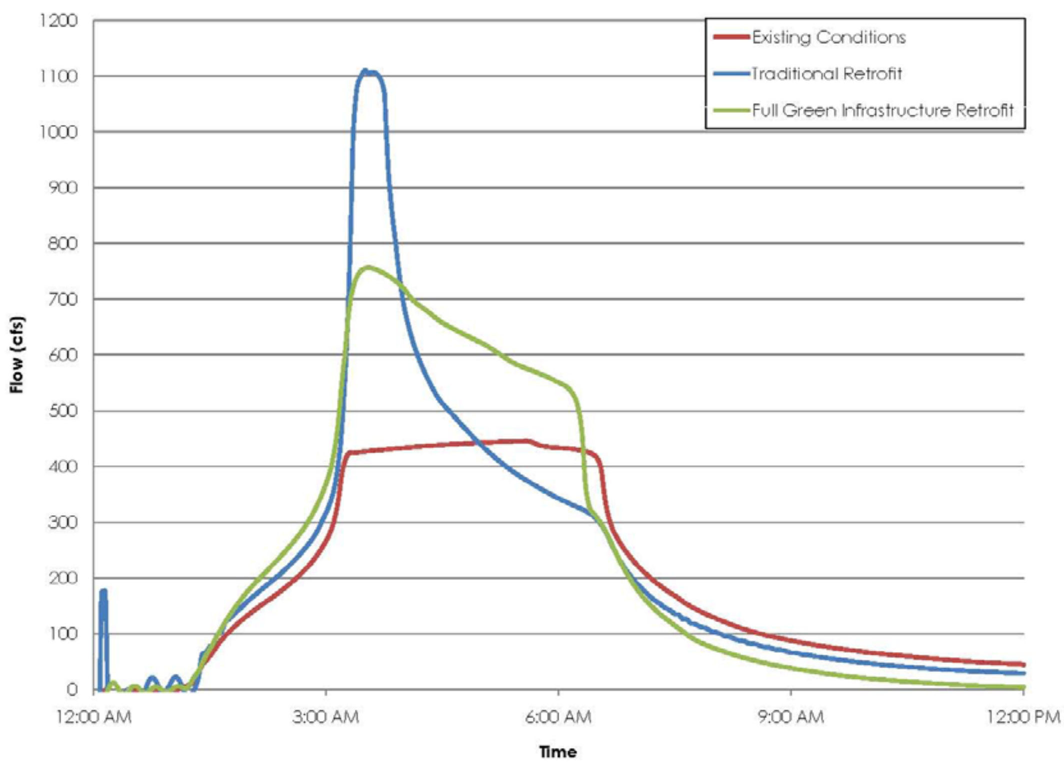

Figure 4-3: Codornices Watershed Design Storm Hydrograph, Main Trunk before outlet into San Francisco Bay.

5 CODORNICES WATERSHED MODELING AND RESULTS

Modeling of the Codornices watershed proceeded in a similar manner to that for the Potter watershed, with the only significant differences being the much larger number of open channel segments (and associated road crossings termed "creek culverts" here) and the use of the XP-Storm modeling platform.

5.1 Existing Conditions Hydrodynamic Model

Model Framework and Special Considerations

The Codornices watershed was divided into a total of 61 catchments for hydrologic modeling purposes. The model routes stormwater runoff through a total of 388 culverts and storm drain pipes with a total length of approximately 39,000 feet. The model also includes open channel segments with a total length of 24,100 feet. The catchments delineations for the watershed model are illustrated in **Figure E-1** and summarized in **Appendix E**.

Substantial estimation of creek culvert and channel inverts, and channel cross section shapes was necessary due to lack of information in the GIS database. The creek passes through numerous private property parcels, and the tree canopy prevents effective aerial imagery or surveying, so a large portion of the channel is undefined, necessitating best estimates for missing data.

The creek channel crosses numerous streets as it makes its way west through Berkeley. Based on the topographic survey data, runoff will most often flow over such streets and back into the channel when the creek culvert capacity at the crossing is exceeded. Overstreet flows at creek culverts were typically modeled as trapezoidal cross sections with a bottom width of 20 to 60 feet and shallow side slope of 20 to 30 ft/ft, depending on the topography. The profile was measured and input directly in some cases where a sag in the road was clearly defined in the topography. Overstreet flow links were not included for locations where a clear flow path over the road was not evident.

Existing Pipe Capacities, Conveyance Limitations, and Localized Flooding

The results of the existing conditions model are included in **Appendix F** with the watershed capacity map shown in **Figure F-1**.

The upper watershed drainage pathways include three main branches above Codornices Park, including: 1) storm drain pipes in Euclid Avenue, 2) the north fork of Codornices Creek up to Shasta Road, and 3) the south fork of Codornices Creek up past La Loma Park. The modeling shows numerous undersized storm drain pipes along Euclid Avenue. Only a few storm drain pipes are indicated as undersized on the north fork, although significant overflow is predicted on Shasta Road above Queens Road. No undersized pipes are shown for the south fork above Codornices Park. The confluence of these upper lines at Codornices Park is an important location where the majority of the upper watershed area concentrates into the main channel and flows into the notably steep creek culvert through the Rose Garden.

Below the Rose Garden the drainage primarily consists of the main channel with a few small tributary lines coming in, three of which are in the City of Albany. The model predicts flooding at the creek culverts under Glen Avenue and under 6th Street. It also shows overflow at nodes in many of the tributary lines coming into the main channel and in some reaches of the channel itself.

Overbank flooding and flow south down 2nd Street west of the UPRR tracks is perhaps the most frequent and visible problem in the Codornices watershed. Overflow is predicted at numerous locations along the lower portion of the Gilman storm drain line, which is included in the model as it is the ultimate route to the Bay for runoff that flows down 2nd Street and south on the east and west railroad rights-or-way. The capacity limitations are tabulated in **Appendix F**.

Table F-1 summarizes the peak flow rates at key points in the watershed as well as the overflow volume at 2nd Street, and the total overflow in the watershed.

5.2 Traditional Retrofit Hydrodynamic Model

Model Framework and Special Considerations

The traditional retrofit model uses increased pipe sizes as needed to eliminate the risk of overflow. This leads to increased peak flow rates as ponded water is no longer stored, but flows through the watershed uninhibited. Upsized pipes were not modeled in locations where creek culverts are undersized but excess water can directly flow over the road.

Retrofit Requirements and Projected Costs

The model output, retrofit requirements, and projected costs for the traditional retrofit are summarized in **Appendix G**. The retrofit cost table shows few retrofits would be needed for the three lines above Codornices Park, most of those needed would be on the Euclid Avenue line. Capital improvement costs for traditional retrofits above Codornices Park are on the order of $1.6 million.

Additional information is provided retrofit costs for the mainstem of the creek, with the laterals tabulated at the end. The total costs of upsizing the creek culverts within the creek mainstem is estimated to be $1.2 million. The table also shows that one or more pipes would have to be upsized in every lateral line to the mainstem except for 9th Street, resulting in a total cost of an additional $1.2 million. Significant capacity improvements are needed in the Santa Fe and Dartmouth laterals, however these are in the City of Albany, so the cost of these retrofits is not included in the total. The resulting total cost for traditional retrofit in the Codornices watershed within the City of Berkeley is on the order of $4.0 million.

It is worth noting that this cost is over an order of magnitude less than the estimated capital improvement costs to retrofit the Potter watershed. The primary reasons for this include:

1. The Potter watershed has over three times the length of pipe than the Codornices watershed, and the pipes in the Codornices watershed that require replacement are generally smaller, shorter, and easier to access and construct.

2. Essentially all conveyance throughout the entire Potter watershed is significantly undersized.

It is also important to note, that the Codornices traditional retrofits modeled do not eliminate all the flooding in the watershed. That would require a new creek culvert under I-80/580, and the additional cost would significantly increase the Codornices retrofit cost.

5.3 Modeling of Potential Green Infrastructure Retrofits

Non-traditional Retrofit Scenario Components

The installation of non-traditional subsurface storage features or pipes was deemed to be infeasible in the branches above Codornices Park due to steep slopes of the streets and terrain, and concerns about slope stability. Further, preliminary model runs indicated that the addition of these features provided a fairly minimal reduction to flow rates or overflow potential.

Codornices Park, however, is well situated for the installation of sub-surface storage features due to its location at the downstream end of the three upper branches and due to the open field area where storage units could be installed. Therefore, hydrodynamic modeling was carried out to assess the effectiveness of an array of 8-foot diameter storage pipes with a total length of 550 feet at this location.

An additional option was evaluated using pipe storage under Henry Street between the creek channel and Eunice Street. This scenario included the interception of the Euclid Avenue line, which presently comes into Codornices Park and joins with the other branches, and re-routing the flow in a new line down Eunice Street to Henry Street. The storage option was based on four 550-foot long sections of 8-foot diameter pipe and associated orifice to meter outflow to the creek channel.

