Appendix 4.14-3:

Draft Hydrological Analysis and Preliminary Stormwater Management Plan for the Greentree Project

DRAFT Hydrologic Analysis and Preliminary Storm-water Management Plan for the Green Tree project, City of Vacaville

Prepared for:

Green Tree Development Group, Inc.

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GREEN TREE
DEVELOPMENT GROUP

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1 INTRODUCTION

1.1 Overview

This report analyzes and summarizes issues and measures related to managing hydrologic and drainage issues, including stormwater runoff, water quality, and groundwater, related to the proposed Green Tree Project ("Project") in the City of Vacaville, Solano County, California. To mitigate potential impacts and protect the functions and values of receiving waters, stormwater management has come to encompass diverse aspects including water quality and peak flow conveyance and control. Development of the Project area will directly address these various issues with on-site measures and infrastructure. For this reason, a thorough understanding of the constraints related to hydrology and drainage at the site is important in identifying the most effective and feasible means of achieving all hydrologic and stormwater management goals.

This report presents the Hydrologic Analysis and Stormwater Management Plan (HASWMP) for the site with special emphasis on stormwater peak flow management and water-quality treatment. That said, the need to consider all relevant hydrologic impacts means that the study area covered by this HASWMP includes nearby areas which currently route stormwater runoff through the existing Green Tree Golf Course. The document begins by summarizing the hydrologic setting of the Project including factors affecting runoff generation as well as drainage patterns and proposed changes to existing drainage. This information is then used as the basis for hydrologic modeling to assess the anticipated magnitude and timing of peak flows for both existing and proposed conditions at the site. The report also details the water-quality and detention volumes required to meet the stormwater runoff requirements set forth by the City of Vacaville. Other hydrologic factors are also discussed, including groundwater resources and possible hydrologic effects of changing climates.

1.2 Purpose

This document represents the HASWMP for the Green Tree Project. It explains anticipated changes to site hydrology and discusses the proposed strategy for addressing stormwater quantity and quality issues.

Objectives of the HASWMP are summarized as follows:

- Identify the hydrologic opportunities and constrains within the study area that frame the approach to drainage management at the site;
- Provide runoff water-quality with the goal of protecting the beneficial uses of the receiving waters;
- Provide sufficient stormwater conveyance and detention capacity on-site to meet the requirement that peak flow rates for the 10-, and 100-year design storms are not increased in Old Ulatis Creek, Ulatis Creek, or Horse Creek, consistent with City design standards;
- Assure that potential impacts due to hydrograph modification are addressed consistent with City and Regional Water Quality Control Board standards;
- Summarize pre- and post-project consumptive water use data to characterize potential impacts on water supply; and
- Review potential impacts to the site hydrology due to climate change and assure that proposed mitigation measures take any such impacts into account.

The HASWMP is meant to accompany the Specific Plan, as well as address the potential hydrologic and water quality impacts of the Project. Appropriate best management practices (BMPs) are identified along with preliminary design parameters.

2 HYDROLOGIC SETTING

2.1 Geographic Location

The Project proposes to change land use on approximately 180 acres in the northeastern portion of the City of Vacaville in Solano County, California (Figure 2-1). For 55 years, the Project site was the location of the Green Tree Golf Course, which closed operation in February 2016.

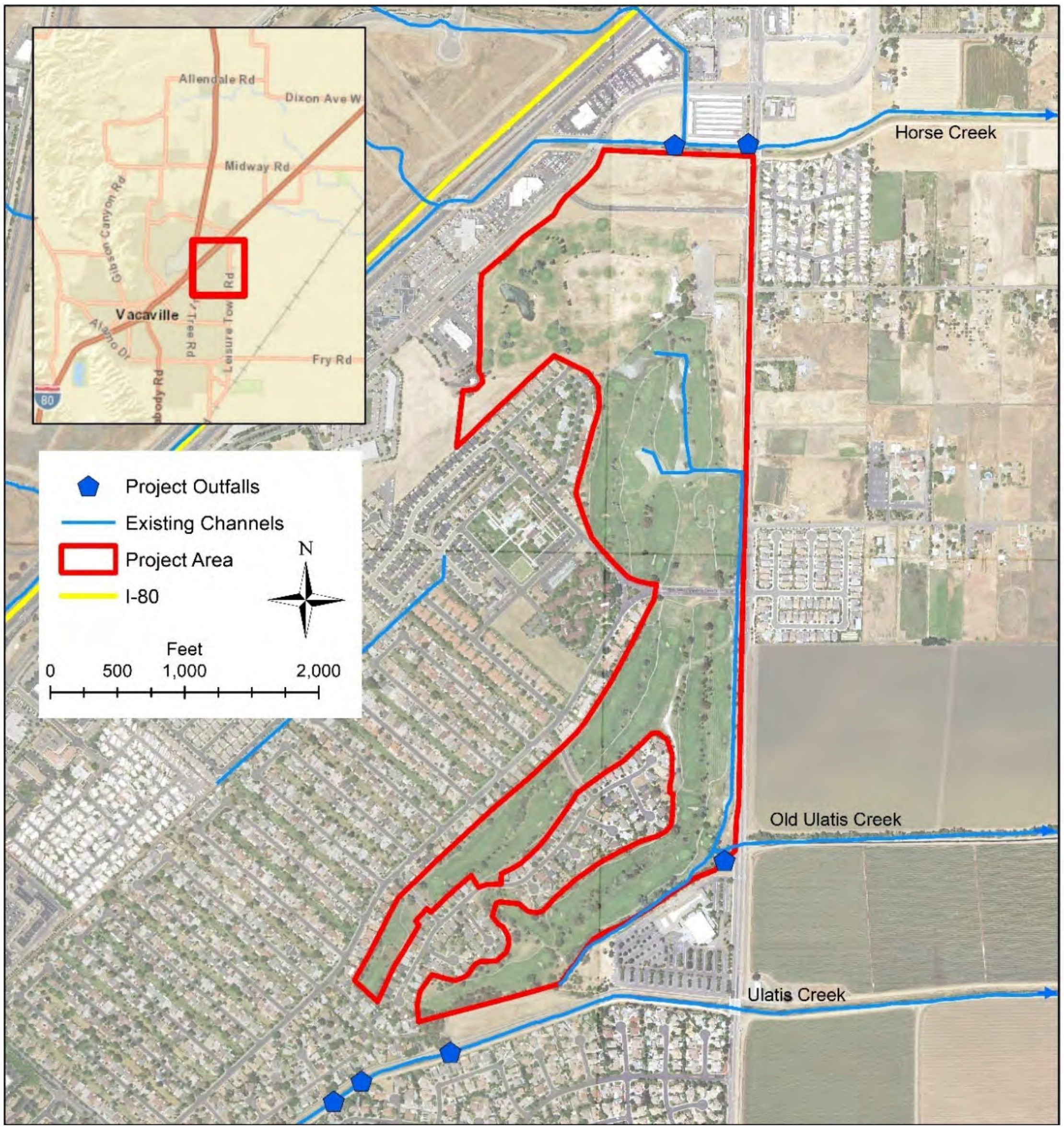

Figure 2-1 Green Tree Project location map.

The Project site is located on the northeastern side of the City of Vacaville and is bordered by Leisure Town Road to the east and by Orange Drive to the northwest. An Auto Mall parallels Interstate 80 on the west side of Orange Drive. Existing residential neighborhoods are located to the west and south of the Project.

2.2 Land Use

2.2.1 Historical and Existing Land Use

The Project site was historically the location of the Green Tree Golf Course and therefore covered by irrigated turf. The only significant tree cover is limited to narrow strips along a series of long and narrow drainage ponds and at the edges of the golf green areas. Prior to the opening of the golf course in the early 1960's, the area was primarily used for agricultural purposes.

2.2.2 Proposed Land Use

The proposed development is designed to meet the growing desire for neighborhoods that are walkable, and navigable via public transportation or bicycle. Proposed housing units are not restricted to single-family homes only but includes multi-family apartment buildings and age-restricted housing for seniors. The Project plans to develop approximately 180 acres, constructing approximately 1,110 residential units ranging from low-density single-family homes in southern area, and medium- to high-density residential homes in the north. The project proposes a total of 20 acres of commercial development, and 10.5 acres of public park. The final unit count and commercial building layouts are still under consideration. The proposed project incorporates walking trails, parks, community gardens, public squares, stormwater management facilities, and maintaining required buffer areas between proposed and existing developments.

2.3 Climate

Climate at the site can be characterized as Mediterranean, with cool, wet winters and hot, dry summers typical of the transition zone from the coastal mountains to the Sacramento Valley. Monthly temperatures range from lows in the 40s and highs in the 50s in the winter, and lows in the 60s and highs in the 100s during the summer. Mean annual precipitation is approximately 25 inches across the Project site (Figure 2-2, SCWA, 1999).

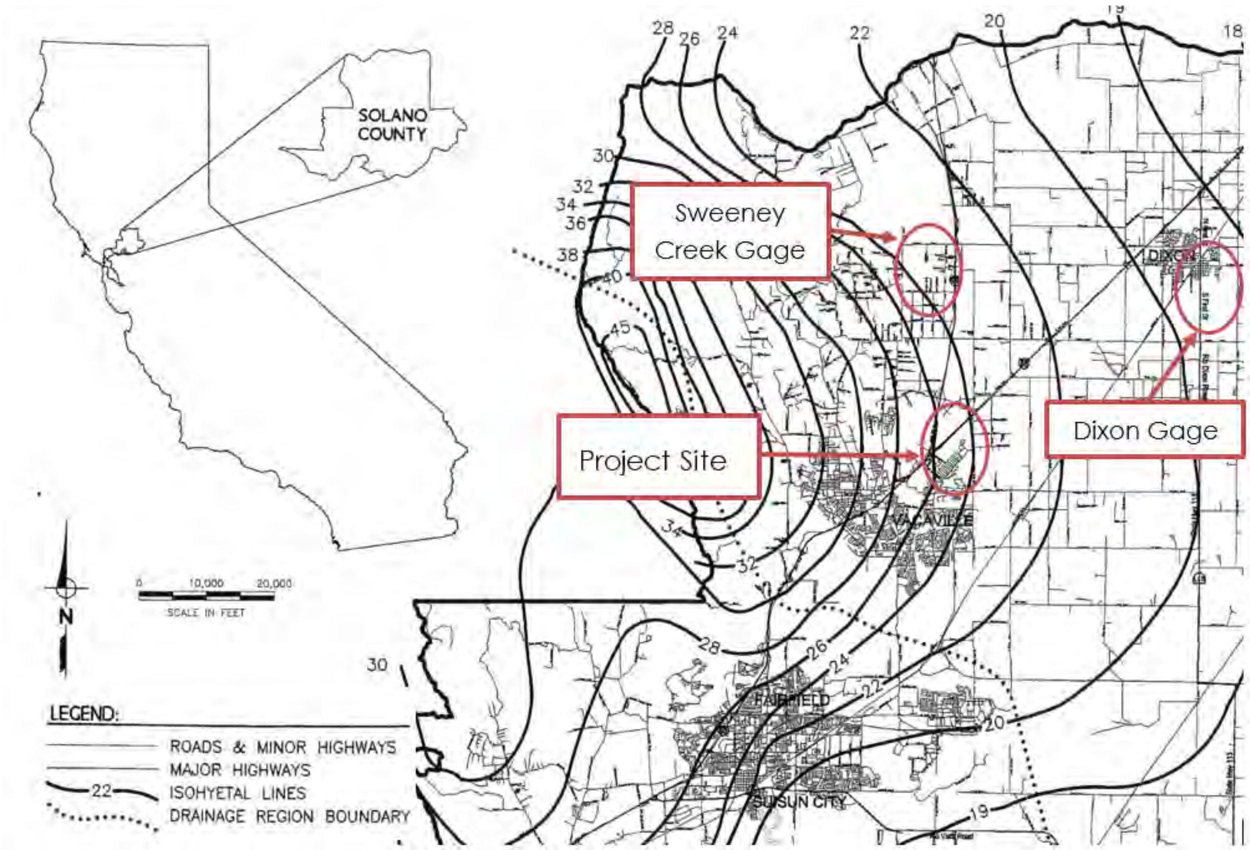

Figure 2-2 Isohyetal map. Excerpt from Solano County Water Agency Hydrology Manual, 1999 (Figure 2-2 in the original document). The locations of the Dixon CIMIS and Solano County Sweeney Creek rain gages highlighted, as well as the project site, in red.

Average monthly reference evapotranspiration (ET) and precipitation at the CIMIS gage in Dixon, California (CIMIS gage 121) are shown in Table 2-1[1]. Reference ET is typically high in the warm summer months; precipitation typically only exceeds ET in the winter months of December, January, and February. The CIMIS gage has a period of record

[1] The average annual precipitation for Dixon gage is somewhat lower than indicated by the isohyets in Figure 2-2, but includes the most recent drought and does not include water year 2017, which was a very wet year.

beginning September 1994. Data included in Table 2-1 are only complete water years[2], from October 1994 through September 2016.

Table 2-1 Average monthly reference ET and precipitation, Dixon, California. Data averaged from October 1994 through September 2016. Data from CIMIS gage 121.

Month	Avg Monthly Reference ET	Average Monthly Precip, Dixon
	(in)	*(in)*
January	1.2	3.7
February	2.0	3.4
March	3.7	2.4
April	5.3	1.1
May	6.9	0.5
June	7.9	0.1
July	8.3	0.0
August	7.3	0.0
September	5.6	0.1
October	4.0	0.6
November	1.9	1.8
December	1.2	4.0
Total	55.3	17.8

Monthly averages calculated for period of record for Dixon, CA CIMIS weather station from Oct 1994 - Sep 2016

2.4 Topography and Soils

The Project site is primarily underlain by Holocene alluvial and floodplain deposits, made up of gravel, sand, silt, and clay (Graymer et al., 2002). A larger area (Figure 2-3, overall watershed) than just the Project (Figure 2-3, Project area) is examined herein to understand the origin and magnitude of flows that may directly affect the Project area. The topography of the overall watershed is very flat and slopes from west to east; surface slopes of the Project area range from 0.5% to 2%.

[2] A water year begins on October 1st and ends the following on September 30th of the named year.

Soils within the overall watershed are predominately characterized as clay or silt loams, with almost all soils classified in either Hydrologic Soil Groups (HSG)[3] B or C (Figure 2-4). The soils in the southernmost portion of the Project are identified as in HSG B, while all other Project soils are identified as in HSG C. Soil types in HSG B have moderate infiltration rates and relatively low runoff potential whereas HSG C types have low infiltration rates and relatively higher runoff potential.

[3] The Natural Resources Conservation Service classifies soils into hydrologic soil groups based on overall runoff potential. The lowest runoff and highest infiltration rate soils are in HSG A, while those with highest runoff potential and lowest infiltration rates are in HSG D.