Green Infrastructure Retrofit and Projected Costs

The addition of storage at Codornices Park and Henry Street as modeled would provide a total of roughly 4.5 acre-feet of storage and was shown to be effective at reducing both peak flows in the creek mainstem and overflow in the watershed. While the reduction in flooding at 2nd Street from approximately 31 acre feet to 29 acre-feet was modest, the overflow reduction from 10.8 to approximately 7 acre-feet elsewhere in the watershed is notable. The improvements are significant relative to the traditional retrofit results, which the modeling shows would cause increased peak flows and increased risk of overflow. The estimated cost for the storage options is on the order of $4.5 million. The storage features would also help to moderate abrupt changes in discharge that result from brief, high intensity storms on the small, steep watershed, which can reduce the potential for erosion of the bed and banks of the creek channel, as well as wash-out of in-stream habitat and biota.

5.4 Additional Modeling of Lower Watershed Routing Options

Modeling results and City experiences indicate significant flooding occurs in the lower watershed. Thus, additional options for stormwater routing were examined. The two most readily available options in the lower watershed to reduce the frequency and volume of flooding at 2nd Street are: 1) constructing a wall or berm at the end of 2nd Street to raise the spill level of the channel, and 2) to open the Village Creek bypass and route water to Village Creek.

Configuration of Scenarios

The first configuration tested in the model was to increase the release elevation at 2nd Street by constructing an earthen berm or low flood wall. This berm or wall would tie into the existing high bank and K-rail to the east and the side slope of the East Shore Highway at the downstream creek culvert headwall. The structure was assumed to have a 40-foot long low section at an elevation of 6.7 feet that would control the release of overflow from the channel. The remaining portion of the structure would be set to a elevation of 7.3 feet, so that above this elevation flow would occur over the full length of the wall, thus minimizing further increases in water surface elevation. For this analysis the water surface elevation was kept below the low point (7.8 feet) at the body shop on the north side of the channel at the East Shore Highway culvert headwall.

The inlet to the Codornices/Village Creek bypass is located on the north bank (City of Albany) of the Codornices Creek channel just upstream of the end of 5th Street. The bypass allows water from Codornices Creek to be diverted to a channel along the east side of the railroad and into Village Creek just upstream of the UPRR bridge. This bypass consists of a 4-foot wide flashboard weir that ranges from a minimum (fully open) elevation of 10.3 to a maximum (closed) elevation of 13.8. This weir discharges into a 54-inch (4.5-foot) diameter pipe that extends north past the soccer fields and then west along the soccer fields into an open channel, which turns at the railroad tracks and goes north to the Village Creek channel. This bypass has remained closed since it was installed, but may provide a means of ameliorating flooding problems further downstream in Codornices Creek.

Results and Projected Costs

The model results show that constructing a berm at 2nd Street could reduce overflow rates and volumes down 2nd Street, increase flows through the I-80 culvert and slightly increase the flow along the UPRR tracks to Village Creek. The predicted peak flow down 2nd Street would decrease from 180 cfs under existing conditions to 131 cfs with the berm in place. Likewise the predicted flood volume would decrease from 16 to 9.5 acre-feet. The increase in the water surface elevation from 6.3 to 7.7 feet at the I-80 culvert headwall would increase the flow through the culvert from 195 to 212 cfs. Flows along the UPRR tracks would also be affected, with the flow north to Village Creek increasing from 16 cfs to 28 cfs, and the flow south to Gilman increasing from 16 to 28 cfs.

Fully opening the Village Creek bypass would divert 25 cfs from the Codornices Creek channel north to Village Creek. This would reduce the peak flow down 2nd Street from 180 to 165 cfs and the volume from 16 to 14 acre-feet. The peak flow rates north and south along the railroad tracks would decrease by approximately 30 percent. However, the estimated Village Creek peak flow rate is predicted to only increase by 2 cfs, which is most likely because the peak of the diverted flow from Codornices Creek arrives after the peak flow from the Village Creek watershed.

Opening the bypass and raising the berm elevation at 2nd Street would compound the individual benefits of these measures. The estimated flow rate down 2nd Street would decrease approximately 30 percent to 120 cfs and the flood volume would decease approximately 50 percent to 8.2 acre-feet. Estimated flows north and south along the UPRR tracks increase from 16 to 24 cfs.

5.5 Combination of Lower Watershed and Non-traditional Storage Options

Additional modeling was completed to assess the effectiveness of combining the lower watershed options with sub-surface storage infrastructure at Codornices Park and Henry Street. The model output for this scenario is presented in **Appendix H** and shows that this combination would provide substantial benefits at all locations in the watershed below Euclid Avenue. Overflow discharge down 2nd Street would decrease over 75 percent to 48 cfs, with the volume decreasing 77 percent to 3.5 acre-feet. An additional benefit to this scenario is that the overflows along the UPRR right-of-way would remain constant at 16 cfs for both the north and the south directions. Peak flow through the I-80 culvert would be 212 cfs (increased from 195 cfs).

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Table A-1: Subcatchments for Potter Model