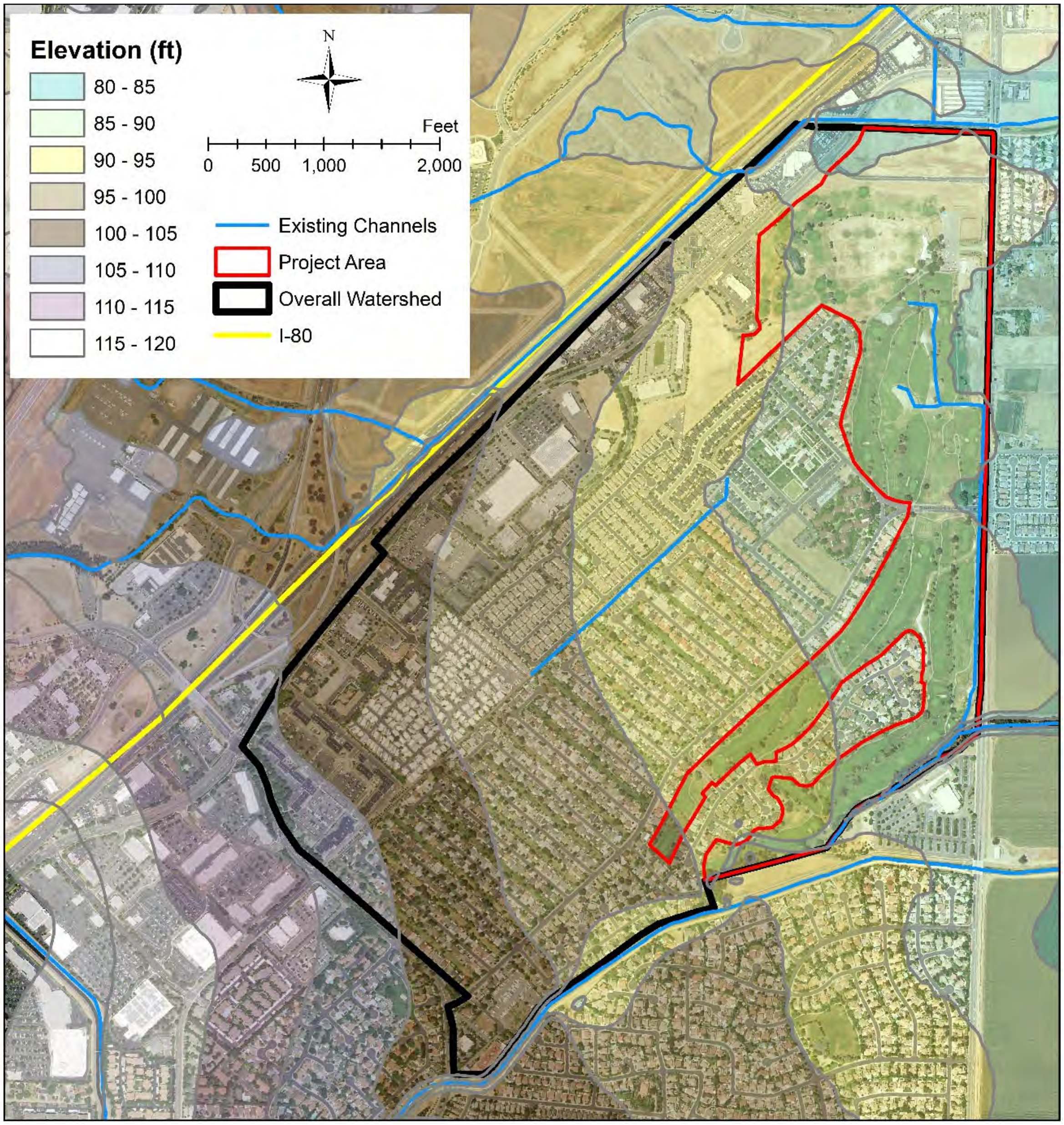

Figure 2-3 Topography of the Project area and overall study watershed. Unless otherwise noted, all elevations cited herein reference the North American Vertical Datum of 1988 (NAVD 88).

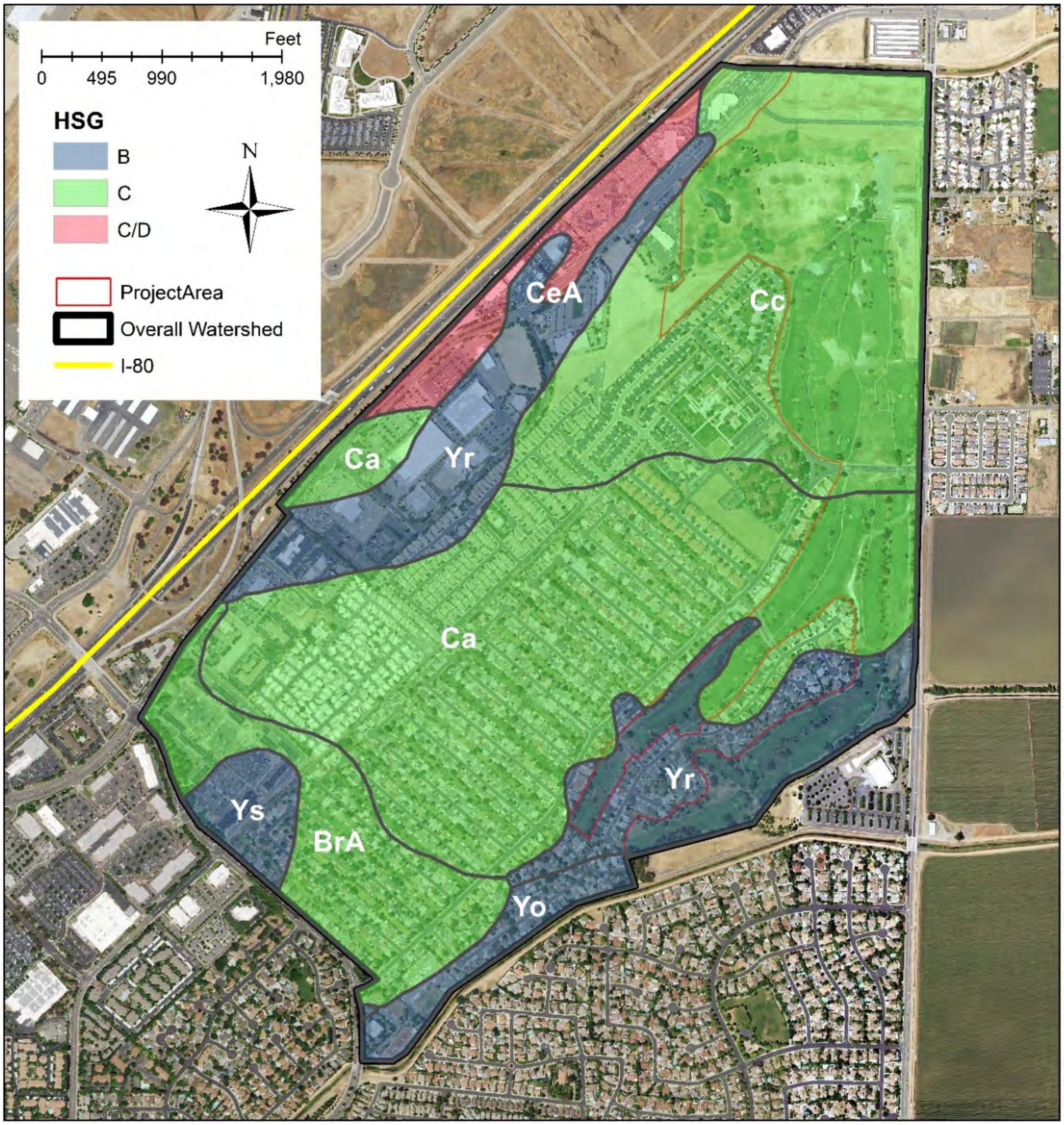

Figure 2-4 Soil map for the study watershed. Data sourced from USDA Web Soil Survey on November 16, 2016.

Soils have a natural infiltration capacity, which can be approximated as a measure of the rate at which water can infiltrate into wetted soils, in units of depth over time (e.g. inches/hour). Infiltration rates are a function of physical properties, including soil particle size and how easily water can move within a given soil column. Soils classified as HSG D will likely have a low infiltration rate, although rates can vary. Variances between soils types and infiltration rates in the overall watershed are presented in Table 2-2, which details the hydrologic soil group (HSG), available water capacity (AWC), and saturated hydraulic conductivity (Ksat) for the most restrictive soil layer for each of the soil types in Figure 2-4.

Table 2-2 Project area soil types and key characteristics. Data sourced from USDA Web Soil Survey on November 16, 2016.

Map Unit Symbol	Map Unit Name	HSG	Available Water Capacity[1]	Min Saturated Hydraulic Conductivity (Ksat)[2]
			(in/in)	*(in/hr)*
BrA	Brentwood clay loam, 0 to 2 percent slopes	C	0.18	0.2 - 0.6
CA	Capay silty clay loam	C	0.17	0.06 - 0.2
Cc	Capay clay	C	0.16	0.06 - 0.2
CeA	Clear Lake clay, 0 to 2 percent slopes, MLRA 17	C/D	0.14	0.06 - 0.2
Yo	Yolo loam, 0 to 4 percent slopes, MLRA 17	B	0.18	0.6 - 2.0
Yr	Yolo loam, clay substratum	B	0.17	0.06 - 0.2
Ys	Yolo silty clay loam, 0 to 2 percent slopes, MLRA 17	B	0.18	0.2 - 0.6

[1] *Available water capacity (AWC) is the amount of water the soil is capable of storing for use by plants*

[2] *Ksat is the rate at which soils can transmit water; listed values from the most restrictive layer in soil column*

2.5 Existing Drainage Features and Patterns

2.5.1 Regional Drainage and Runoff

The Project site is located in the Sacramento River watershed, and creeks in the City of Vacaville generally drain in a southeasterly direction. Major creeks in the area include Alamo Creek, Gibson Canyon Creek, Horse Creek, and Ulatis Creek, all of which drain into the Sacramento River via Cache Slough. The Project receiving waters include Horse Creek, Ulatis Creek, and Old Ulatis Creek, which are summarized in Table 2-3. The Project site is bounded to the south by Old Ulatis Creek, which joins Horse Creek approximately 2 miles downstream of Leisure Town Road, which is the eastern boundary of the project site. The headwaters of Old Ulatis Creek are located directly adjacent to the Project site and nearly all runoff that enters Old Ulatis Creek upstream of Leisure Town Road either originates from the Project site or is routed via overland flow from the adjacent neighborhood during very large storm events. The southern-most tip of the Project site drains directly into Ulatis Creek. The Project site is also bounded by Horse Creek to the north, which flows from west to east. Horse Creek joins with Ulatis Creek approximately 3 miles downstream of Leisure Town Road.

Table 2-3 Receiving Waters Hydraulic Parameters. Data sourced from FEMA, 2016.

	Drainage Area	Peak Discharges		
		10-year	50-year	100-year
	(sq mi)	(cfs)	(cfs)	(cfs)
Horse Creek @ Leisure Town Rd	7.8	2,200	2,700	2,700 [2]
Ulatis Creek @ Leisure Town Rd	16.6	2,700	2,800	2,800 [2]
Old Ulatis Creek @ Leisure Town Rd	0.6 [1]	-	-	-

[1] *drainage area estimated from subwatershed sizes*

[2] *100-year peak discharges are the same as 50-year discharges due to overbank losses*

Horse Creek between Orange Drive and Leisure Town Road, and adjacent to the Project site, is an engineered flood control channel. A weir drop structure in this section slows water velocity and ponds water upstream of the structure (Figure 2-5). An 84” culvert enters Horse Creek between Orange Drive and Leisure Town Road, downstream of the drop structure. This culvert drains the area northwest of the loop streets, between Nut Tree Road and Horse Creek, but does not include the proposed development areas.

2.5.2 Local Drainage and Runoff

The primary pre-project mechanism for collection and conveyance of stormwater through the Project site is a series of inter-connected ponds (Figure 2-6,Table 2-4) that handle both on-site runoff and that originating in the substantial off-site areas to the west. These ponds were constructed as part of the golf course facilities. Runoff from the existing residential area surrounding Grand Canyon Drive collects in a gravel-lined drainage ditch that eventually drains into a feature referred to herein as Pond 1, which drains into Pond 2[4]. Runoff from the areas around Monterey Drive and the Leisure Town Center drain directly into Pond 3. Both Pond 2 and Pond 3 drain into Pond 4, which drains into a ditch

[4] The pond numbering convention adopted herein generally follows the drainage pathways and is not necessarily consistent with past drainage studies in the area.

that runs south along Leisure Town Road on the eastern extent of the Project. This ditch is referred to as Pond 5 as it can store substantial amounts of runoff during storm events. A higher-elevation drainage ditch exits Pond 4 during moderate to high flow events and flows North along Leisure Town Road and into Horse Creek. Outflow from Pond 5 crosses under Sequoia Drive and flows into Pond 7, which is a narrow, channel-like pond. Pond 6 is adjacent to the intersection of Yellowstone and Rushmore Drives and collects runoff from the nearby "Loop Streets" neighborhood (Carlsbad Circle, etc.). Both Pond 6 and Pond 7 drain into the terminal Pond 8. Pond 8 drains into Old Ulatis Creek via three culverts and, during large storm events, an overflow spillway. Finally, the farthest south pond is referred to as Little Pond, which receives runoff from nearby existing residential areas and drains directly into a narrow slot channel that provides detention upstream of the outfall into Ulatis Creek. This channel/detention area is named "City Pond" as it's on City property. The outfall for the City Pond is located on Ulatis Creek just upstream of the large off-line detention basin constructed to attenuate flows in the Ulatis Creek channel.

Figure 2-5 Horse Creek looking east and downstream from Orange Drive toward the drop structure.

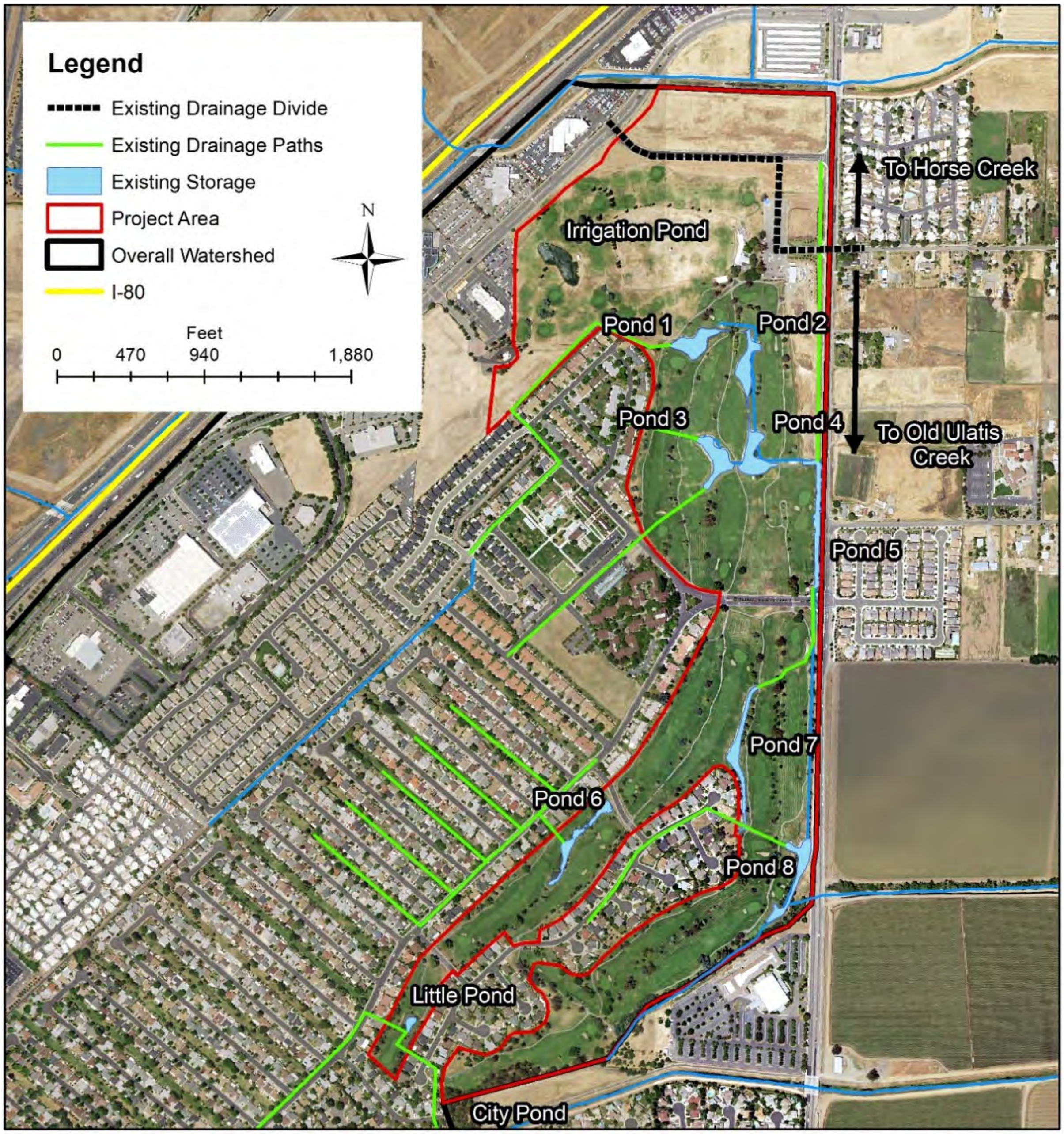

Figure 2-6 Existing drainage features that discharge through the Project site. Areas south of the dashed black line currently drain to Old Ulatis Creek via the pond network.

Table 2-4 Active Storage of Existing Ponds. Active storage calculated from outlet elevation to top of bank.

Pond Name	Active Storage *(ac-ft)*	Drains Into
Pond 1	3.2	Pond 2
Pond 2	3.1	Pond 4
Pond 3	6.7	Pond 4
Pond 4	4.5	Pond 5
Pond 5	3.5	Pond 7
Pond 6	2.2	Pond 7
Pond 7	4.9	Pond 8
Pond 8	5.3	Old Ulatis Creek
Little Pond	1.3	Ulatis Creek
Total	**34.6**	

An additional pond in the northwest-most corner of the Project site and adjacent to Orange Drive was used for storing groundwater pumped for on-site for irrigation. Water from this pond can be released into the series of ponds for irrigation purposes but is not typically connected and therefore does not impact the stormwater routing. This irrigation pond was therefore not included in the hydrologic modeling.

The area north of Gilley Way drains via overland flow either into the northern part of the Leisure Town Road Ditch to Horse Creek, or directly into Horse Creek.