MUID	Tag	Raingage ID	Drainage Area (ac)	Width (ft)	Ground slope (ft/ft)	Imperviousness (%)	Impervious Manning	Pervious Manning	Impervious d. storage	Pervious d. storage	% DCIA w/o d. storage
10	Potter & 7th	MAP20_10YR	50.1	820	1.0	75	0.02	0.425	0.07	0.15	25
13	Carleton & UPRR	MAP20_10YR	16.2	530	1.0	80	0.02	0.425	0.08	0.15	25
23	Derby & Shattuck	MAP22_10YR	28.1	520	3.0	55	0.02	0.425	0.08	0.15	25
38	Carrison & San Pablo	MAP20_10YR	23	600	1.0	55	0.02	0.425	0.08	0.15	25
11_A	Heinz & 5th	MAP20_10YR	20.4	840	1.0	80	0.02	0.425	0.08	0.15	25
11_B	Heinz & 7th	MAP20_10YR	29.3	750	1.0	75	0.02	0.425	0.08	0.15	25
12_A	Grayson & 5th	MAP20_10YR	22.8	1010	1.0	75	0.02	0.425	0.08	0.15	25
12_B	Pardee & 8th	MAP20_10YR	22.1	680	1.0	75	0.02	0.425	0.08	0.15	25
14_A	Parker & 5th	MAP20_10YR	32.2	1180	1.0	80	0.02	0.425	0.08	0.15	25
14_B	Dwight & 8th	MAP20_10YR	34.5	1170	1.0	70	0.02	0.425	0.08	0.15	25
20_A	Oregon & Grant	MAP22_10YR	20.1	580	1.0	55	0.02	0.425	0.08	0.15	25
20_B	Ward & Grant	MAP22_10YR	26	630	1.0	55	0.02	0.425	0.08	0.15	25
20_C	Carleton & Grant	MAP22_10YR	28	650	1.0	55	0.02	0.425	0.08	0.15	25
20_D	Blake & Grant	MAP22_10YR	24.9	610	1.0	55	0.02	0.425	0.08	0.15	25
20_E	Dwight & MLK	MAP22_10YR	33.6	960	1.0	55	0.02	0.425	0.08	0.15	25
21_A	Ashby & Mabel	MAP20_10YR	24.4	740	1.0	60	0.02	0.425	0.08	0.15	25
21_B	Russell & Wallace	MAP20_10YR	10.1	570	1.0	55	0.02	0.425	0.08	0.15	25
21_C	Russell & Mabel	MAP20_10YR	13.801	410	1.0	55	0.02	0.425	0.08	0.15	25
21_D	Oregon & Park	MAP20_10YR	21.7	610	1.0	45	0.02	0.425	0.08	0.15	25
21_E	Ward & Mabel	MAP20_10YR	16.808	410	1.0	35	0.02	0.425	0.08	0.15	25
21_F	Carleton & Mabel	MAP20_10YR	21	740	1.0	55	0.02	0.425	0.08	0.15	25
21_G	Blake & Mabel	MAP20_10YR	19.6	700	1.0	55	0.02	0.425	0.08	0.15	25
21_H	Dwight & Mabel	MAP20_10YR	16.152	440	1.0	55	0.02	0.425	0.08	0.15	25
22_A	Stuart & Sacramento	MAP22_10YR	29.4	720	1.0	55	0.02	0.425	0.08	0.15	25
22_B	Derby & Sacramento	MAP22_10YR	27.5	690	1.0	55	0.02	0.425	0.08	0.15	25
22_C	Parker & Sacramento	MAP22_10YR	43.78	930	1.0	55	0.02	0.425	0.08	0.15	25
24_A1	Tyler & Sacramento	MAP20_10YR	26.362	540	1.0	55	0.02	0.425	0.08	0.15	25
24_A2	Woolsey & California	MAP20_10YR	13.8	610	1.0	55	0.02	0.425	0.08	0.15	25
24_A3	Harmon & California	MAP20_10YR	34.6	920	1.0	55	0.02	0.425	0.08	0.15	25
24_A4	Prince & Ellis	MAP20_10YR	6.5	460	1.0	55	0.02	0.425	0.08	0.15	25
24_A5	Woolsey & Harper	MAP20_10YR	6.682	410	1.0	55	0.02	0.425	0.08	0.15	25
24_B1	Ashby & California	MAP20_10YR	8.902	300	1.0	55	0.02	0.425	0.08	0.15	25
24_B2	Ashby & King	MAP22_10YR	22.757	400	1.0	55	0.02	0.425	0.08	0.15	25
24_C1	Julia & Sacramento	MAP20_10YR	10.904	290	1.0	55	0.02	0.425	0.08	0.15	25
24_C2	Russell & California	MAP20_10YR	18.8	520	1.0	60	0.02	0.425	0.08	0.15	25
25_A	Dwight & California	MAP22_10YR	31.4	920	1.0	55	0.02	0.425	0.08	0.15	25
25_B	Channing & California	MAP22_10YR	28.4	780	1.0	55	0.02	0.425	0.08	0.15	25
25_C	Bancroft & Sacramento	MAP22_10YR	30.263	600	1.0	55	0.02	0.425	0.08	0.15	25
26_A	Oregon & San Pablo	MAP20_10YR	14.322	300	1.0	60	0.02	0.425	0.08	0.15	25
26_B	Derby & San Pablo	MAP20_10YR	23.886	650	1.0	55	0.02	0.425	0.08	0.15	25
26_C	Blake & San Pablo	MAP20_10YR	17.2	720	1.0	55	0.02	0.425	0.08	0.15	25
26_D	Dwight & Byron	MAP20_10YR	22.6	720	1.0	55	0.02	0.425	0.08	0.15	25
27_A	Channing & MLK	MAP22_10YR	31	840	2.0	60	0.02	0.425	0.08	0.15	25
27_B	Bancroft & Milvia	MAP22_10YR	30.7	1040	2.0	60	0.02	0.425	0.08	0.15	25
29_A	Ashby & Adeline	MAP22_10YR	59.3	1140	3.0	50	0.02	0.425	0.08	0.15	25
29_B	Oregon & Adeline	MAP22_10YR	55.4	1060	3.0	50	0.02	0.425	0.08	0.01	25
30_A	Blake & Shattuck	MAP22_10YR	34.7	830	2.0	60	0.02	0.425	0.08	0.15	25
30_B	Channing & Shattuck	MAP22_10YR	29.3	780	2.0	60	0.02	0.425	0.08	0.15	25
30_C	Bancroft & Shattuck	MAP22_10YR	31.8	770	2.0	60	0.02	0.425	0.08	0.15	25
31_A-1	Dwight & Bowditch	MAP26_10YR	13.5	810	4.0	50	0.02	0.425	0.08	0.15	25
31_A-2	Haste & College	MAP26_10YR	15.2	890	4.0	50	0.02	0.425	0.08	0.15	25
31_B	Bancroft & Bowditch	MAP26_10YR	28.4	700	4.0	50	0.02	0.425	0.08	0.15	25
31_C	Channing & Piedmont	MAP26_10YR	21.7	760	6.0	45	0.02	0.425	0.08	15	25
31_D	Dwight & Prospect	MAP26_10YR	94.26	1190	10.0	35	0.02	0.425	0.08	0.15	25
32_A	Derby & Regent	MAP24_10YR	30.7	660	4.0	50	0.02	0.425	0.08	0.15	25
32_B	Derby & Etna	MAP24_10YR	30.6	870	4.0	50	0.02	0.425	0.08	0.15	25
32_C	Derby & Piedmont	MAP26_10YR	23.2	580	6.0	45	0.02	0.425	0.08	0.15	25
32_D	Derbry & Claremont	MAP26_10YR	99.8	1370	10.0	30	0.02	0.425	0.08	0.15	25
33_A	Ashby BART	MAP22_10YR	7.6	350	1.0	75	0.02	0.425	0.08	0.15	25
33_B	Essex & Adeline	MAP22_10YR	16.3	530	1.0	55	0.02	0.425	0.08	0.15	25
33_C	Woolsey & Adeline	MAP22_10YR	7	450	1.0	60	0.02	0.425	0.08	0.15	25
33_D	Woolsey & Tremont	MAP22_10YR	15.339	650	2.0	55	0.02	0.425	0.08	0.15	25
33_E	Prince & Wheeler	MAP22_10YR	31.3	810	2.0	55	0.02	0.425	0.08	0.15	25
34_A	Ashby & Benvenue	MAP24_10YR	32.4	520	4.0	50	0.02	0.425	0.08	0.15	25
34_B	Russell & College	MAP24_10YR	48.3	1020	4.0	50	0.02	0.425	0.08	0.15	25
35_A	Parker & Ellsworth	MAP24_10YR	36.8	970	3.0	60	0.02	0.425	0.08	0.8	25
35_B	Channing & Ellsworth	MAP24_10YR	29.6	700	3.0	60	0.02	0.425	0.08	0.15	25
35_C	Parker & Ellsworth	MAP24_10YR	22.9	570	3.0	60	0.02	0.425	0.08	0.15	25
35_D	Telegraph & Dwight	MAP24_10YR	33.2	790	3.0	60	0.02	0.425	0.08	0.15	25
36_A	Woolsey & Dana	MAP22_10YR	31.9	790	4.0	50	0.02	0.425	0.08	0.15	25
36_B	Ashby & Telegraph	MAP22_10YR	32.2	720	4.0	50	0.02	0.425	0.08	0.15	25
36_C	Stuart & Telegraph	MAP22_10YR	37.3	820	4.0	50	0.02	0.425	0.08	0.15	25
37_A	Woolsey & College	MAP26_10YR	40.9	700	3.0	50	0.02	0.425	0.08	0.15	25
37_B	Webster & College	MAP26_10YR	27.6	610	3.0	50	0.02	0.425	0.08	0.15	25