The overall watershed, with a total area of 723 acres (1.1 square miles), encompasses the entire proposed Project area as well as the surrounding neighborhoods (Figure 2-3). The local neighborhood has existing stormwater drainage facilities that were built to drain directly into the golf course ponds (Figure 2-6). Stormwater runoff can enter the Project area and the ponds in the form of overland flow from these same neighborhoods under certain conditions. Runoff from the neighborhoods will eventually drain into Old Ulatis Creek to the south and the remaining runoff will drain to Horse Creek to the north (see existing drainage divide, Figure 2-6).

2.6 Proposed Drainage Features and Patterns

The proposed drainage features will continue to route stormwater to Horse Creek, Ulatis Creek, and Old Ulatis Creek, including off-site runoff and the anticipated increased quantity of runoff from within the Project site. The proposed drainage patterns will serve two purposes, reducing the total infrastructure distance required to drain stormwater runoff, and reducing existing flooding hazards that occur because of low slopes and existing infrastructure across the Project site. As a result, the relative proportion of stormwater volume released into Horse Creek will increase compared with Old Ulatis Creek, but stormwater detention will be designed such that peak flows will not increase in any of the three receiving waters. Figure 2-7 shows the proposed detention basin locations and drainage patterns.

Important proposed drainage features include the use of multi-function stormwater basins, which will play a key role in managing runoff water quality and quantity. Stormwater basins will be integrated with park and open space areas using naturalized contouring and landscaping where appropriate. Stormwater basins will be designed as "dry" basins to minimize vector control (e.g., mosquito) concerns. Given the elevation constrains at the project site and the proposed size of the detention basins, we propose to manage ponding in the detention basins with a central low-flow channel with a minimum slope of 0.35 percent, and a basin cross-slope to keep low-flows in the low-flow channel. On-site water quality will be mitigated using both combination bioretention/detention basins where elevation constraints permit, and local bioretention features such as bioswales or rain gardens where necessary.

2.7 FEMA Floodplain Information

The lower-lying areas along the Project site adjacent to Horse Creek have been mapped by FEMA as Special Flood Hazard Areas, Zones AE and A, commonly referred to as "100-year floodplains".

Figure 2-8 shows these areas as depicted on the Flood Insurance Rate Maps (FIRMs) for the area. This mapping indicates that approximately 23 acres of the proposed Project site lies within the estimated 100-year floodplain.

The currently effective Flood Insurance Study (FEMA, 2009) included detailed hydraulic analyses of flood conditions for the northwest side of I-80, whereas areas subject to inundation in a 100-year flood in the Project site were determined using approximate methods, which is another common FEMA practice. The approximated hazard areas within the Project boundary are defined as Zone A, which may have been estimated under the assumption that I-80 could be overtopped during a 100-year flood. The extent of the 100-year floodplain suggests that all other reaches of Horse Creek downstream of the project site do not overtop their banks during 100-year flood event[5].

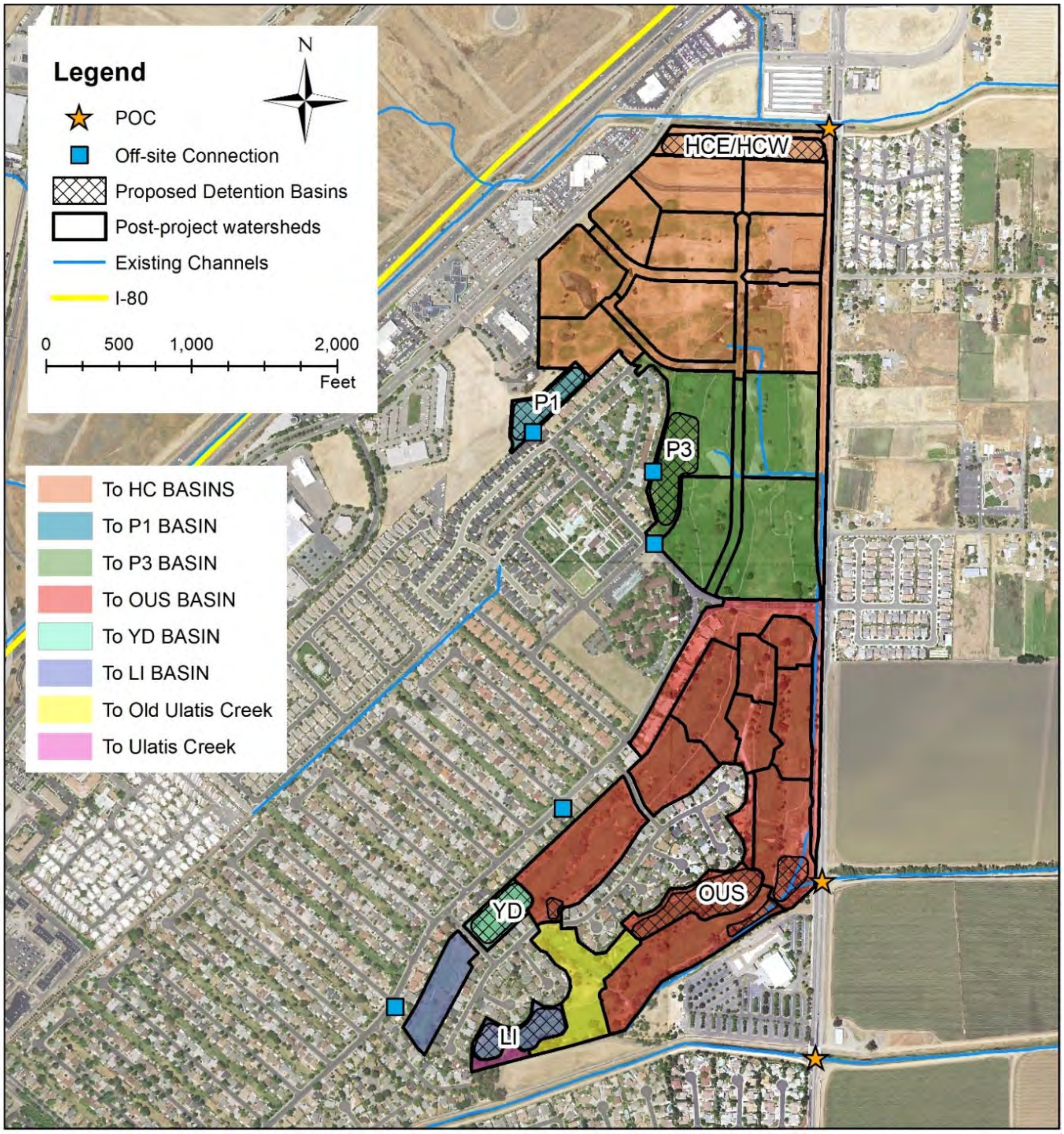

Figure 2-7 Proposed drainage patterns and new detention basins.

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[5] While the FEMA FIRM shows that Horse Creek will not overtop its banks downstream of the project site, this does not confirm that capacity, defined by freeboard requirements in the City of Vacaville Design Standards, is met.

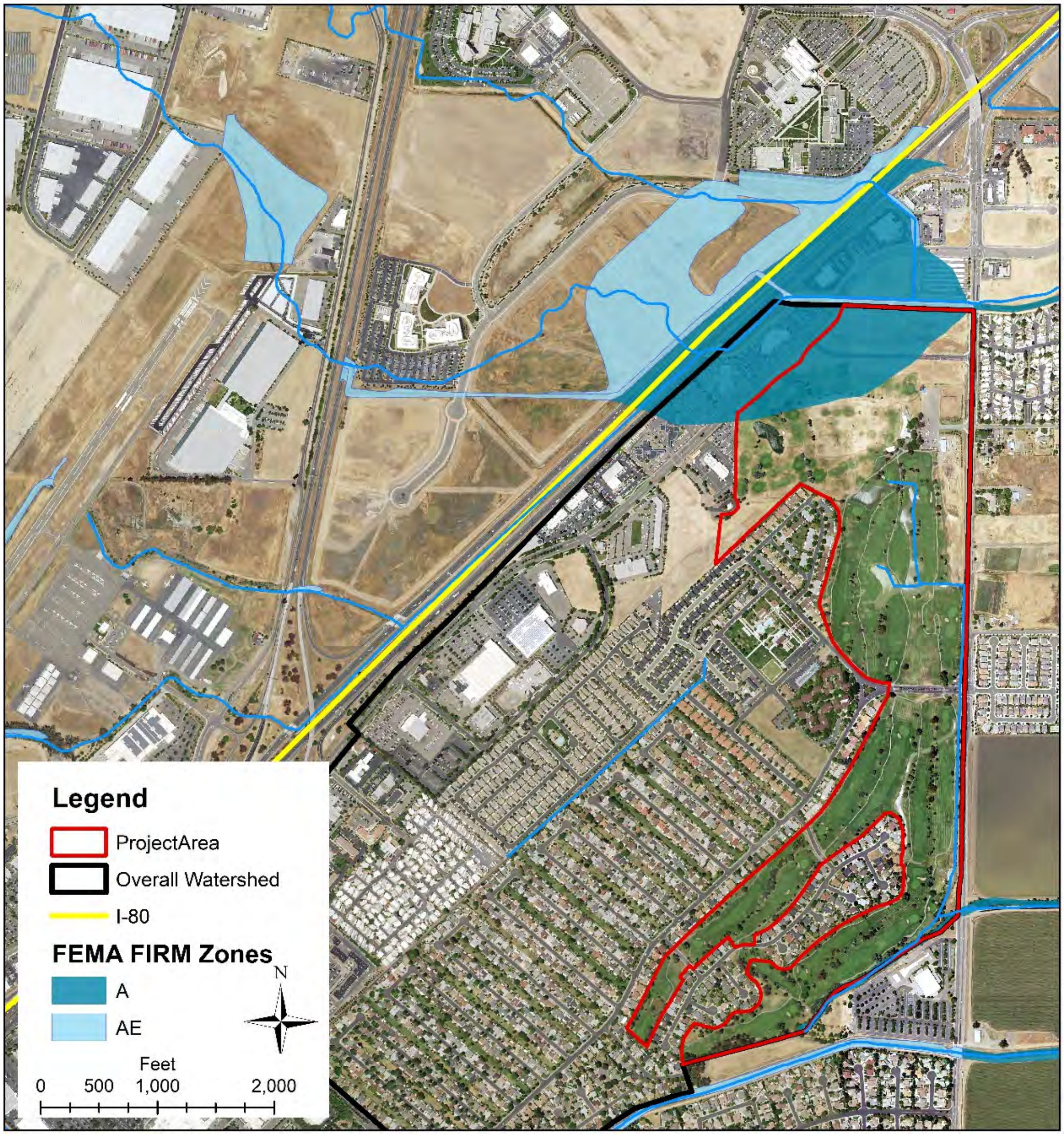

Figure 2-8 FEMA special flood hazard areas ("100-year floodplains"). Flood zone information sourced from FEMA National Flood Hazard Layer for Solano County.

The floodplain on Ulatis Creek has been mapped to be contained within the banks of the engineered flood control channel, likely because of overbank flooding and storage upstream of the project site. Old Ulatis Creek has not been mapped by FEMA.

The Project proposes to limit encroachments into the defined Zone A floodplain area in the northwestern portion of the project site. The Project grading plans will be designed to mitigate flood risk to any housing facilities built within the designated floodplain. Floodplain impacts will be mitigated through implementation of the comprehensive stormwater management strategy, and post-project floodplain extents and water surface elevations will be reviewed and documented through Conditional and Final Letters of Map Revision (CLOMR and LOMR) processed through FEMA.

3 GROUNDWATER

To comply with the California Environmental Quality Act (CEQA) standard of significance, the proposed Project must not substantially deplete groundwater supplies or interfere substantially with groundwater recharge such that there would be a net deficit in aquifer volume or a lowering of the local groundwater table. The City of Vacaville operates its own Groundwater Sustainability Agency (Vacaville GSA) and is a participant in the Solano Collaborative which will be responsible for the adoption of the Solano Subbasin Groundwater Sustainability Plan (GSP) by 2022. The importance of groundwater to the Project is examined through an analysis of estimated historical groundwater use and projected post-project water use.

3.1 Groundwater Recharge and Supply

City of Vacaville groundwater is sourced from the Sacramento River groundwater basin; more specifically the Solano subbasin, which is defined by Putah Creek to the north, the Sacramento River to the east, the North Mokelumne River to the southeast, and the San Joaquin River to the south. To the west, the subbasin boundary is defined by the hydrologic divide that runs along the ridges of the English Hills and the Montezuma Hills and separates lands draining to the San Francisco Bay from those draining to the Sacramento-San Joaquin River Delta.

The Tehama Formation is the thickest fresh-water-bearing geologic unit in the Solano subbasin, and ranges in thickness from 1,500 to 2,500 feet (Bulletin 118, 2004). The Tehama Formation consists of moderately compacted silt, clay, and silty fine sand enclosing lenses of sand and gravel, silt and gravel, and cemented conglomerate. As a result, permeability of the Tehama Formation is variable. Overlying sediments are younger alluvium, with a similar variety of sediment sizes to the Tehama Formation, but with higher permeability. Primary recharge to the aquifers in the younger alluvium is likely sourced from the modern channel corridor, where coarser alluvial deposits accelerate deep percolation of surface runoff. Despite moderate to low permeability, wells in the Tehama Formation can yield several thousand gpm because of its relatively greater thickness (Bulletin 118, 2004). Primary recharge for the Tehama Formation is an outcrop area of over 35 square miles in the English Hills north of the city (Vacaville UWMP, 2010). Groundwater in the units under the Tehama Formation is saline and not used for water supply.

The City of Vacaville owns and operates twelve groundwater wells, ten of which withdraw water from the deep aquifer zone of the Tehama Formation, which has a thickness of 2,200 feet near Elmira Field where most of the city-operated wells are located. Approximately 5,000 acre-feet is withdrawn annually to meet City water demands. The average per capita water use in the City was 150 gallons per day in 2010; 76 percent of water use was attributed to the residential sector at a rate of 114 gallons per day (Vacaville UWMP, 2015).

The majority of the Project site has Hydrologic Soil Group (HSG) C soils composed of sandy clay loam that have low infiltration rates when thoroughly wetted (Figure 2-4). High rates of ET (discussed in Section 2.3) combined with low infiltration potential suggests that historically, the Green Tree Golf Course was likely not a significant source of groundwater recharge for the Tehama Formation. Any percolation of rainfall on the Project site may recharge the shallow aquifers located in the younger alluvium; this would most likely occur in the southern portion of the Project area where HSG B soils are located. An increase in the amount of impervious cover would not likely impact the total water supply available in the aquifer in the younger alluvium, especially with a reduction in pumping from this aquifer as a result of the Project (discussed further in Section 2.3). The source water for the developed Project would likely be supplied by the City of Vacaville from the Tehama Formation. This would place a new water demand upon the source aquifer of the City water supply but would cease groundwater withdrawals from the younger water-bearing alluvium layer.

3.2 Historical Golf Course Water Use

Historically, water consumption on the Project site was in the form of irrigation for golf course turf. The Green Tree Golf Course operated one well with a design pump capacity of 600 gpm, which pumped from 300 feet below ground surface. The Tehama Formation is approximately 400 feet deep near the Project site (Vacaville UWMP, 2010), so the existing Green Tree well pulled groundwater from younger alluvial floodplain deposits composed of inter-bedded sands, clays, and silts. Wells within the younger alluvium likely produce relatively low yields compared with the Tehama Formation, but were sufficient for the needs of Green Tree Golf Course irrigation.

The well is located in the northwest corner of the golf course property adjacent to the irrigation pond. Pumped water flowed from the irrigation pond through the inter-connected pond network (Figure 2-6). Water was siphoned from each pond to irrigate turf local to each pond. The irrigation pond was kept full at all times, either via direct runoff during rainy months, or filled with groundwater during dry months. When the golf course was in operation, water was kept in all ponds for visual appeal. Water levels were lowered throughout the pond system just before large storm events by removing flashboards in Pond 8 to create more stormwater storage capacity.

To quantify historical water use from irrigation, a power bill associated with the groundwater pump was used to estimate total pumping from December 2010 to February 2016 when all irrigation ceased upon golf course closure. Under the assumption that the 50.8 horsepower pump was able to maintain the 600 gpm target capacity under seasonally fluctuating groundwater levels, derived water use can be estimated using monthly power bills (Figure 3-1).

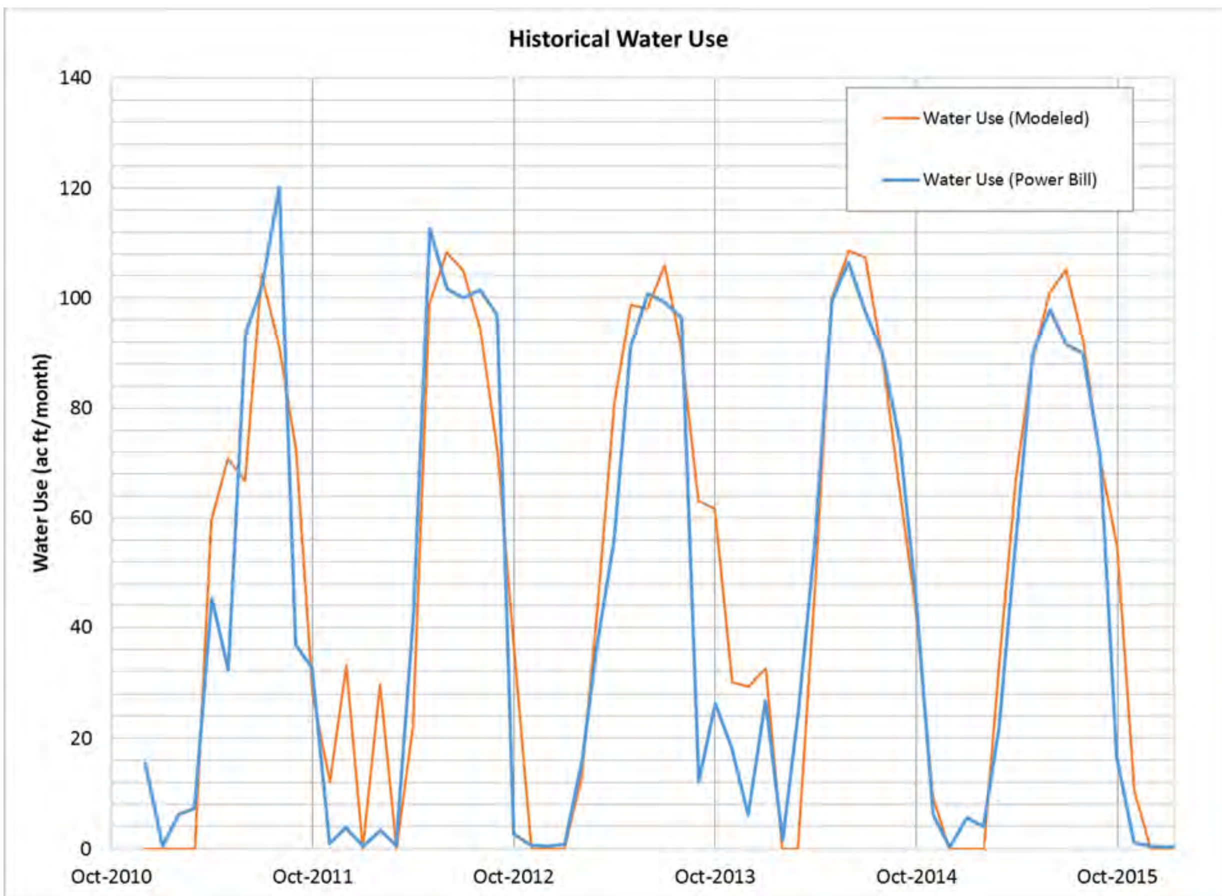

Figure 3-1 Modeled historical water use and water use derived from power bill.

A water balance model was created to calculate historical water use over a longer time-period. The water balance used a monthly record ET from the CIMIS gage in Dixon, CA from September 1994 through November 2016. Reported ET values are measured from irrigated turf, which is the same land use category as golf course turf irrigation and thus a reasonably accurate estimation of conditions within the Project area. However, precipitation differs by significant amount between the CIMIS Dixon gage and the project site (Figure 2-2). A limited precipitation record from December 2006 onward was available for the Solano County rain gage near the crossing of Sweeney Creek and the Putah South Canal in Allendale, California (Solano County gage 510-SWEECK). With the periods of overlapping precipitation record, monthly rainfall at the Sweeney Creek gage was determined to be approximately 24% higher than monthly rainfall rates at the CIMIS Dixon gage. Mean annual precipitation values for Allendale, California range from 23-34 inches, compared with 25 inches at the project site (Figure 2-2). For the following modeling effort, we therefore use the CIMIS Dixon monthly rainfall data scaled up by approximately 24% as a representation of the rainfall record at the project site.

Soil moisture storage was calculated as accumulating when precipitation was greater than ET for any given month. When ET was greater than precipitation, soil moisture storage was reduced, until no soil moisture was left, or no precipitation occurs. Soil moisture storage was taken to be limited by the available water capacity in a 36-inch soil column. The area-weighted available water supply (AWS) of each of the soil types was found to be 5.72 inches (NRCS, Web Soil Survey). Thus, maximum soil moisture storage was capped at 5.72 inches and any additional water was assumed to infiltrate below the turf root zone or flow away as surface runoff. The water balance assumed that the Project site historically maintained an average soil moisture storage of 0.6 inches. If cumulative soil moisture exceeded the target soil moisture, irrigation was not required to meet moisture targets and no irrigation was required. Conversely, if cumulative soil moisture was lower than the target soil moisture, irrigation was applied to make up the soil moisture deficit. The water use model has good agreement with the water use calculated from the power bill. The model over-predicts the water usage increasing over time, as irrigation practices may have adjusted to drought conditions. The model was not adjusted to fit the power bill data in these years and can be considered a conservative estimate of water consumption.

Target soil moisture was applied to an estimated 95% of the 150-acre golf course, to account for the areas of the project site which were not irrigated turf, such as golf cart paths. Average modeled annual water usage from 1995 to 2015 was estimated as 577.5 acre-feet per year (Table 3-1).

Table 3-1 Historical Water Use. Note that the power bill was only available for Calendar Years 2011 – 2015

	Total Annual Water Use (ac-ft)	
Calendar Year	*Modeled*	*Power Bill*
1995	551.9	
1996	510.4	
1997	633.1	
1998	441.6	
1999	625.2	
2000	545.8	
2001	608.1	
2002	571.8	
2003	520.3	
2004	590.2	
2005	534.0	
2006	500.2	
2007	650.7	
2008	659.7	
2009	653.7	
2010	484.1	
2011	538.5	481.8
2012	566.2	563.4
2013	715.1	559.6
2014	603.8	628.4
2015	623.2	547.1
Average:	577.5	556.1

Over 21 years of this historical record, AWS exceeded the maximum soil storage capacity of 5.72 inches only during very wet years (e.g. winter of 1995-1996, 2005-2006, 2010-2011), and usually in December, January, and/or February. With these results and the above knowledge of aquifer characteristics, recharge potential from direct precipitation on the Project site into the younger alluvium shallow aquifer is most likely to be very small for two main reasons: 1) clay and silt layers within the younger alluvial deposits likely restrict vertical connectivity and reduce the potential for direct infiltration of precipitation, and

2) the low infiltration potential of HSG C soils, which covers most of the Project site, slows initial infiltration and ET likely predominates instead of percolation.

3.3 Post-Project Water Use

Projected water use for post-project conditions was estimated in three main categories: residential, commercial, and irrigation of open space. The proposed water demand is in Table 3-2.

Residential water use was determined on a per-unit basis, using the 2018 City of Vacaville Water System Water Plan (WSMP, 2018) of 335 gallons per day (gpd) for Residential Low Density, 265 gpd for Residential Medium Density, and 230 gpd for Residential High Density. The estimated water use of 245 gpd per unit for the Residential Medium High Density was estimated based on the density of the proposed units compared the Residential Medium Density and Residential High Density demand factors. Total potable water use from the proposed residential units is an estimate 321 gpd per unit. The number of each type of unit was based on the best estimate at the time this report was compiled and may be subject to change.

Commercial water use was calculated based on the estimated total acreage of commercial space within the planned development[6]. Estimated water use of 1,615 gpd per acre was used in calculations (1,230 gpd per acre potable, 385 gpd per acre irrigation), resulting in an estimated total water usage of 37.8 acre-feet per year (VWSMP, 2018).

In the current Project plan, 55 acres are dedicated to open space, parks, and retention basins. To our knowledge, only the parks (5.7 acres) are currently planned for irrigation. However, given the uncertainty in the proposed land use for the remaining open space, we have conservatively assumed that approximately 50% of the total open space (27.5 acres) would be irrigated turf. A more precise calculation of landscaping water use can be made when more detailed plans are available and actual areas planned for irrigation are delineated. We have used a water demand factor of 1,250 gpd per acre irrigation

[6] The recommended demand factor for the Commercial Service land use type was used (VWSMP, 2018)

water for the public park space (VWSMP, 2018), which results in an estimated total water usage of 38.5 acre-feet per year (Table 3-2).

Table 3-2 Estimated water usage under post-project conditions.

Residence Type	Number of Units *units*	Potable Water Use *gpd/unit*	Irrigation Water Use *gpd/unit*	Potable Annual Use *ac-ft/yr*	Irrigation Water Use *ac-ft/yr*
Residential Low	187	335	0	70.2	0.0
Residential Medium	179	265	0	53.1	0.0
Residential Medium High	342	245	0	93.9	0.0
Residential High	402	230	0	103.6	0.0
	Commerical Area *acres*	**Potable Water Use** *gpd/unit*	**Irrigation Water Use** *gpd/unit*	**Potable Annual Use** *ac-ft/yr*	**Irrigation Water Use** *ac-ft/yr*
Commercial	21	1,230	385	28.8	9.0
Landscaping	**Open Space** *acres*	**Potable Water Use** *gpd/unit*	**Irrigation Water Use** *gpd/unit*	**Potable Annual Use** *ac-ft/yr*	**Irrigation Water Use** *ac-ft/yr*
Irrigated Acres	27.5	0	1,250	0.0	38.5
				Total Potable	349.5
				Total Irrigation	47.5
				Total Water Use	397.0

The post-project water use for landscaping assumes modern and efficient irrigation practices gained from new irrigation equipment. Irrigation systems at the golf course were last installed in the early 2000's and were likely a relatively efficient system designed to optimize water use. However, the main transport of water across the golf course was through the inter-connected pond system, which resulted in significant water losses from ET from the surface of the ponds at a rate of approximately 30 acre-feet per year[7], as well as some infiltration into the shallow subsurface.

Estimated post-project water use is based on the Project site plan available when this report was written. Any changes to the size of commercial properties or the number or type of residential units will change water use estimates. In addition, refinement of the landscape plan may necessitate a revision of the water use estimates for landscaping.

[7] Calculated under the assumption that reference ET from the Dixon, CA CIMIS gage is a good approximation of ET from a pond surface, applied over the ~6.6 acres of golf course ponds.

Under the previous land use of a golf course, approximately 556 to 578 ac-feet per year of water was used over the past 20 years for irrigation (Table 3-1), whereas it is estimated that the proposed Project will use an estimate 397 acre-feet of water each year.

Finally, it is important to note that existing golf course irrigation water was extracted from shallower groundwater in younger alluvial deposits, whereas all water used in the post-project development will be drawn from City groundwater supplies extracted from the Tehama Formation. Therefore, water use for the proposed Project will increase the amount of water the City will need to supply from that source.

4 CLIMATE CHANGE

As changing climates affect the amount and timing of precipitation and air temperatures, water supplies may be additionally taxed beyond increasing city populations. In addition, water supply and flood management infrastructure could be additionally affected when considering the potential impacts of climate change.

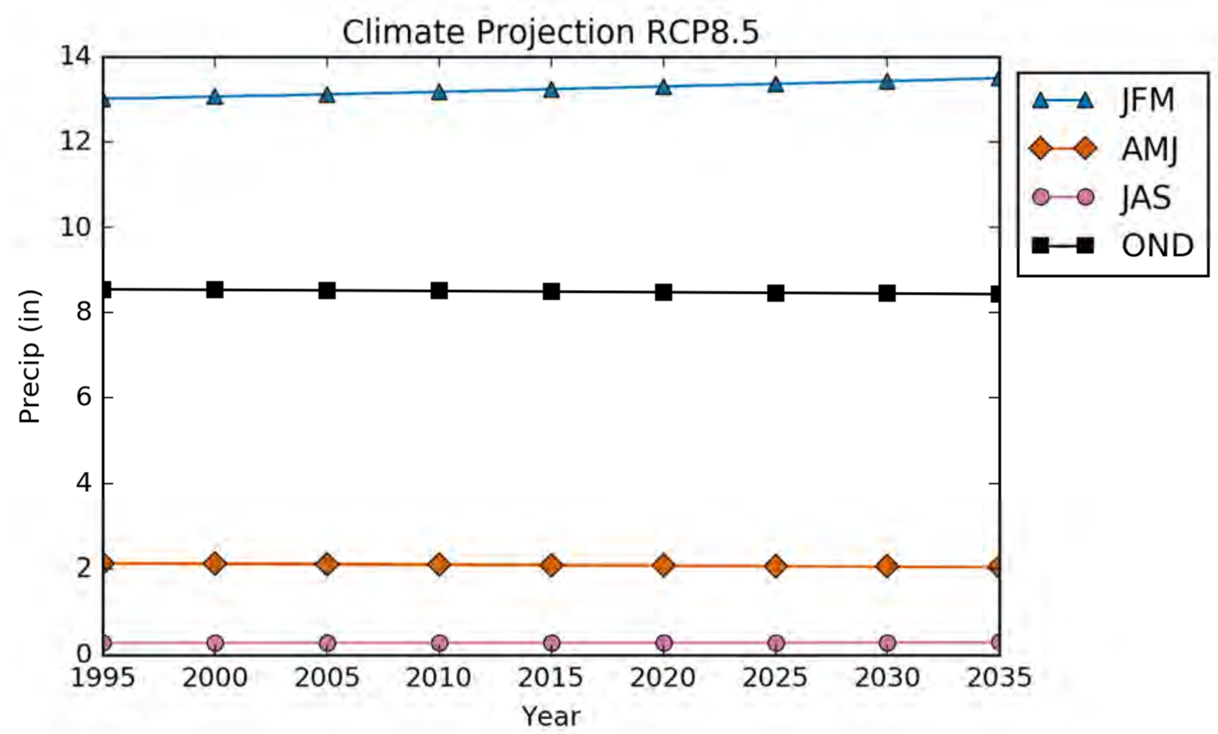

Figure 4-1 Baseline and projected average precipitation totals (inches). Each line represents precipitation totals averaged over three-month periods. Data sourced from CLIMsystems, 2017.

Figure 4-1 and Figure 4-2 show historical baseline data and then projected climate scenarios for mean temperature and precipitation averaged by three month periods for the City of Vacaville. Mean temperature and precipitation projections were derived from SimCLIM (CLIMsystems, 2017), a climate change software application that produces spatial data and builds databases of climate projections for a variety of parameters. The historical baseline data is an average of the years 1981 to 2010 and is plotted as the central baseline year of 1995. SimCLIM uses 40 of the most recent global circulation models (GCMs) with the current generation (CMIP5) of global coupled ocean-atmosphere modules at a 0.5° x 0.5° model resolution. These GCMs were used in the Fifth

Assessment Report of the United Nations Intergovernmental Panel on Climate Change (IPCC, 2013). SimCLIM downscales the model to a 1km x 1km grid cell resolution. The SimCLIM user interface allows for selection of one of four representative concentration pathways (RCP): 2.6, 4.5, 6.0, and 8.5. Each RCP represents a greenhouse gas concentration trajectory, which were updated in 2014 (CLIMsystems, 2017). The 8.5 RCP, shown in Figure 4-1 and Figure 4-2, represents the “worst-case” scenario and is used in this analysis as the most conservative approach.