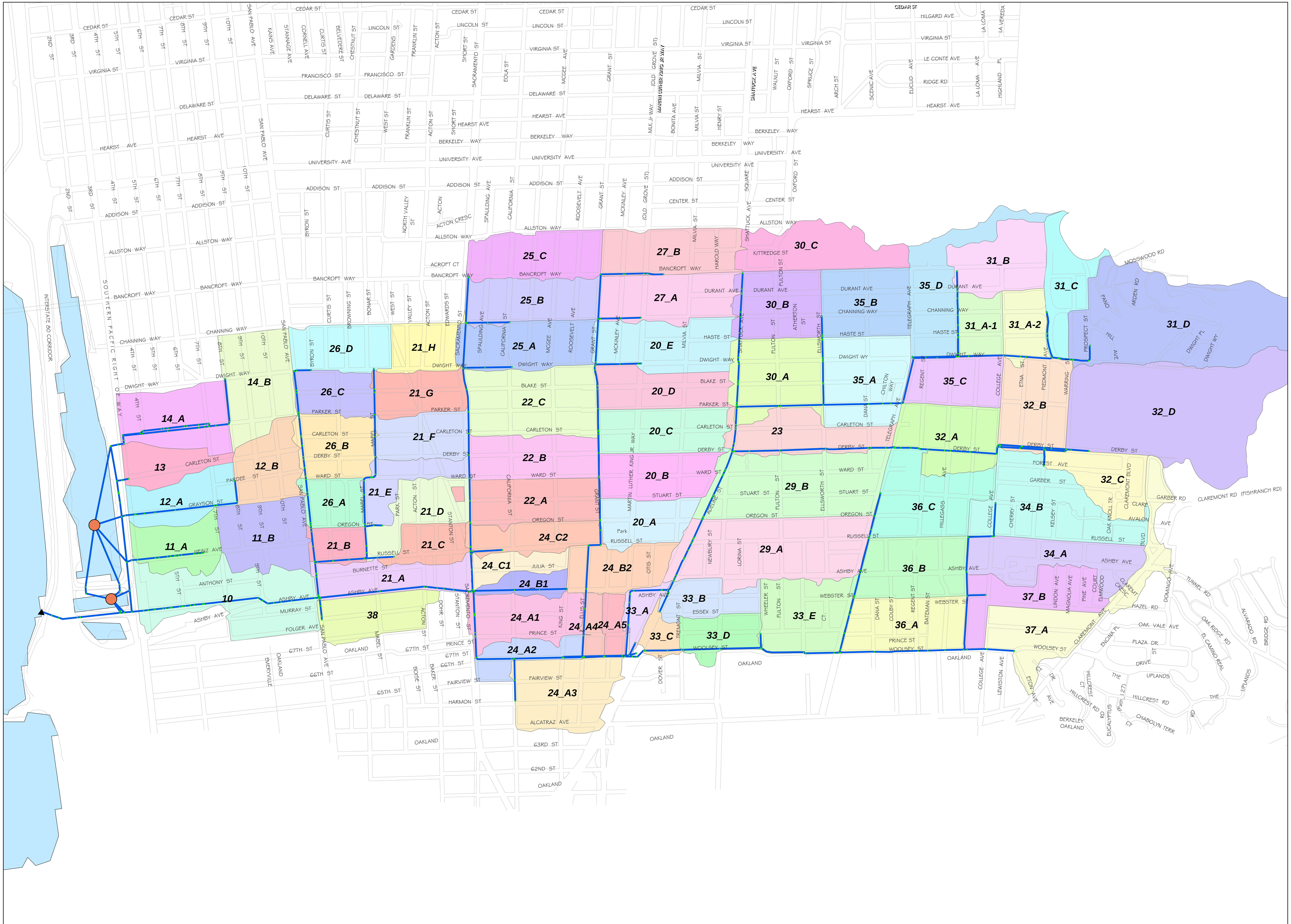

Figure A-1: Potter Catchment Map

0 500 1,000 1,500 2,000
Feet

LEGEND

- Node
- Storage
- Outlet
- Existing Links

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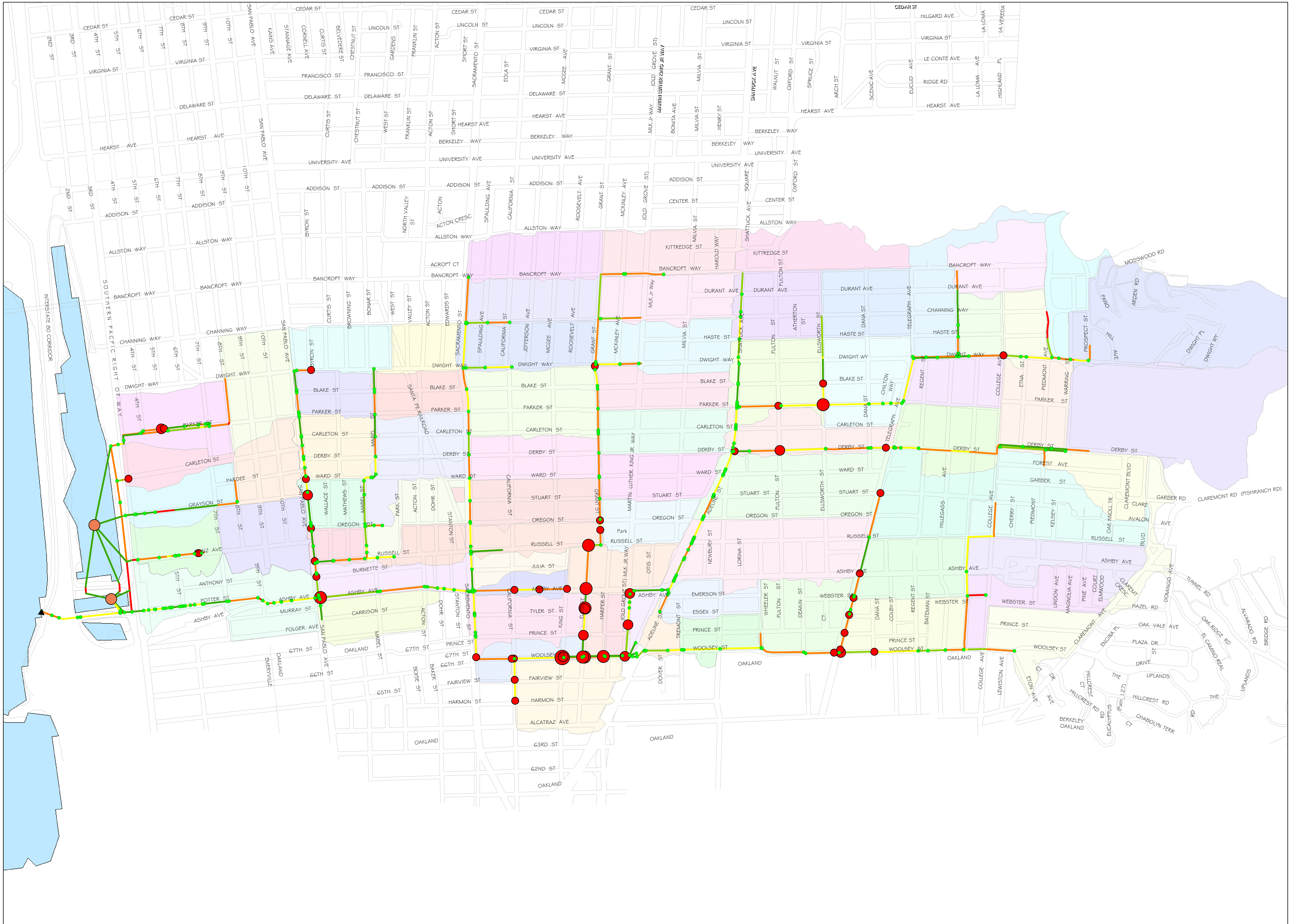

FIGURE B-1:

Potter Existing System Results

N

0 500 1,000 1,500 2,000

Feet

LEGEND

Infrastructure

- Storage
- Outlet

Spilled Volume (AF)

- 0.00 - 0.05
- 0.06 - 0.50
- 0.51 - 1.00
- 1.01 - 3.00
- 3.01 - 5.00

Percent Capacity

- 0.11 - 0.70
- 0.71 - 0.80
- 0.81 - 1.00
- 1.01 - 2.00
- 2.01 - 50.00

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FIGURE D-1:
Potter Green Retrofit System Results

Table E-1: Codornices Creek Watershed Summary Table (Pre-Henry GI Retrofit)

Catchment Name	Rainfall Reference	Area (ac)	Max Flow (cfs)	Total Runoff Depth (in)	Width (ft)	Slope	Impervious Percentage (%)	Total Rainfall (in)
034-700	10YR 6HR 20IN	7.13	9.22	1.66	385.37	0.01	67.20	1.81
034-703	10YR 6HR 20IN	8.63	10.70	1.47	516.32	0.02	57.40	1.81
034-705	10YR 6HR 20IN	10.45	11.83	1.52	344.54	0.02	68.60	1.81
034-110	10YR 6HR 20IN	2.98	4.01	1.64	140.85	0.01	83.20	1.81
ALB-REDO	10YR 6HR 20IN	8.47	8.86	1.42	354.54	0.01	56.10	1.81
Village_in	10YR 6HR 20IN	102.00	85.72	1.49	1600.00	0.01	70.00	1.81
Gilman_in	10YR 6HR 20IN	168.00	141.00	1.68	2000.00	0.01	91.00	1.81
042-712	10YR 6HR 22IN	6.31	9.93	1.69	444.22	0.02	70.10	1.94
ALB-519	10YR 6HR 22IN	1.39	2.12	1.57	120.88	0.04	65.00	1.94
ALB-518	10YR 6HR 22IN	9.04	10.57	1.46	282.25	0.04	57.90	1.94
040-704	10YR 6HR 22IN	14.98	14.23	1.48	357.88	0.01	54.40	1.94
026-004	10YR 6HR 22IN	21.98	25.85	1.45	813.98	0.07	50.20	1.94
026-100	10YR 6HR 22IN	13.03	13.33	1.35	496.14	0.04	46.20	1.94
046-127	10YR 6HR 22IN	18.97	14.94	1.19	448.81	0.04	37.60	1.94
040-708	10YR 6HR 22IN	18.02	19.64	1.43	394.35	0.06	55.60	1.94
044-820	10YR 6HR 22IN	22.62	24.35	1.45	529.47	0.05	52.60	1.94
026-707	10YR 6HR 22IN	19.01	15.59	1.23	402.30	0.03	43.00	1.94
046-122	10YR 6HR 22IN	23.30	23.24	1.40	470.12	0.04	55.60	1.94
ALB-420.3	10YR 6HR 22IN	15.12	19.50	1.53	508.65	0.05	63.50	1.94
ALB-422	10YR 6HR 22IN	5.36	7.67	1.55	282.09	0.07	60.00	1.94
ALB-418.1	10YR 6HR 22IN	22.11	25.74	1.51	547.33	0.04	62.40	1.94
ALB-512	10YR 6HR 22IN	2.38	2.54	1.42	127.57	0.01	52.30	1.94
ALB-509	10YR 6HR 22IN	12.81	12.28	1.50	300.87	0.01	60.70	1.94
044-ALBI2	10YR 6HR 22IN	12.64	13.08	1.36	437.92	0.06	44.10	1.94
028-012	10YR 6HR 24IN	12.15	12.37	1.50	262.99	0.12	36.80	2.11
028-734	10YR 6HR 24IN	22.66	18.67	1.34	382.33	0.07	34.40	2.11
028-008	10YR 6HR 24IN	12.04	14.32	1.53	364.56	0.07	48.80	2.11
028-732	10YR 6HR 24IN	24.45	23.20	1.42	461.62	0.08	39.60	2.11
030-001	10YR 6HR 24IN	3.72	3.60	1.40	253.51	0.21	7.30	2.11
028-823	10YR 6HR 24IN	14.83	11.90	1.37	429.76	0.12	23.60	2.11
028-736	10YR 6HR 24IN	16.24	15.24	1.39	409.98	0.04	40.00	2.11
026-105	10YR 6HR 24IN	12.85	12.84	1.43	448.37	0.05	38.40	2.11
028-002	10YR 6HR 24IN	15.55	13.46	1.38	338.73	0.05	35.00	2.11
028-728	10YR 6HR 24IN	1.66	2.26	1.54	158.25	0.14	25.20	2.11
028-013	10YR 6HR 24IN	14.63	15.10	1.51	433.45	0.14	31.30	2.11
030-120	10YR 6HR 24IN	5.05	6.04	1.54	196.06	0.09	40.50	2.11
028-902	10YR 6HR 24IN	23.08	21.44	1.45	635.36	0.08	31.90	2.11
028-903	10YR 6HR 24IN	17.44	18.09	1.47	425.91	0.04	47.20	2.11
030-728	10YR 6HR 26IN	19.36	7.62	1.05	361.77	0.16	8.50	2.27
030-025	10YR 6HR 26IN	10.93	9.28	1.18	637.09	0.37	7.30	2.27
030-205	10YR 6HR 26IN	13.99	6.76	1.07	427.16	0.15	9.30	2.27
030-711	10YR 6HR 26IN	24.26	13.27	1.08	390.78	0.17	16.60	2.27
030-020	10YR 6HR 26IN	8.40	9.72	1.29	635.32	0.36	15.50	2.27
030-018	10YR 6HR 26IN	14.29	10.17	1.17	612.76	0.28	10.30	2.27
030-015	10YR 6HR 26IN	6.25	4.53	1.20	302.59	0.13	14.90	2.27
030-014	10YR 6HR 26IN	6.11	5.07	1.25	367.16	0.13	16.50	2.27
030-010	10YR 6HR 26IN	25.13	17.92	1.27	662.61	0.18	17.60	2.27
030-043	10YR 6HR 26IN	20.40	14.53	1.19	562.80	0.18	18.60	2.27
030-037	10YR 6HR 26IN	14.88	10.12	1.19	472.70	0.16	16.90	2.27
030-029	10YR 6HR 26IN	1.07	0.82	1.18	78.95	0.14	9.30	2.27
032-147	10YR 6HR 26IN	14.66	10.89	1.17	343.62	0.14	22.60	2.27
032-015	10YR 6HR 26IN	10.36	8.17	1.25	280.19	0.13	24.30	2.27
032-025	10YR 6HR 26IN	22.73	11.08	1.04	382.34	0.15	14.20	2.27
032-031	10YR 6HR 26IN	1.19	1.67	1.35	163.21	0.19	19.30	2.27
032-115	10YR 6HR 26IN	4.67	3.25	1.17	235.74	0.18	10.50	2.27
032-107	10YR 6HR 26IN	28.71	15.32	1.01	455.08	0.13	16.90	2.27
030-749	10YR 6HR 26IN	27.20	14.04	1.05	435.19	0.14	16.10	2.27
032-038	10YR 6HR 26IN	4.83	3.74	1.23	190.42	0.20	16.90	2.27
032-161	10YR 6HR 26IN	1.39	1.45	1.29	113.28	0.18	16.90	2.27
032-162	10YR 6HR 26IN	4.72	3.49	1.22	137.06	0.12	21.70	2.27
032-163	10YR 6HR 26IN	3.74	2.25	1.13	163.54	0.18	8.70	2.27
030-729.1	10YR 6HR 26IN	13.87	6.81	1.13	441.18	0.20	6.00	2.27
030-037.2	10YR 6HR 26IN	4.74	5.31	1.30	472.70	0.16	16.90	2.27

	Ref	Area				MAP	Impervious %	
	20.00	37.66				20.00	332.50	500.00
	22.00	239.07				22.00	931.20	1700.00
	24.00	196.34				24.00	480.00	1400.00
	26.00	307.87				26.00	372.30	2500.00
		780.93						
Weighted MAP =	23.98	(no Gillman & VC)			Weighted Impervious % =	0.3447	(no Gillman & VC)	
	22.96	(with Gillman & VC)				0.4387	(with Gillman & VC)	

Figure E-1: Codornices Creek Catchment Map

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Table E-2: Codornices Creek Watershed Summary Table (Post-Henry GI Retrofit)

Catchment Name	Rainfall Reference	Area (ac)	Max Flow (cfs)	Total Runoff Depth (in)	Width (ft)	Slope	Impervious Percentage (%)	Total Rainfall (in)
034-700	10YR 6HR 20IN	7.13	9.22	1.66	385.37	0.01	67.20	1.81
034-703	10YR 6HR 20IN	8.63	10.70	1.47	516.32	0.02	57.40	1.81
034-705	10YR 6HR 20IN	10.45	11.83	1.52	344.54	0.02	68.60	1.81
034-110	10YR 6HR 20IN	2.98	4.01	1.64	140.85	0.01	83.20	1.81
ALB-REDO	10YR 6HR 20IN	8.47	8.86	1.42	354.54	0.01	56.10	1.81
Village_in	10YR 6HR 20IN	102.00	85.72	1.49	1600.00	0.01	70.00	1.81
Gilman_in	10YR 6HR 20IN	168.00	141.00	1.68	2000.00	0.01	91.00	1.81
042-712	10YR 6HR 22IN	6.31	9.93	1.69	444.22	0.02	70.10	1.94
ALB-519	10YR 6HR 22IN	1.39	2.12	1.57	120.88	0.04	65.00	1.94
ALB-518	10YR 6HR 22IN	9.04	10.57	1.46	282.25	0.04	57.90	1.94
040-704	10YR 6HR 22IN	14.98	14.23	1.48	357.88	0.01	54.40	1.94
026-004	10YR 6HR 22IN	21.98	25.85	1.45	813.98	0.07	50.20	1.94
026-100	10YR 6HR 22IN	13.03	13.33	1.35	496.14	0.04	46.20	1.94
046-127	10YR 6HR 22IN	18.97	14.94	1.19	448.81	0.04	37.60	1.94
040-708	10YR 6HR 22IN	18.02	19.64	1.43	394.35	0.06	55.60	1.94
044-820	10YR 6HR 22IN	22.62	24.35	1.45	529.47	0.05	52.60	1.94
026-707	10YR 6HR 22IN	19.01	15.59	1.23	402.30	0.03	43.00	1.94
046-122	10YR 6HR 22IN	23.30	23.24	1.40	470.12	0.04	55.60	1.94
ALB-420.3	10YR 6HR 22IN	15.12	19.50	1.53	508.65	0.05	63.50	1.94
ALB-422	10YR 6HR 22IN	5.36	7.67	1.55	282.09	0.07	60.00	1.94
ALB-418.1	10YR 6HR 22IN	22.11	25.74	1.51	547.33	0.04	62.40	1.94
ALB-512	10YR 6HR 22IN	2.38	2.54	1.42	127.57	0.01	52.30	1.94
ALB-509	10YR 6HR 22IN	12.81	12.28	1.50	300.87	0.01	60.70	1.94
044-ALBI2	10YR 6HR 22IN	12.64	13.08	1.36	437.92	0.06	44.10	1.94
028-012	10YR 6HR 24IN	6.70	7.93	1.55	262.99	0.12	36.80	2.11
028-734	10YR 6HR 24IN	13.15	12.28	1.40	382.33	0.07	34.40	2.11
028-008	10YR 6HR 24IN	7.57	9.86	1.56	364.56	0.07	48.80	2.11
028-732	10YR 6HR 24IN	12.02	13.38	1.49	461.62	0.08	39.60	2.11
030-001	10YR 6HR 24IN	3.72	3.60	1.40	253.51	0.21	7.30	2.11
028-823	10YR 6HR 24IN	14.83	11.90	1.37	429.76	0.12	23.60	2.11
028-736	10YR 6HR 24IN	16.24	15.24	1.39	409.98	0.04	40.00	2.11
026-105	10YR 6HR 24IN	12.85	12.84	1.43	448.37	0.05	38.40	2.11
028-002	10YR 6HR 24IN	15.55	13.46	1.38	338.73	0.05	35.00	2.11
028-728	10YR 6HR 24IN	1.66	2.26	1.54	158.25	0.14	25.20	2.11
028-013	10YR 6HR 24IN	14.63	15.10	1.51	433.45	0.14	31.30	2.11
030-120	10YR 6HR 24IN	5.05	6.04	1.54	196.06	0.09	40.50	2.11
028-902	10YR 6HR 24IN	23.08	21.44	1.45	635.36	0.08	31.90	2.11
028-903	10YR 6HR 24IN	17.44	18.09	1.47	425.91	0.04	47.20	2.11
Node2226	10YR 6HR 24IN	5.45	6.80	1.57	262.00	0.12	36.80	2.11
Node2228	10YR 6HR 24IN	14.63	15.10	1.51	433.45	0.14	31.30	2.11
Node2229	10YR 6HR 24IN	12.43	13.72	1.49	460.00	0.08	39.60	2.11
Node2238	10YR 6HR 24IN	9.51	9.57	1.44	380.00	0.07	34.40	2.11
Node2239	10YR 6HR 24IN	4.47	6.45	1.58	364.00	0.07	48.80	2.11
030-728	10YR 6HR 26IN	19.36	7.62	1.05	361.77	0.16	8.50	2.27
030-025	10YR 6HR 26IN	10.93	9.28	1.18	637.09	0.37	7.30	2.27
030-205	10YR 6HR 26IN	13.99	6.76	1.07	427.16	0.15	9.30	2.27
030-711	10YR 6HR 26IN	24.26	13.27	1.08	390.78	0.17	16.60	2.27
030-020	10YR 6HR 26IN	8.40	9.72	1.29	635.32	0.36	15.50	2.27
030-018	10YR 6HR 26IN	14.29	10.17	1.17	612.76	0.28	10.30	2.27
030-015	10YR 6HR 26IN	6.25	4.53	1.20	302.59	0.13	14.90	2.27
030-014	10YR 6HR 26IN	6.11	5.07	1.25	367.16	0.13	16.50	2.27
030-010	10YR 6HR 26IN	25.13	17.92	1.27	662.61	0.18	17.60	2.27
030-043	10YR 6HR 26IN	20.40	14.53	1.19	562.80	0.18	18.60	2.27
030-037	10YR 6HR 26IN	14.88	10.12	1.19	472.70	0.16	16.90	2.27
030-029	10YR 6HR 26IN	1.07	0.82	1.18	78.95	0.14	9.30	2.27
032-147	10YR 6HR 26IN	14.66	10.89	1.17	343.62	0.14	22.60	2.27
032-015	10YR 6HR 26IN	10.36	8.17	1.25	280.19	0.13	24.30	2.27
032-025	10YR 6HR 26IN	22.73	11.08	1.04	382.34	0.15	14.20	2.27
032-031	10YR 6HR 26IN	1.19	1.67	1.35	163.21	0.19	19.30	2.27
032-115	10YR 6HR 26IN	4.67	3.25	1.17	235.74	0.18	10.50	2.27
032-107	10YR 6HR 26IN	28.71	15.32	1.01	455.08	0.13	16.90	2.27
030-749	10YR 6HR 26IN	27.20	14.04	1.05	435.19	0.14	16.10	2.27
032-038	10YR 6HR 26IN	4.83	3.74	1.23	190.42	0.20	16.90	2.27
032-161	10YR 6HR 26IN	1.39	1.45	1.29	113.28	0.18	16.90	2.27
032-162	10YR 6HR 26IN	4.72	3.49	1.22	137.06	0.12	21.70	2.27
032-163	10YR 6HR 26IN	3.74	2.25	1.13	163.54	0.18	8.70	2.27
030-729.1	10YR 6HR 26IN	13.87	6.81	1.13	441.18	0.20	6.00	2.27
030-037.2	10YR 6HR 26IN	4.74	5.31	1.30	472.70	0.16	16.90	2.27