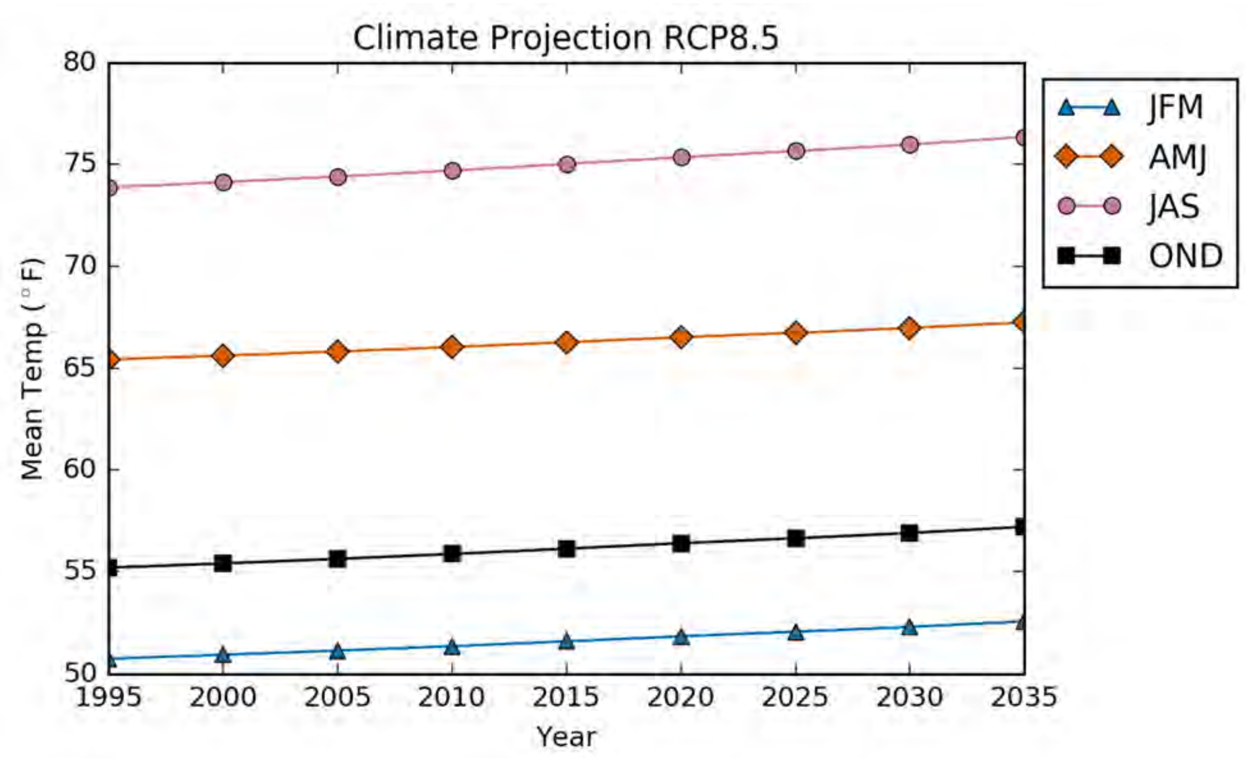

Figure 4-2 Baseline and projected average temperature totals (degrees Fahrenheit). Each line represents daily temperatures averaged over three-month periods. Data sourced from CLIMsystems, 2017.

Table 4-1 summarizes the impacts of climate change for each three-month period, comparing the modeled baseline values, to projected values for 2035. General trends indicate that Vacaville will experience overall warmer temperatures for all time periods, with the largest percentage increases expected for the fall and winter months. Overall, mean annual precipitation is expected to increase slightly, with nearly all the changes projected to occur in the wet months of January, February, and March. Notably, precipitation in the spring and fall months are projected to decrease by small amounts.

Table 4-1 Projected temperature and precipitation data. Comparing baseline 1995 to projected 2035.

	Project Climate Change Variables for Worst Case RCP 8.5						
	Precipitation (in)				Average Air Temperature (°F)		
Time Period	*1995*	*2035*	*% Change*		*1995*	*2035*	*% Change*
JFM (Jan, Feb, Mar)	13.0	13.5	3.8%		50.7	52.6	3.7%
AMJ (Apr, May, Jun)	2.1	2.0	-4.8%		65.4	67.2	2.8%
JAS (Jul, Aug, Sep)	0.3	0.3	0.0%		73.9	76.3	3.2%
OND (Oct, Nov, Dec)	8.5	8.4	-1.2%		55.2	57.2	3.6%
Total:	23.9	24.24	1.4%	Average:	61.3	63.3	3.3%

Considering the impacts of changing climate conditions, the current 10-, and 100-year design storms may not remain applicable over long-term time scales. Specifically, the magnitude of the flood peaks in the winter months may increase slowly over time. The proposed Project would convert a grassy golf course into urban development, which would result in an increase in surface runoff because of an increase in impervious surfaces. Under potential climate change impacts, urban runoff flood peaks may increase. Thus, there is a possibility that stormwater basins sized for the current design storm may not be adequate for floods with higher peak flows. However, the potential flooding risks associated with impacts from the proposed Project and potential climate change can be mitigated by additional flooding safeguards, such as freeboard built into the design of the stormwater basins, which will help mitigate infrequent and higher-magnitude floods that may occur because of climate change.

5 REGULATORY SETTING

The California State Water Resources Control Board (SWRCB or State Board) and the nine Regional Water Quality Control Boards (RWQCB or Regional Board) have the authority in California to protect and enhance water quality, both through their designation as the lead agencies in implementing the Section 319 nonpoint source program of the federal Clean Water Act, and from the state's primary water-pollution control legislation, the Porter-Cologne Act. The RWQCB Region 5 office guides and regulates water quality in streams and aquifers of the Central Valley, Sacramento River and San Joaquin River Basins, through designation of beneficial uses, establishment of water-quality objectives, administration of the National Pollution Discharge Elimination System (NPDES) permit program for stormwater and construction site runoff, and 401 water-quality certification where development results in fill of jurisdictional wetlands or waters of the United States, under Section 404 of the Clean Water Act. The Project site is located within the jurisdiction of Region 5.

5.1 California State Laws and Regulations

The California Municipal Storm Water Permitting Program regulates storm water discharges from municipal separate storm sewer systems (MS4s). MS4s are defined as a conveyance or system of conveyances (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, manmade channels, or storm drains) owned or operated by a public body (i.e. city, county, etc.) designed or used to collect or convey stormwater, not a combined sewer, and not part of a publicly owned treatment works. The City of Vacaville operates under the State Water Resources Control Board Region 5 (SWRCB5) permit (no. CAS000004), which designates regulations of post-construction stormwater management (Section E.12) in the following sections:

- Hydromodification in Section E.12.f
- Low-impact development (LID), including bioretention stormwater retention in Section E.12.e

The National Pollutant Discharge Elimination System (NPDES) permit regulates discharges from storm sewer systems into receiving water to minimize impairments. The City of Vacaville operates under the NPDES permit for the SWRCB5 (NPDES no. CAS0085324, order no. R5-2016-0040). Recognized impairments to the receiving waters related to this Project include Ulatis Creek, which is the receiving waters for the southern end of the

Project, and downstream of all the Project site receiving waters (Horse Creek, and Old Ulatis Creek). See Section 6.1 for details.

The importance of maintaining sustainable groundwater supplies has driven the formation of a regulatory framework, as passed into law by the statewide Sustainable Groundwater Management Act (SGMA, 2014). In the City of Vacaville, which includes the Green Tree project area, a groundwater basin planning process is underway. Local stakeholders have formed the City of Vacaville Groundwater Sustainability Agency to develop implementation plans for sustainable groundwater use in the city.

5.2 Local Regulations and Policies

In addition to the NPDES permitting program, the RWQCB regulates water quality for the region in accordance with the Water Quality Control Plan or 'Basin Plan' (RWQCB, 2016). The Basin Plan presents the beneficial uses that the Regional Board has designated for local aquifers, streams, marshes, rivers, and the Delta as well as the water-quality objectives and criteria that must be met to protect these uses. The objectives, though occasionally numeric, are generally in narrative form and are to be applied so as to ensure that there is not degradation of the receiving waters and that the "beneficial uses" are not impaired. Multiple beneficial uses have been designated for the Sacramento River Delta, the receiving waters for the Project area, and include municipal, domestic, and agricultural supply, industrial process and service supply, contact and non-contact water recreation, warm and cold freshwater habitat and fish migration, warm water fish spawning, wildlife habitat, and navigation.

Aquatic habitat in the Delta is the most sensitive beneficial use to potential impacts from development of the proposed Project. Pollution from driveways, roads, and parking lots, could contribute petroleum products and heavy metals to storm runoff and degrade water quality downstream. Pesticides and fertilizers applied to residential and commercial landscaping could also be mobilized by rainfall and be transported to the delta sloughs, potentially affecting aquatic and terrestrial wildlife species in the river or the adjacent riparian zone.

6 WATER-QUALITY MANAGEMENT

Implementation of an effective water-quality management approach is a key component of the overall stormwater management strategy at the site. Successful water-quality programs are based on an understanding of existing conditions and constraints, and this section addresses those issues.

6.1 Existing Water Quality Conditions

The most immediate water-quality concern with respect to the Project is to protect the functions and values of the aquatic resources in the area. The Project area drains to either Horse Creek, to Old Ulatis Creek, or to Ulatis Creek. Old Ulatis Creek joins with Horse Creek approximately 2 miles downstream of Leisure Town Road. Horse Creek is a tributary to Ulatis Creek, which was added to the 303(d) list as an impaired body of water in 2014 and is impaired by three pesticides, chlorpyrifos, diazinon, and diuron (California 303(d) List, 2014). The pesticides were banned for residential uses in the mid-2000’s but are still allowed for agricultural uses. The proposed Project does not include any land use designated for agricultural purposes and therefore will likely not affect the levels of chlorpyrifos, diazinon, and diuron in Ulatis Creek. Ulatis Creek is also impaired with respect to toxicity. Proposed water quality treatment of on-site runoff is designed to mitigate urban impact on toxicity. A Total Maximum Daily Load (TMDL) program for Ulatis Creek for toxicity is scheduled for completion by 2027.

6.2 Stormwater Management Constraints and Opportunities

The Project site has several constraints and opportunities that frame the stormwater management approach with respect to water quality.

The primary constraint is related to the high runoff soils. As described in Section 2.4, the entire project site is characterized by soils that have relatively high runoff potential. Many effective stormwater best management practices (“BMPs”) rely on infiltration of stormwater runoff as a water-quality control measure. Examples of such approaches include infiltration basins, drywells, and the use of permeable pavement (CASQA, 2003). In general, the Project site is underlain by soils that are unfavorable for infiltration-based measures.

The significant need to preclude infiltration-based control measures is compensated for by a number of marked opportunities presented by the type of proposed land uses, and the physical characteristics of the site. These opportunities are used to the greatest extent possible in the selection and design of BMPs and include the following:

- *Clustered development and low level of built-up land use*. The Project will cluster dense development in the northern section of the Project area, utilizing a design that minimizes impervious area for the amount of floor space provided. The southern section of the project area has mostly low- to medium-density single-family homes, with a considerable amount of open space, to aid in biofiltration through vegetated areas. A park area North of Sequoia Drive will also increase biofiltration capacity.

- *Low slopes*. The Project area and adjacent contributing sub-watersheds have very low surface slopes, which allow for less rapid runoff and more options in stormwater quality treatment.

- *High runoff soils*. Although listed as a constraint above, the high runoff soils at the site can also be viewed as an opportunity since the proposed construction of impervious cover will have a less significant impact than at a site located on more permeable soils.

6.3 Water-Quality Best Management Practices

In light of the opportunities and constraints that exist at the Project site, developing an effective BMP framework requires implementing a number of practices specific to the site conditions. The BMP framework will be based on a hierarchical approach advocated by stormwater quality regulators (e.g. see BASMAA, 1999). The hierarchical approach has the following levels:

- *Level I – Site Design.* One of the key elements of the stormwater strategy for the Project will be incorporating appropriate site design elements that enhance efforts to limit water-quality impacts. Properly implemented features in essence “set the stage” for an effective plan by establishing a land use pattern that limits the amount of directly connected impervious areas (DCIAs) to the greatest extent practicable.

- *Level II – Source Control.* Another of the primary focuses of this plan is a strong and broad-based source control program. This approach capitalizes on the fact that it is generally more effective, both in impacts and costs, to prevent or limit constituents of concern from being released than it is to remove them from the environment once they have been mobilized (BASMAA, 1999).

- *Level III – Treatment Controls*. The term "treatment controls" refers to those BMPs that are designed to reduce constituents of concern once they have been mobilized in stormwater runoff. Treatment controls are generally considered necessary BMPs since even the most aggressive site design and source control programs cannot guarantee that constituents of concern will not be mobilized from the site. In sites with low infiltration soils, such as the Project site, treatment controls will be practically essential to mitigate for potential hydromodification effects from development.

6.3.1 Site Design Elements

The primary goal of water-quality sensitive site design is to limit the amount of DCIAs within the development envelope. Limiting DCIAs promotes infiltration (though modestly in areas with low permeability), increases times of concentration within drainage areas, and reduces runoff volumes. Additionally, less impervious area generally leads to increased amounts of space that can be dedicated to landscaping and open space uses that limit the introduction of pollutants to the environment and can filter out pollutants that already have been mobilized.

Specific site design features that will be included to the maximum extent practicable include the following:

- *Reduced street widths.* The project proposes to use the minimum street widths compatible with safety of the residents and in conformance with the requirements of the City of Vacaville. Average street widths will be on the order of 30 feet, markedly less than average widths in other locations.

- *Home design.* Homes will utilize designs that have a number of positive aspects with respect to stormwater management. Notably, the designs will minimize impervious area for a given interior floor space and will use disconnected downspouts to direct roof runoff to the vegetated areas.

- *Open space.* The Project includes a considerable amount of vegetated buffer areas and other public area (parks, plazas, gardens, etc.) which will remain as open space.

6.3.2 Source Control Elements

The source control program will incorporate a number of strategies:

- *Education and outreach.* The City of Vacaville has several outreach strategies designed to engage residents in the need to control non-point source pollution. One proven tactic in this regard is the marking of storm drain inlets and collection points to indicate that runoff can directly impact receiving waters. At this site, such markings may be along the lines of “Drains to the Delta” or “Drains to Waterways”.

- *Landscaping.* All landscaping will incorporate plant species appropriate for the site soils and climate. Per the Specific Plan, the Project will utilize drip irrigation to the maximum extent practicable.

- *Trash storage areas.* All trash storage areas in commercial areas will be covered to prevent run-on and contained to prevent off-site transport of pollutants and trash.

- *Regular street sweeping*. Regular street sweeping can have a significant impact on the control of such constituents of concern as trash and debris, particulates, and heavy metals. The City of Vacaville coordinates a regular street sweeping program that will include the Project area.

6.3.3 Treatment Control Elements

Treatment controls are generally considered necessary as a final element in water-quality protection even when the use of approved site planning and source control BMPs is maximized. Pollutants typically found in urban runoff include heavy metals (i.e., copper, lead, zinc, cadmium, mercury), oils and greases, nutrients (nitrogen and phosphorus), household and lawn-care chemicals (insecticides, herbicides, fungicides, and rodenticides), and coliform bacteria.

Ultimately, BMPs must comply with the requirements of the Phase II Small Municipal Separate Storm Sewer System (MS4) General Permit (Order No. 2013-0001 DWQ, effective July 1, 2013). The site design measures of the MS4 Permit are generally more stringent than past requirements in that Permitees must design facilities to evapotranspire, infiltrate, harvest/use, and biotreat storm water. The clayey soils and low infiltration rates of the Project site make infiltration generally infeasible. Rainwater may be harvested and used for irrigation, but the demand for domestic uses far exceeds the supply. For these reasons, bioretention basins are proposed as the primary treatment mechanism. Provisions of the MS4 Permit will require the basin floors to include an 18-inch layer of select soil mix suitable to maintain infiltration rates of up to 5 inches per hour, underlain by a gravel sub-drain layer. Underdrains will be installed near the top of the gravel layer to facilitate percolation through the bioretention medium and to prevent long-duration ponding.

Preliminary sizes for bioretention basins were estimated with a combination flow and volume design basis. The generalized sizing approach of multiplying the effective impervious area by a factor of 0.04 is a strictly flow-based method and does not consider the volume of runoff that is treated by infiltrating during the respective design storm (having an intensity of 0.2 inches/hour). This approach ignores the passive storage volume that is available in the bioretention facility, and that is available to accommodate short periods of peak intensity. An alternative sizing convention uses a combination of flow and volume-based approaches which consider: (1) the volume of runoff infiltrating through the bioretention facility over the course of the design storm, and (2) the volume of runoff held in the bioretention facility during the design event. This approach results in basin floor areas equal to roughly three percent of the effective impervious area, and sometimes less. For this analysis, preliminary basin sizes to meet the water-quality requirements of the MS4 permit were estimated as 3 percent of the effective impervious area.

The required bioretention areas are summarized in Table 6-1 below for only the Project area being developed, not including the pre-existing developments which drain through the Project site, see Figure 7-3 for reference. Information on the contributing watersheds can be found in Section 7. Under the above assumption, a total of 2.9 acres of the Project area must be dedicated to bioretention facilities. This requirement can be met using a combination of distributed “rain gardens” or bioswales in green streets and strips, and biofiltration soils designed for infiltration built into the bottoms of the multi-function stormwater basins also designed for peak flow detention. The use of infiltration in addition to biofiltration in the southern portion of the Project site may be advantageous as soil type B has higher infiltration potential compared with soil type C, which covers most of the Project site (Figure 2-4).

Table 6-1 Water quality treatment bioretention area. See Figure 7-3 for an illustration of the sub-watershed land use classifications.

Sub-watershed	Total Area (acres)	Impervious (%)	Effective Imp. Area (sq ft)	Effective Imp. Area (ac)	Bioretention Area (ac)	Land Use (--)
LTR South	9.0	60	234,900	5.4	0.16	Streets
OUS BASIN	4.8	85	178,500	4.1	0.12	Basin
SA 520	7.6	55	182,100	4.2	0.13	Residential
SA 527	4.4	55	106,100	2.4	0.07	Residential
SA 528	2.3	55	55,100	1.3	0.04	Residential
SA 532	3.1	55	73,500	1.7	0.05	Residential
SA 534	2.5	55	60,200	1.4	0.04	Residential
SA 537	5.6	50	123,000	2.8	0.08	Residential
SA 540	5.1	50	111,200	2.6	0.08	Residential
YD BASIN	2.6	85	94,500	2.2	0.07	Basin
CA 715	9.3	45	183,100	4.2	0.13	Basin
CE 830	8.8	50	191,700	4.4	0.13	Basin
WS 905	1.3	5	2,800	0.1	0.00	Open Space
SC Park	6.7	10	29,000	0.7	0.02	Open Space
CP BASIN	0.7	5	1,600	0.0	0.00	Basin
LI BASIN	3.0	85	110,200	2.5	0.08	Basin
TETON PARK	3.9	5	8,500	0.2	0.01	Open Space
HCWB	2.1	80	71,500	1.6	0.05	Basin
Res 7 West	6.0	60	155,800	3.6	0.11	Residential
WB Streets	5.9	85	216,900	5.0	0.15	Streets
Comm 2 East	5.2	80	181,400	4.2	0.12	Commercial
Residential 5	8.6	55	207,200	4.8	0.14	Residential
Comm 2 West	4.0	80	138,100	3.2	0.10	Commercial
Comm 3	6.0	80	210,000	4.8	0.14	Commercial
Residential 6	5.0	55	119,800	2.8	0.08	Residential
HCEB	2.0	80	69,400	1.6	0.05	Basin
Res 7 East	4.8	60	125,600	2.9	0.09	Residential
EB Streets	1.0	85	36,500	0.8	0.03	Streets
Comm 1	4.7	80	164,400	3.8	0.11	Commercial
Residential 4	7.1	55	171,100	3.9	0.12	Residential
HCP3	10.3	45	202,800	4.7	0.14	Basin/Open Space
Residential 2	5.9	55	141,300	3.2	0.10	Residential
P3 Streets	2.6	85	95,600	2.2	0.07	Streets
Residential 3	8.5	55	204,000	4.7	0.14	Residential
Residential 1	10.9	55	261,700	6.0	0.18	Residential
HCP1	2.7	80	93,700	2.2	0.06	Basin
HC-02 Residual	5.8	65	163,300	3.7	0.11	Streets
Total	**189.8**		**4,776,100**	**110**	**3.3**	

7 STORMWATER HYDROLOGIC AND HYDRAULIC MODELING METHODOLOGY

To more accurately quantify the project impacts to existing off-site flooding, we have combined the hydrologic and hydraulic modeling into one software package, XPStorm version 2019.1.1 by Innovyze®. We chose the model for the hydrology and pipe hydraulics for the Project area and off-site which contribute flow to take full advantage of the integrated software functionality which includes: 1) 2D distributed rainfall overland flow modeling, 2) 1D pipe hydraulics, 3) point-load input hydrographs for proposed project areas, and 4) complete integration between 1D and 2D model components.

Modeling was completed to estimate runoff rates and volumes across the Project site both for existing and proposed project conditions. Modeling was completed in accordance with criteria described in the City of Vacaville Design Standards (DS-4, 2006) where possible given software compatibilities.

7.1 Hydrologic Modeling Methodology

Below are the key hydrologic model components and parameters.

Design storms. Rainfall depths for the 10-, and 100-year 24-hour design storms were determined using Appendix A from the SCWA Hydrology Manual (1999) and the mean annual precipitation (MAP) of 25 inches applied to each sub-watershed. The 24-hour rainfall distribution was defined using Table DS 4-2 from the City of Vacaville Standard Specifications and Standard Drawings (2006). The rainfall distribution for the 100-year, 24-hour design storm for sub-watersheds with MAP of 25 inches is illustrated in Figure 7-1. The total precipitation depth at the project site is 4.8 and 6.7 inches for the 10-year and 100-year 24-hour design storms, respectively (SCWA, 1999).

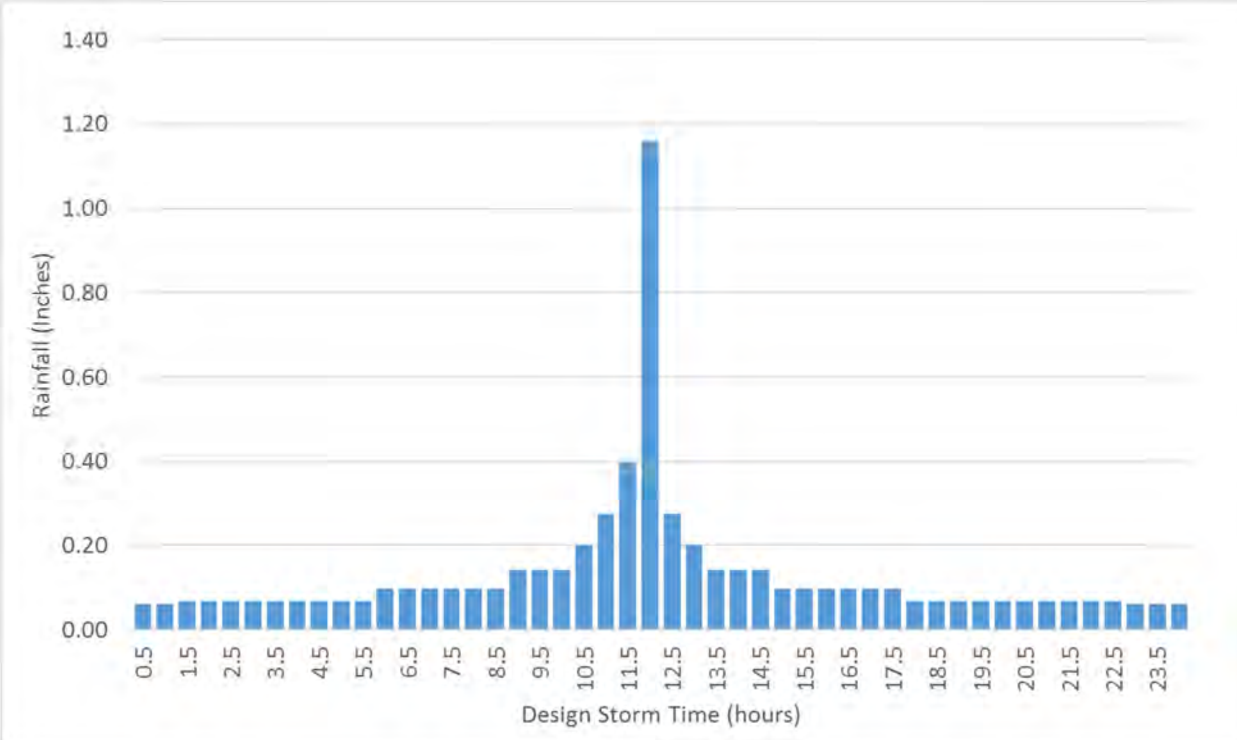

Figure 7-1 Sample Design Storms. 100-year precipitation amounts for the 24-hour event for watersheds with mean annual precipitation of 25 inches.

2D Distributed Rainfall-Runoff. The 10- and 100-year design storms are distributed evenly over the model domain, including the project area on the Green Tree Golf Course and the surrounding neighborhoods which drain to the project area. The model domain is divided up into one of five land use types: commercial, residential, open space, park, or roads. Each land use type is assigned a percent impervious, overland Manning's roughness, and rainfall abstraction in the form of initial and continuing loss. XPStorm applied initial and loss coefficients to the proportion of designated pervious model domain and so uniformly set to 0.25 inches and 0.11 inches per hour, respectively. These parameters are summarized in Table 7-1 and the land use boundaries are illustrated in Figure 7-2.

Table 7-1 Summary of land use type hydrologic parameters

	Manning's N	Percent Imperv.	Initial Loss	Constant Loss
		(percent)	*(in)*	*(in/hr)*
Commercial	0.04	90	0.25	0.11
Open Space	0.2	10	0.25	0.11
Park	0.3	35	0.25	0.11
Residential	0.3	60	0.25	0.11
Roads	0.02	95	0.25	0.11

Notes: Constant Loss assumes soil group C

2D Digital Elevation Model and Model Grid. The overall watershed topography is a combination of the 2009 Solano County Lidar dataset and aerial topography collected of the project site. Elevations are used to generate a digital elevation model (DEM) of the model domain. The 2D model domain is drawn to be slightly larger than the 2D rainfall area to be able to understand potential flow pathways away from the model area. The model domain grid cell size of 20 feet was selected to balance model resolution and computation time. Breaklines were added to the centerline of roads in the model and are assigned the elevation of the DEM to allow preferential flow in the streets along the central axis, increasing the total number of modeled grid cells.

Post-project sub-watersheds. To appropriately model the proposed development of the project area, but in advance of a detailed grading plan required for 2D overland flow modeling, on-site sub-watersheds are modeled using point-loaded hydrographs. The post-project 2D rainfall area is limited to the off-site contributing watershed. Key parameters for these sub-watersheds are summarized in Table 7-2, with the respective sub-watersheds illustrated in Figure 7-3.

Impervious area percentages were estimated from proposed land plans. Pervious areas in the sub-watersheds were modeled using an initial and then constant loss methodology. An initial loss rate was universally set to 0.25 inches (Vacaville DS-4, 2006), and constant loss rates we set to 0.11 inches per hour consistent with hydrologic soil group C.

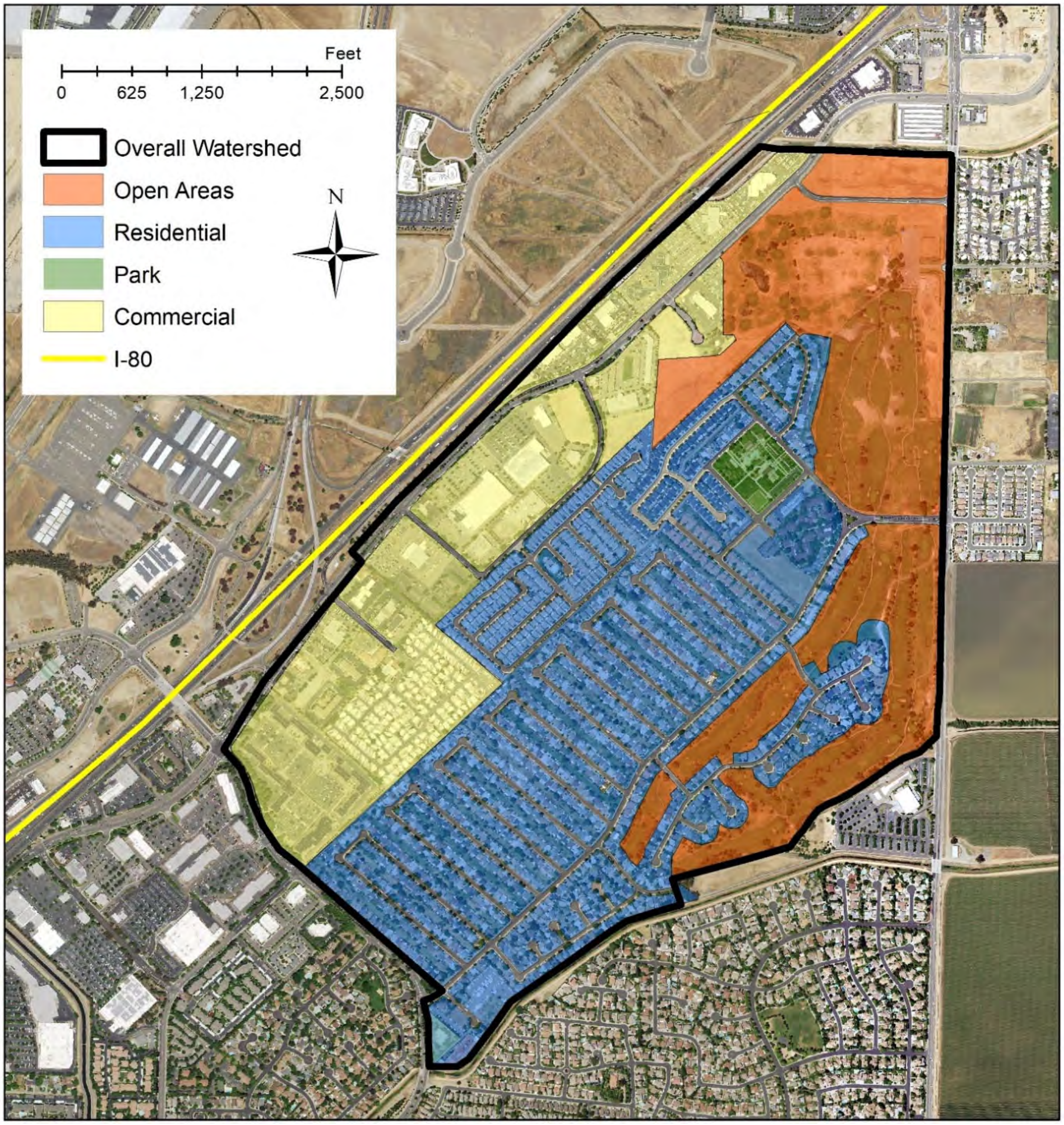

Figure 7-2 Land use zones for commercial, residential, park, and open space land use. Unmapped areas are designated as roads.

To be consistent with the City of Vacaville Standard Specifications and Standard Drawings (City of Vacaville, 2015), the Kinematic Wave Transform Method was used to convert runoff depths to flow hydrographs. Until a grading plan is finalized, basin slope was set to 0.01 feet per feet. Sub-watershed width is reported in Table 7-2.

Table 7-2 Post-project hydrologic parameters. Basin slope is set universally to 0.01 ft/ft. See Figure 7-3 for an illustration of the sub-watershed land use classifications.

Sub-watershed	Model Node	Total Area	Losses Initial	Losses Constant	Impervious	Width	Land Use
		(acres)	*(inches)*	*(inches)*	*(%)*	*(feet)*	*(--)*
LTR South	SDMH 555	8.988	0.25	0.11	60%	350	Streets
OUS BASIN	OUS BASIN	4.821	0.25	0.11	85%	250	Basin
SA 520	SDMH 520	7.600	0.25	0.11	55%	350	Residential
SA 527	SDMH 527	4.430	0.25	0.11	55%	250	Residential
SA 528	SDMH 528	2.298	0.25	0.11	55%	250	Residential
SA 532	SDMH 532	3.069	0.25	0.11	55%	250	Residential
SA 534	SDMH 534	2.513	0.25	0.11	55%	250	Residential
SA 537	SDMH 537	5.645	0.25	0.11	50%	300	Residential
SA 540	SDMH 540	5.106	0.25	0.11	50%	300	Residential
YD BASIN	YD BASIN	2.552	0.25	0.11	85%	250	Basin
CA 715	SDMH 715	9.339	0.25	0.11	45%	350	Basin
CE 830	SDMH 830	8.801	0.25	0.11	50%	300	Basin
WS 905	SDMH 905	1.301	0.25	0.11	5%	250	Open Space
SC Park	Node 243	6.666	0.25	0.11	10%	300	Open Space
CP BASIN	CP BASIN	0.717	0.25	0.11	5%	250	Basin
LI BASIN	LI BASIN	2.977	0.25	0.11	85%	250	Basin
TETON PARK	UC-06	3.912	0.25	0.11	5%	250	Open Space
HCWB	HCW BASIN	2.053	0.25	0.11	80%	203	Basin
Res 7 West	SDMH 205	5.961	0.25	0.11	60%	282	Residential
WB Streets	SDMH 225	5.859	0.25	0.11	85%	213	Streets
Comm 2 East	SDMH 232	5.204	0.25	0.11	80%	294	Commercial
Residential 5	SDMH 234	8.647	0.25	0.11	55%	414	Residential
Comm 2 West	SDMH 237	3.962	0.25	0.11	80%	298	Commercial
Comm 3	SDMH 242	6.025	0.25	0.11	80%	398	Commercial
Residential 6	SDMH 250	5.000	0.25	0.11	55%	369	Residential
HCEB	HCE BASIN	1.993	0.25	0.11	80%	212	Basin
Res 7 East	SDMH 305	4.807	0.25	0.11	60%	299	Residential
EB Streets	SDMH 315	0.987	0.25	0.11	85%	159	Streets
Comm 1	SDMH 316	4.718	0.25	0.11	80%	419	Commercial
Residential 4	SDMH 318	7.143	0.25	0.11	55%	438	Residential
HCP3	P3 BASIN	10.344	0.25	0.11	45%	626	Basin/Open Space
Residential 2	SDMH 412	5.897	0.25	0.11	55%	273	Residential
P3 Streets	SDMH 415	2.582	0.25	0.11	85%	146	Streets
Residential 3	SDMH 422	8.513	0.25	0.11	55%	412	Residential
Residential 1	SDMH 415	10.925	0.25	0.11	55%	429	Residential
HCP1	P1 BASIN	2.688	0.25	0.11	80%	217	Basin
HC-02 Residual	HC-58	5.766	0.25	0.11	65%	84	Streets