MAP	Area
20.00	37.66
22.00	239.07
24.00	210.96
26.00	307.87
	795.56

Weighted MAP = 23.98 (no Gillman & VC)
22.97 (with Gillman & VC)

MAP	Impervious %	
20.00	332.50	500.00
22.00	931.20	1700.00
24.00	670.90	1900.00
26.00	372.30	2500.00

Weighted Impervious = 0.3473 (no Gillman & VC)
0.4394 (with Gillman & VC)

FIGURE F-1:

Codornices Creek Existing Conditions

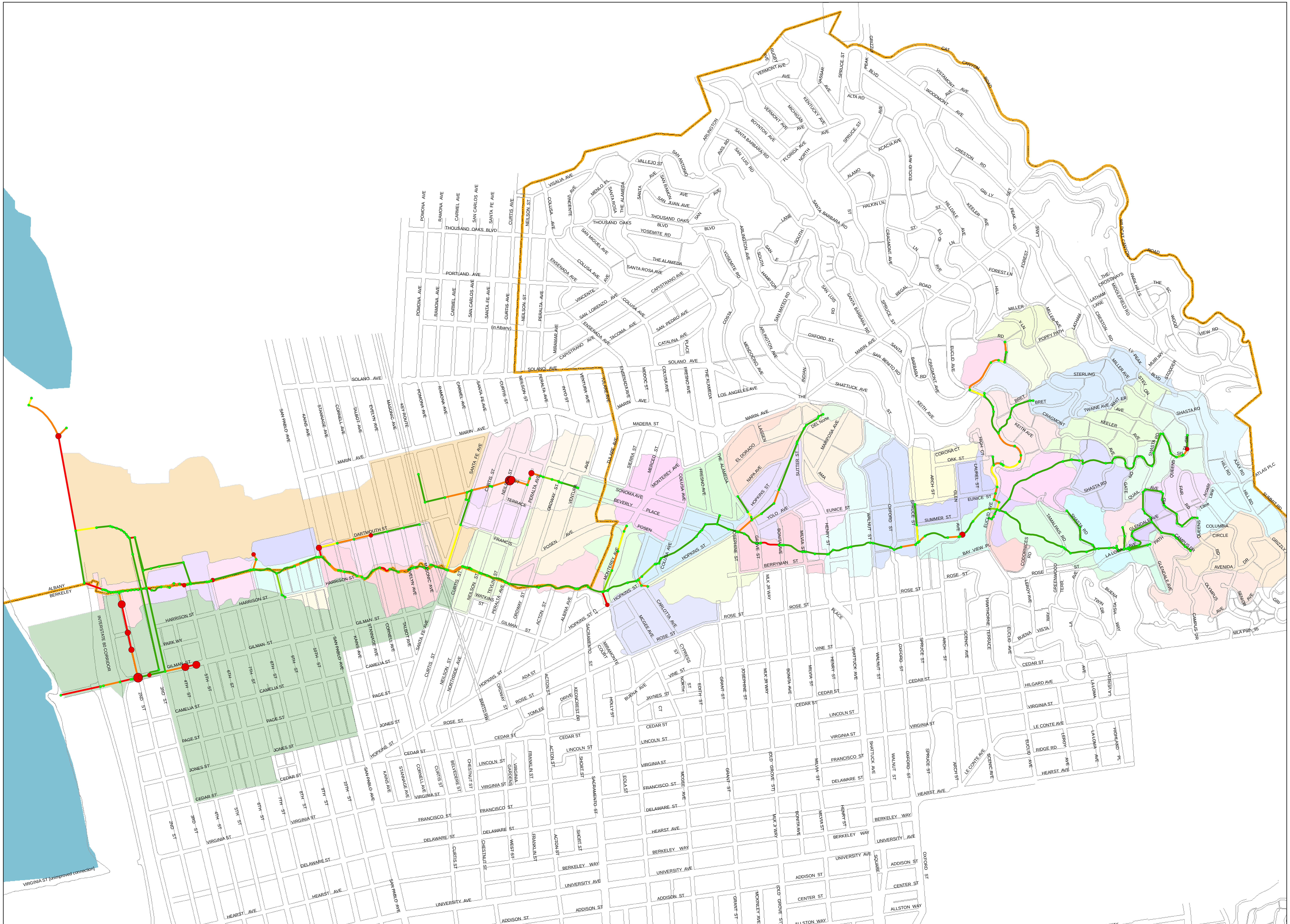

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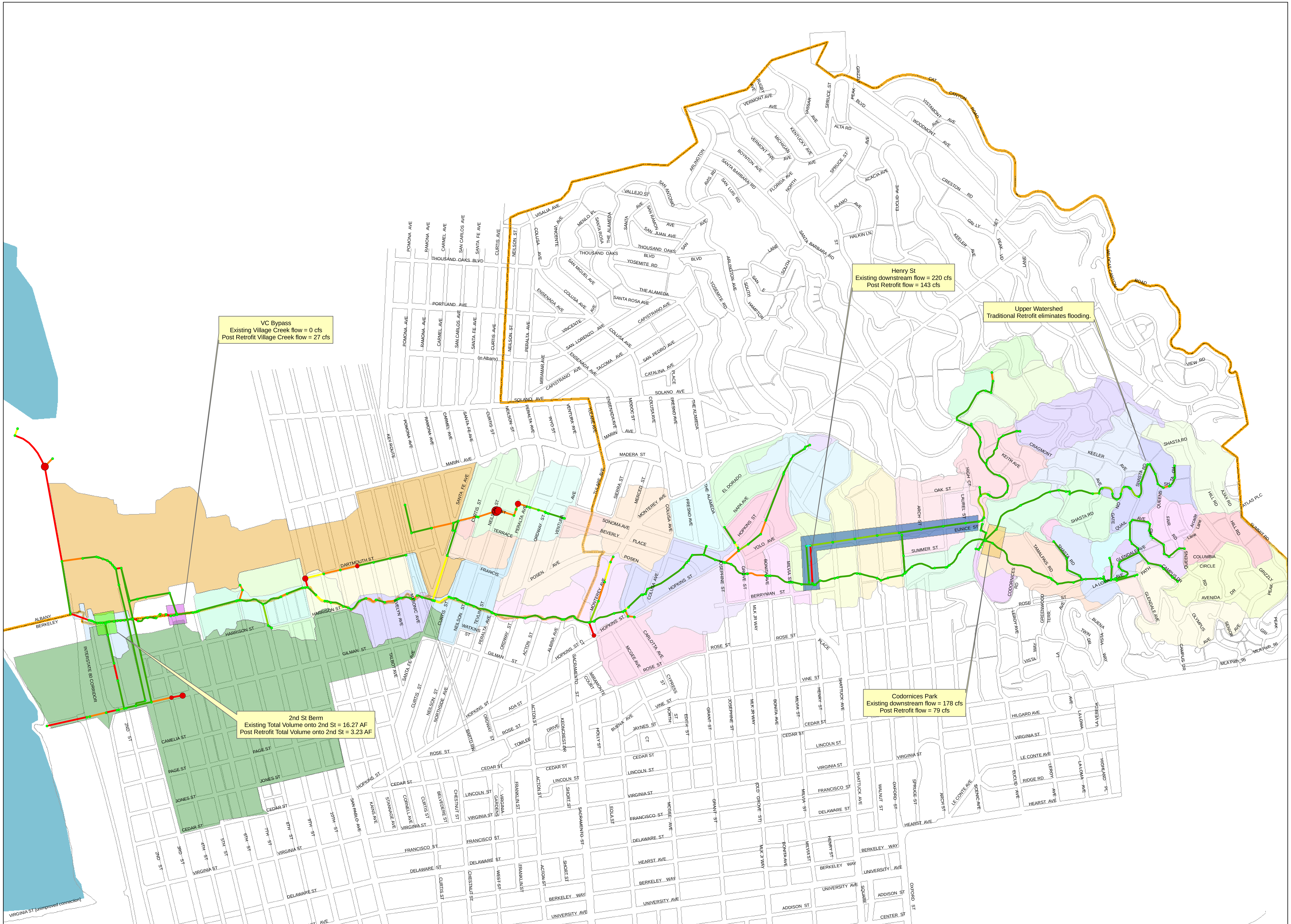
FIGURE H-1:

Codornices Creek Green Retrofit Results



N

0 500 1,000 1,500 2,000

Feet

LEGEND

Berkeley City Limit

Percent Capacity

- 0.00 - 0.70
- 0.71 - 0.80
- 0.81 - 0.90
- 0.91 - 1.50
- 1.51 - 16.50

Volume Flooded (AF)

- 0.00 - 0.05
- 0.06 - 0.25
- 0.26 - 0.50
- 0.51 - 1.00
- 1.01 - 2.00

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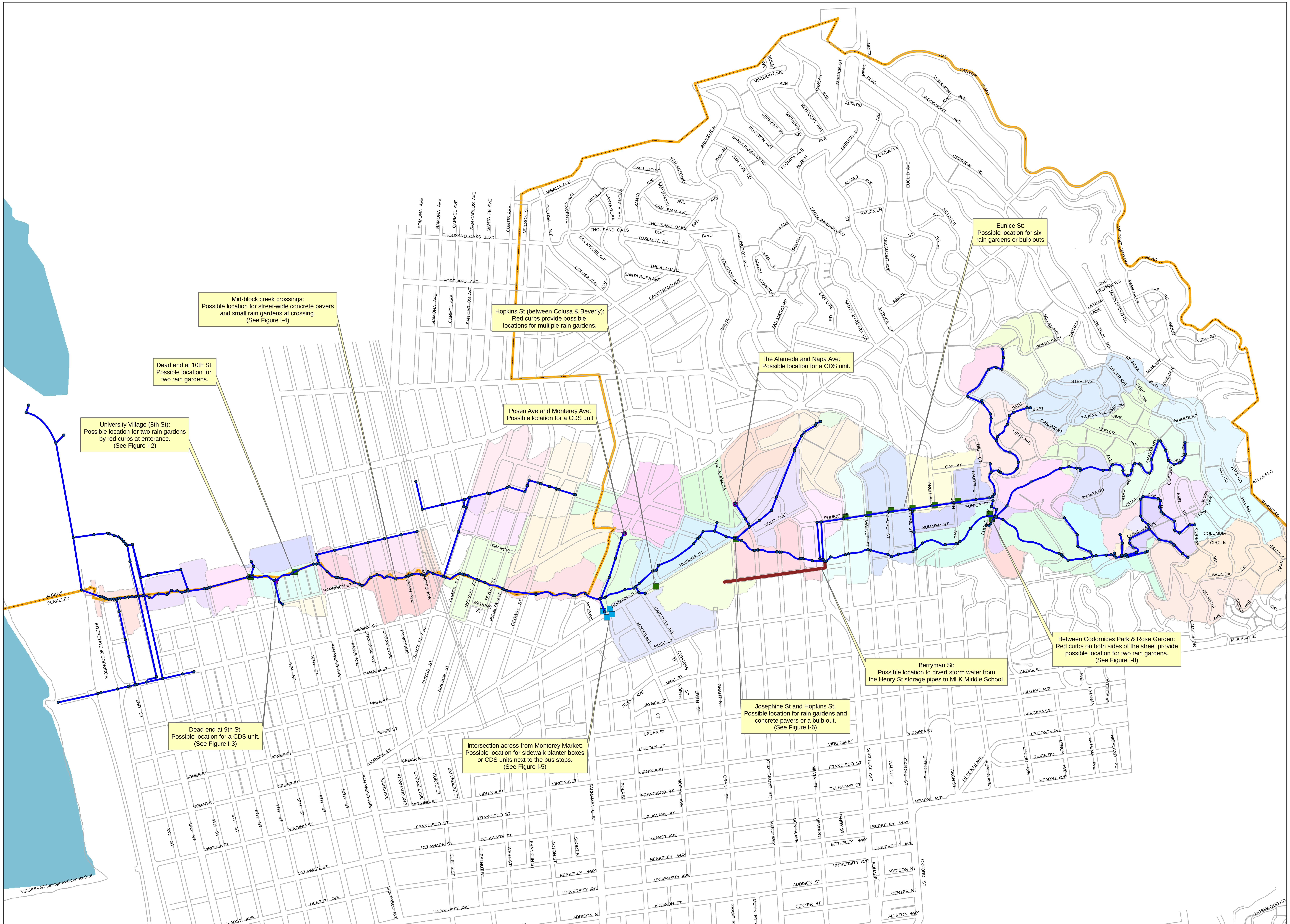

FIGURE I-1:

Green Infrastructure Possibilities

N

0 500 1,000 1,500 2,000

Feet

LEGEND

- Berkeley City Limit
- Potential Rain Garden/ Bulb Out Location
- Potential Sidewalk Planter Box Location
- Potential Continuous Deflective Separation (CDS) Unit Location
- Potential Bermyman St Green Infrastructure Retrofit

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The existing rain gardens at the end of 6th Street provide an attractive entrance for residents, while filtering trash and other debris before storm water enters Codornices Creek, which passes under the street. The narrow lane encourages drivers to slow down without the need for a speed bump.

The entrance to University Village at the end of 10th Street is an ideal location for a similar feature. The rain gardens could filter trash and debris out of the storm water before it enters Codornices Creek, which passes under the street, and the narrow entrance would reduce the need for the speed bump.

Balance Hydrologics, Inc.

Figure I-2: Example of implementing Rain Gardens in the Codornices Creek Lower Watershed.

The dead end at 9th Street is an existing depression that receives a high amount of industrial traffic that requires space to move around when loading/unloading goods. A Continuous Deflective Separation (CDS) unit in this location would help prevent trash and debris from entering the creek while using a minimal foot print to not block the industrial traffic. The main concern with this idea is whether there will be enough elevation difference between the buried CDS unit and the creek.

Balance Hydrologics, Inc.

Figure I-3: Example of implementing a CDS Unit in the Codornices Creek Watershed.

When Codornices Creek passes under streets in the middle of a block, the GI options are more limited. There are few existing mid-block catchments near the creek, and even fewer red curbs (infrastructure would need to be installed and parking spots would need to be used). However, concrete pavers could be placed along with small rain gardens to clean storm water before entering the creek.

Balance Hydrologics, Inc.

Figure I-4: Example of limited options when Codornices Creek passes under streets in the middle of a block.

Across from Monterey Market (at the corner of Hopkins Street and Monterey Street) is a prime example of a location to install sidewalk planter boxes. Where rain gardens would create obstacles for buses and patrons, and concrete pavers would not support the heavy loads, a sidewalk planter box would beautify the area while preventing trash and debris from entering Codornices Creek.