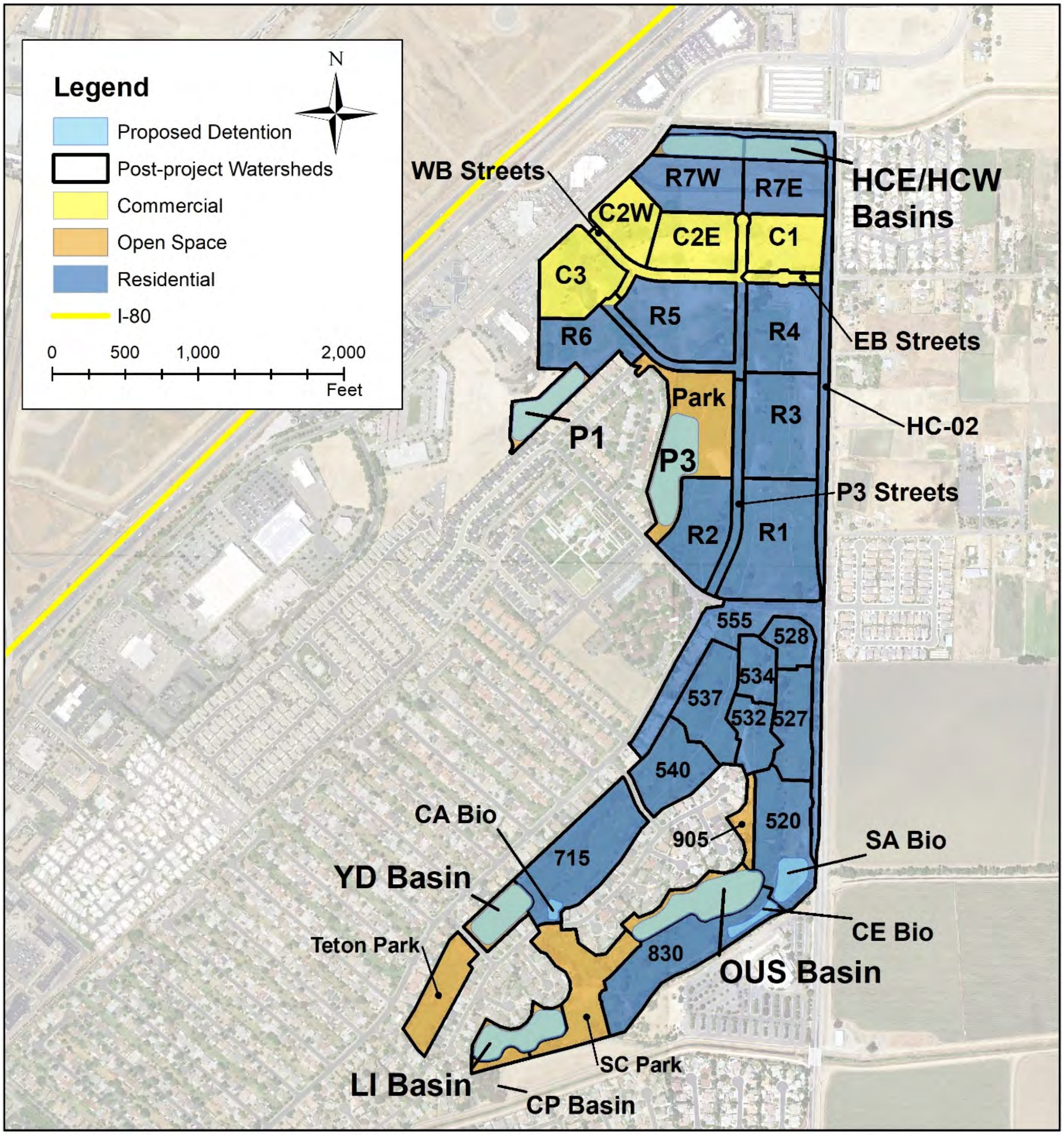

Figure 7-3 Post-project sub-watersheds. Land use classifications are based on current proposed land plan.

7.2 Hydraulic Modeling Methodology

Below are the key hydraulic model components and parameters.

Stormdrain System. The locations and elevations of the storm drains in the existing project area and the contributing off-site areas were field surveyed by Carlson, Barbee & Gibson to the extent feasible or accessible. Each storm drain is represented by a model link parameterized by the upstream and downstream invert elevation, diameter, length, and roughness set to 0.015 per City guidelines to account for minor losses at pipe junctions and inlets. Drainage ditches are represented by a natural channel link with a typical cross-section approximated using cross-section data. Storm drain links are connected by model nodes, which are parameterized by a basin invert elevation set to the lowest connected pipe, a spill elevation, assigned by the DEM elevation at the node location, and a ponding type. Nodes which interact with the 2D surface are linked so that overflow spills onto the DEM and ponding on the DEM enters the model node if there is capacity in the pipe. A 2D inflow capture relationship is entered for each model node such that discharge (Q) is a power law function of water depth (D) in the 2D grid cell, $Q = aD^b$, where a is equal to 23.7 for a standard inlet and 50 when a single node likely represents a collection of catch basins, and b is equal to 0.8. Nodes that represent ponds or drainage ditches and are connected to the 2D surface via 1D/2D interface lines use values of 100 and 1 for coefficients a and b, respectively, to maximize inflow into a single node that represents many linear feet of "inlet." These values are the maximum values allowed by the XPStorm software. In the 1D post-project model, nodes are set to "ponding allowed" so we can evaluate pipe overflows and volume is conserved.

In off-site area with no storm drain survey information, inlet catch basin locations were inferred either in the field or via aerial imagery and/or Google street view imagery. Invert elevations and pipe parameters were entered to minimize water ponded on the surface in areas where no flooding issues are known.

Ponds and Detention Basins. The existing pond network is represented in the pre-project model by a series of storage nodes which are parameterized by a depth-volume storage curve. The existing pond pipe network is modeled by individual pipe sections. The appropriate sediment depth is entered in pond outlet pipes when siltation in the ponds has raised the pond bottom elevation to higher than the outlet pipe invert. To avoid double counting the storage volume represented by a depression in the 2D DEM and the explicitly defined storage volume for each storage node, the pond areas are set to

"Inactive Areas" where 2D overland flow is not permitted. Instead, a 1D/2D interface and connection is parameterized around each pond node at the pond perimeter and overland flow is routed into the storage node directly.

Detention basins in the post-project model are parameterized by a similar depth-volume storage curve but are only linked to the 2D overland flow model if off-site flow is likely to enter a basin directly (e.g. P1 Basin). Each basin outlet is modeled as an orifice with a cross-sectional area optimized to meet peak flow requirements. In some cases, additional orifices are used at higher elevations to meet peak flow requirements for the 10- and 100-year peak flow requirements.

There are three proposed bioretention features adjacent to Street A, Court A, and Court E (SA BIO, CA BIO, and CE BIO in the model, respectively). In this case, the invert elevation of the storage node represents the underdrain elevation under the bioretention soil and rock drain material. We assumed a total soil and underdrain depth of up to 3.5 feet to account for 18 inches of bioretention soil and at least 12 inches of underdrain material and addition depth for the required cross-slope. In this bottom 3.5 feet basin depth, we have included storage volume equal to an assumed void space of 30 percent in both the soil and rock underdrain material. One additional outlet structure is added to bioretention basins which uses a Special Internal Rating Curve outlet function parameterized using the orifice equation which assumes a maximum flow rate of 5 inches per hour when the water surface elevation in the basin is equal to the top of the bioretention soil.

Leisure Town Road Overflow. When applicable, 2D overland flow over Leisure Town Road is quantified by connecting a 1D/2D interface line to a link and node outfall.

Receiving Tailwater Conditions. The project area drains to three channels: Horse Creek in the north, and Ulatis Creek and Old Ulatis Creek in the south.

Flow rate in Old Ulatis Creek is limited by the existing box culverts under Leisure Town Road and the geometry of the downstream agricultural ditch. To account for tailwater conditions in the existing pond network and proposed SDET basin, a tailwater curve was specified using a discharge-stage relationship. Discharge was calculated using the following assumptions about Old Ulatis Creek and were based on site observations:

- Channel bottom width of 5 feet

- Side slopes of 0.57:1
- Channel has trapezoidal shape
- Average channel slope of 0.0025
- Channel roughness of 0.08 based on low thin trees, shrubs, and other vegetation

This tailwater discharge-stage relationship was applied to both the pre- and post-project model.

Tailwater elevations in Ulatis Creek were derived from an unsteady-state HEC-RAS model provided by West Yost in January 2020, which simulated the major December 2005 storm event. A time series of water-surface elevation was extracted from the model for each of the five modeled outfalls within the model domain along Ulatis Creek. Given the limited capacity of the Ulatis Creek flood control channel and associated overbank flooding upstream, the December 2005 storm event represents the assumed maximum water surface elevation in Ulatis Creek in both the 10- and 100-year event.

The Horse Creek tailwater stage graph was derived using the Horse Creek gage data collected at Leisure Town Road from the December 2005 storm event, provided by West Yost. We received daily stage and flow data, and hourly flow data. To derive an hourly record of water-surface elevation to use as a tailwater condition for the two Horse Creek outfalls, we transformed the hourly flow record into hourly stage using the relationship between daily stage and flow (e.g. rating curve). Stage data was shifted down 0.7 feet to account for the difference between the recorded stage and flow depth. Flow-elevation curves were constructed for both modeled Horse Creek outfall location using the cross-section measured with the 2009 Solano County Lidar data and the HEC-RAS hydraulic design function. Manning's N values were solved for using known flow-elevation relationship at the gage location and used at the upstream outfall locations.

Inferred storm lag time between the rainfall and flow peak was estimated using the available rain and flow gage data provided by West Yost. Table 7-3 reports the time of peak rainfall and flow for both creeks. For Ulatis Creek, Rain Gage 21 at Mount Vaca is the most appropriate rain gage to use to calculate the Ulatis Creek lag time as mean annual precipitation at Mount Vaca is the highest in the watershed and therefore contributes a large proportion of the total runoff. Lag time between the Mount Vaca rain gage and flow at Leisure town is approximately 2.67 hours. Similarly, for Horse Creek, Rain

Gage18 is the available rain gage with the highest mean annual precipitation (30 inches per year) still in the contributing watershed. The lag between peak rainfall at Gage 18 and flow at Leisure Town Road is also 2.67 hours. Because flow data was provided at an hourly timestep, we rounded both lag times down to the more conservative 2 hours. As a result, the stage graphs for Ulatis and Horse Creek peak at a simulation time of 14 hours, with the design storm precipitation peaking at 12 hours. The modeled tailwater curves for the outfalls on Ulatis Creek and Horse Creek are in Figure 7-4 and Figure 7-5, respectively.

Table 7-3 Lag times for Ulatis and Horse Creeks for the December 2005 storm event.

Creek	Rain Gage	Rainfall Peak Time	Flow Peak Time	Lag Time
Horse Creek	18 BVRAIN	2005-12-31 4:20	2005-12-31 7:00	2.67
Horse Creek	19 WELL9RAIN5MIN	2005-12-31 6:15	2005-12-31 7:00	0.75
Ulatis Creek	21 MTVACA	2005-12-31 4:20	2005-12-31 7:00	2.67
Ulatis Creek	17 CCOGENRAIN	2005-12-31 5:40	2005-12-31 7:00	1.33

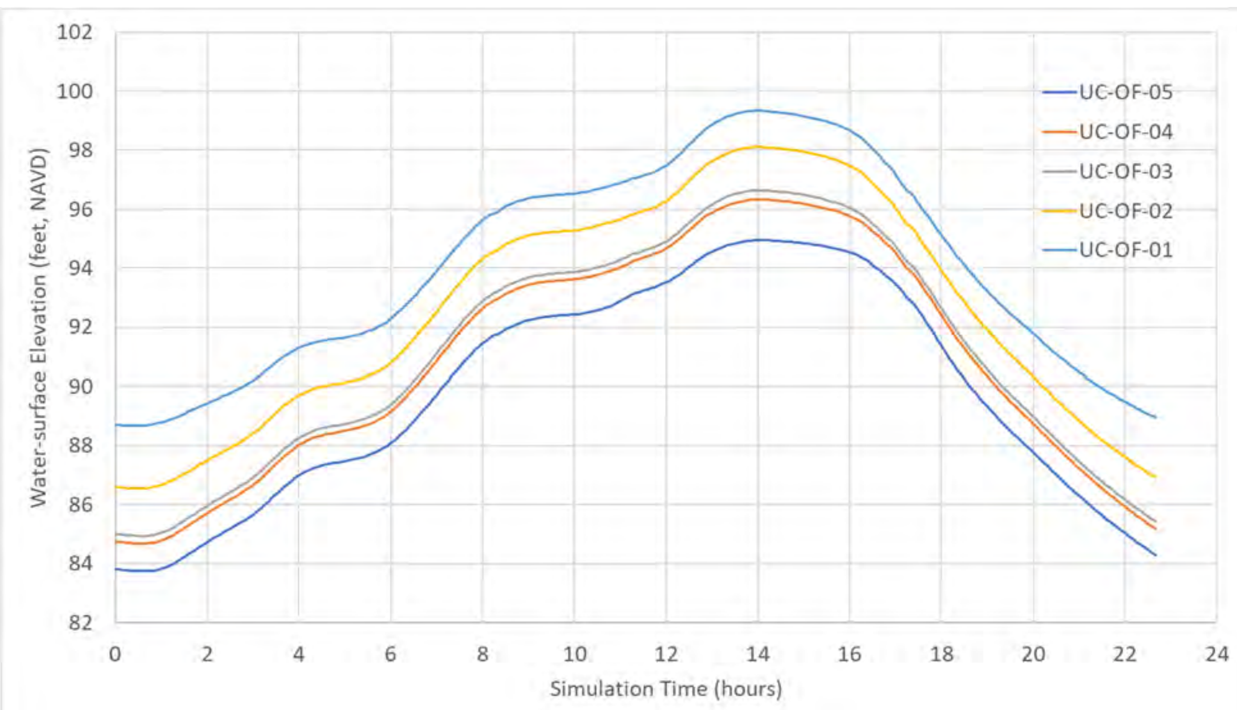

Figure 7-4 Ulatis Creek tailwater curves for the five modeled outfall locations.

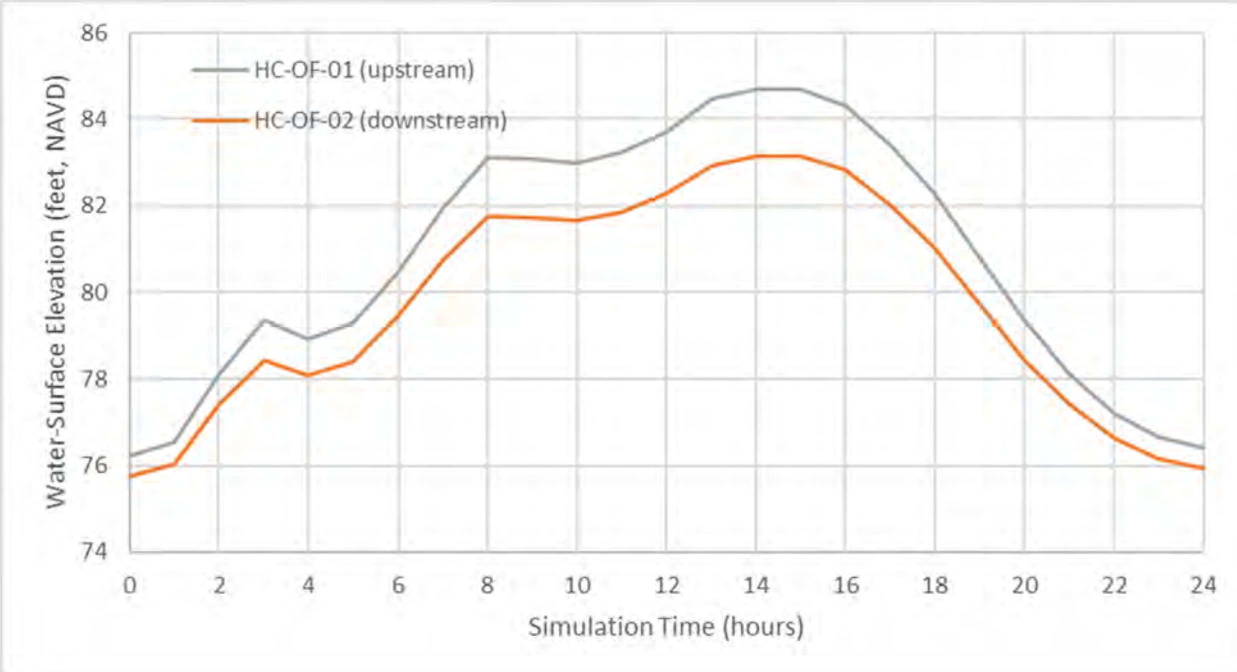

Figure 7-5 Horse Creek tailwater curves for the two modeled outfall locations.