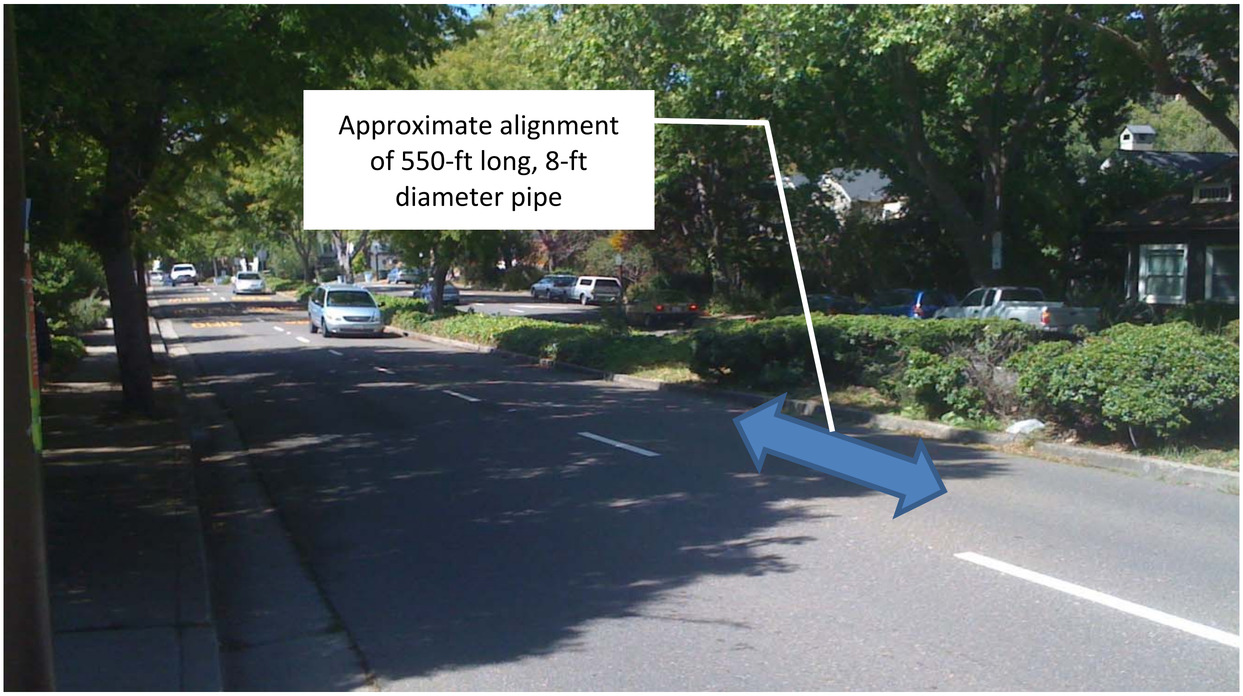
Balance Hydrologics, Inc.

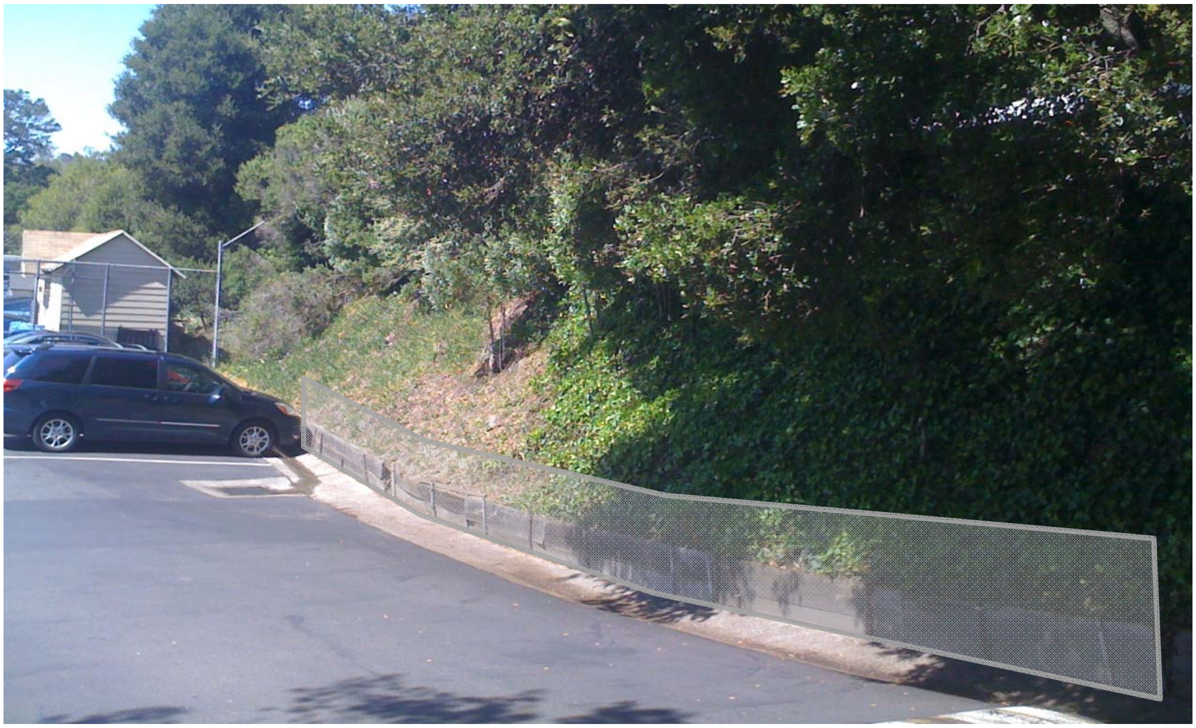
Figure I-5: Example of implementing Sidewalk Planter Boxes in the Codornices Creek Watershed.

The elevated island within the Hopkins and Josephine intersection is not a feasible location for a rain garden. However, the south easterly corner of Josephine and Hopkins is a possible location for a rain garden bulb out or rain gardens with a small area of concrete pavers over the creek alighnment. As an additional benefit, any corner improvement in this location would be visible to the pubilc due to its close proximity to the library.

Balance Hydrologics, Inc.

Figure I-6: Example of implementing a Rain Garden with Concrete Pavers in the Codornices Watershed

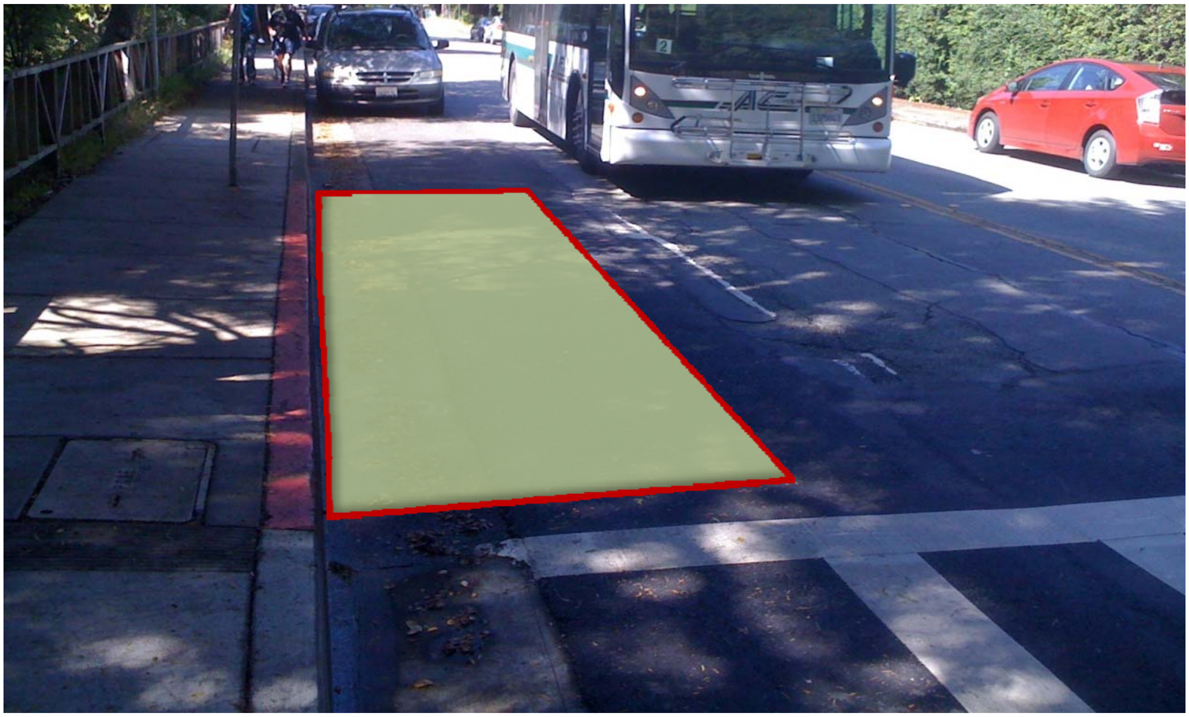




Henry Street is an excellent location to install a major stormwater storage feature. The wide street could contain four 550-ft long barrels, each with 8-ft diameters. The location is relatively flat, meaning excavation costs would be limited. However, the side slope adjacent to the street is steep and may require a new retaining wall to support the street when the barrels are full.

Balance Hydrologics, Inc.

Figure I-7: Example of implementing a large storage feature under a city street.



Euclid Avenue between the entrances to Codornices Park and the Rose Garden has two red curbs that could potentially be used as rain gardens.

Balance Hydrologics, Inc.

Figure I-8: Example of implementing Rain Gardens in the Codornices Creek Upper Watershed.

Balance Hydrologics, Inc.

Figure I-9: Photographs of existing berm at 2nd Street.