8 DRAINAGE ANALYSIS AND MODELING RESULTS

8.1 Modeling Regulatory Requirements

To meet regulatory requirements, the proposed Project drainage infrastructure was designed to meet the full range of stormwater quantity management criteria. The proposed Project site drains to Ulatis Creek, Old Ulatis Creek, and Horse Creek, all downstream of Peabody Road, and so the proposed Project must adhere to the City of Vacaville Engineering standards (DS-4, 2006) such that 10- and 100-year post development peak flows shall be reduced to pre-development levels. Demonstration of meeting these constrains was accomplished using three primary points of compliance (POC): 1) the dual 4’ x 5’ box culverts in Old Ulatis Creek at Leisure Town Road, 2) runoff into Horse Creek from the Project site and contributing watershed area, and 3) runoff into Ulatis Creek from the Project site and contributing watershed area.

8.2 Existing Off-site Drainage Issues

The pre-project model results appear to be consistent with the known off-site drainage issues in the 10-year event. Particularly, this includes flooding in excess of 0.5 feet deep along Yellowstone Drive, and on Sequoia Drive near Leisure Way. Model results in the 100-year event indicate deeper inundation depths on Yellowstone Drive and Sequoia Drive, but also inundation greater than 0.5 feet on White Sands Drive, Teton Drive, and Ponderosa Drive. Flood inundation maps for the pre-project 10- and 100-year event are in Appendix A.

Drainage through the Green Tree Golf Course encompasses such a large and already developed off-site area that the drainage infrastructure through the proposed project has the potential to play a key role in drainage patterns off-site as well. As a result, one main objective for the proposed project is to design on-site storm water infrastructure that does not make the existing flooding issues worse, and to the extent possible, improves overall drainage functionality and reduces existing flooding for adjacent properties.

Therefore, the modeling included analysis of the hydraulic controls for frequently observed localized flooding along Yellowstone Drive, as this issue is of primary importance for City staff. The loop streets neighborhood from Lassen Circle to Rushmore Drive drain into the existing trunk storm drain in Yellowstone Drive. The north and south branch of the Yellowstone Drive pipe meet at Carlsbad Circle and drain directly into Pond 6 under pre-project conditions (Figure 8-1). Figure 8-2 shows the hydraulic grade line (HGL) through the southern branch of the Yellowstone storm drain, the storm drain easement leading off the street, and into Pond 6.

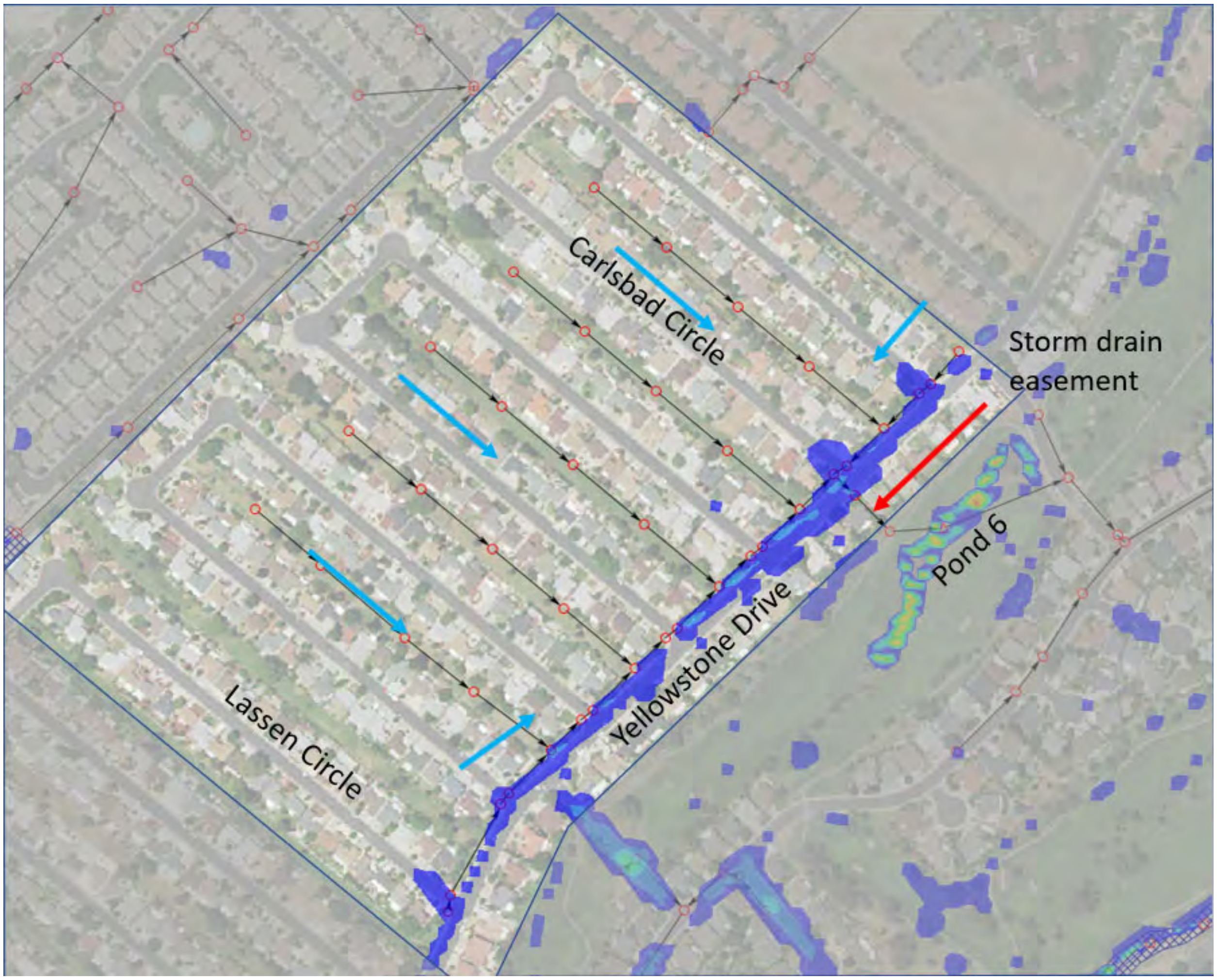

Figure 8-1 Pre-project, 10-year flooding along Yellowstone Drive.

The sharp increase in the HGL elevation through the storm drain easement from Yellowstone Drive to Pond 6 suggest that pipe sizes are the hydraulic control on flood inundation along Yellowstone Drive and thus proposed storm drain infrastructure cannot completely alleviate flooding unless that pipe run is up-sized.

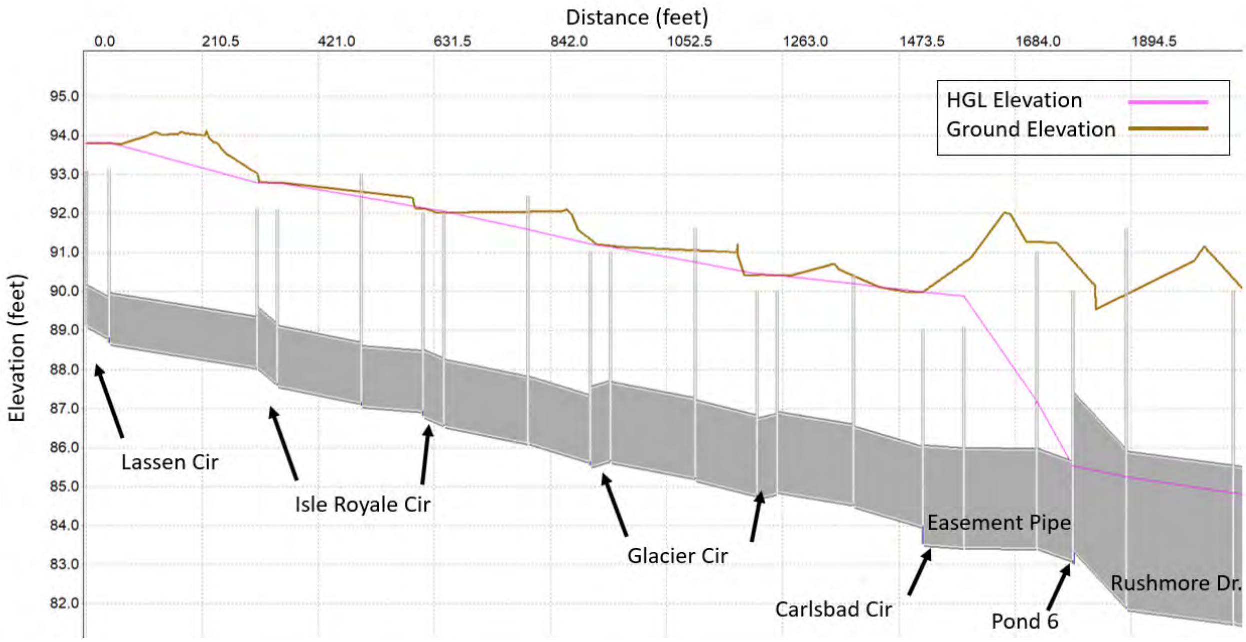

Figure 8-2 Pre-project, 10-year HGL along Yellowstone Drive from Lassen Circle to Carlsbad Circle and into Pond 6.

Currently the undersized pipes provide active (but unwanted) de facto detention storage along Yellowstone Drive. Under the assumption that the City will want to address the flooding along Yellowstone Drive and eventually increase the pipe sizes, we have completed our project detention basin sizing with Yellowstone pipe sizes that can effectively convey the 10-year storm event. This allows stormwater detention facilities to be sized appropriately now in anticipation that off-site drainage is improved, rather than assume long-term detention along Yellowstone Drive. The pipe sizes along Yellowstone Drive used in the post-project model are in Appendix B.

8.3 Pre- and Post-Project Modeling Results

Maximum flood inundation maps of the pre- and post-project models for the 10- and 100-year events are in Appendix A.

The pre-project pond system detention volume is summarized in Table 8-1. The reported detention volumes are only for water stored within the top of bank of each pond and does not include overland flow, which is temporarily stored in low points within the golf course boundary. Total detention volume within the ponds is 26.2 acre-feet and 37.0 acre-feet for the 10- and 100-year event, respectively.

Proposed post-project detention volumes are summarized in Table 8-2. Total required detention volume to achieve required peak flow attenuation is 46.4 acre-feet and 69.8 acre-feet for the 10- and 100-year event, respectively. However, the currently land plan includes accommodation for a total of 91.1 acre-feet of total detention, while still abiding by freeboard requirements. Note that P3 Basin has been modeled with a freeboard elevation of 86.6 feet, which is 0.1 feet higher than the current land plan and will be incorporated in the next land plan revision.

Table 8-1 Pre-project pond detention volume for 10- and 100-year events.

	10-year Peak Detention Volume *(ac ft)*	100-year Peak Detention Volume *(ac ft)*
Little Pond	1.1	1.1
Pond 1	1.4	2.5
Pond 2	2.1	3.0
Pond 3-4	12.1	15.1
Pond 6	1.1	1.9
Pond 7	4.7	7.7
Pond 8	3.8	5.7
Total	**26.2**	**37.0**

Table 8-2 Post-project detention volume for 10- and 100-year events.

	Detention Capacity			
	At Spillway Elevation *(ac ft)*	At Freeboard Elevation *(ac ft)*	10-year Peak Detention Volume *(ac ft)*	100-year Peak Detention Volume *(ac ft)*
HCE BASIN	9.1	8.4	6.1	7.7
HCW BASIN	10.2	9.5	7.1	8.8
LI BASIN	11.9	10.9	7.9	10.6
P3 BASIN	18.0	16.5	11.8	16.6
P1 BASIN	9.4	8.6	4.7	7.8
YD BASIN	13.7	12.9	2.7	6.2
OUS BASIN	25.1	24.3	6.1	12.0
Total	**97.4**	**91.1**	**46.4**	**69.8**

Despite appreciable changes in the drainage patterns from Old Ulatis Creek to Horse Creek, the proposed detention and storm drain infrastructure is sufficient to comply with the required peak flow reductions at each POC for both the 10- and 100-year event (Table 8-3); therefore, the Project complies with the City of Vacaville Engineering standards. Moreover, the size of the stormwater basins will allow for full compliance with the 2-year hydromodification requirement, outlined in SWRCB5's MS4 permit, as land use details are further refined.

Table 8-3 Pre- and post-project peak flow comparison for the 10- and 100-year events.

	10-year Peak Flow		**100-year Peak Flow**	
	Pre-project *(cfs)*	Post-project *(cfs)*	Pre-project *(cfs)*	Post-project *(cfs)*
Horse Creek	225.1	199.4	329.4	308.4
Ulatis Creek	52.5	37.2	59.5	48.7
Old Ulatis Creek	52.5	40.6	59.5	56.9
Leisure Town Road Overflow	0.9	0.0	0.8	0.0

8.3.1 Notable Drainage Features

After thorough investigation of drainage in and around the Green Tree Golf Course we have identified several drainage feature or processes that are worth noting. This exercise has served as a useful validation of the appropriateness of the model, and a valuable tool for documenting the existing drainage interactions in this interconnected system.

Ulatis Creek Overflow to Old Ulatis Creek. When Ulatis Creek water-surface elevations are high, the flap gate on the "City Pond" outfall is kept closed and so all upslope runoff is stored in the City Pond. Due to the relatively high height of the berm separating City Pond and the Ulatis Creek detention basin, overflow from City Pond is directed into a topographic low point in the existing golf course. This overflowing water is then conveyed through the low point directly into the head of Old Ulatis Creek. Based on the available elevation data, we estimate that this overflow in City Pond occurs when the water-surface elevation reaches approximately 93 feet; results suggest that this overflow path is activated during both the 10- and 100-year flood event.

Although no explicit changes to City Pond are proposed, the placement of the LI basin would alter the existing drainage pattern from City Pond to Old Ulatis Creek. To maintain the existing overflow drainage, the spillway elevation for the future LI basin will be set at 92.0 feet – a foot below the elevation that overflow currently occurs. This spillway elevation will allow back-flow between City Pond and the LI basin, and therefore maintaining the connection to Old Ulatis Creek. Moreover, this feature of the design has the added benefit of lowering the anticipated water-surface elevation in the neighboring White Sands Drive when Ulatis Creek tailwater elevations are high.

Leisure Town Road Overflow and Ditch. The existing ditch along Leisure Town Road primarily directs flows south and toward Pond 8. When the water-surface elevation exceeds the estimated north ditch spillway elevation of 83.6 flow is diverted north toward Horse Creek. This northern ditch directs a peak discharge of 27.9 cfs toward Horse Creek in the 100-year event, but is not wetted during the 10-year event. In the 100-year event, capacity of the northern ditch is exceeded and approximately 0.4 acre-feet is predicted to spill over Leisure Town Road.

Golf Cart Path Crossing at Grand Canyon Drive. When the Green Tree golf course was in operation, a golf cart path bridge was installed over the existing drainage ditch for access between the neighborhood and golf course. The bridge has three 18-inch culverts which will fill in with sediment without frequent maintenance. In the proposed post-project conditions, the existing drainage ditch is removed, and P1 Basin is constructed to intercept existing flows that enter the golf course at the intersection of Grand Canyon Way and Sequoia Drive. However, the north end of Grand Canyon Drive at the bridge crossing would still drain via overland street flow into the drainage ditch. When the land plan and storm drain infrastructure is finalized for the area adjacent to Grand Canyon Drive, we recommend consideration of the overland flows which would otherwise collect at the north end of Grand Canyon Drive.

8.4 Post-project Impacts on Off-Site Flooding

A comparison between the off-site pre-project flood inundation and the proposed post-project flood inundation in several problem areas is presented in Appendix C. The post-project model results show two conditions: the first assume that pipes along Yellowstone Drive and White Sands Drive are up-sized to convey the 10-year flood event; and the second – shown in Figure C2 – assumes the existing Yellowstone Drive pipes remain. In each of the cases, the proposed drainage infrastructure either improves or maintains the maximum flood inundation depth.

9 PROJECT INDUCED IMPACTS TO WATER QUALITY AND QUANTITY

9.1 Standards of Significance

Under accepted criteria for assessing impacts under the California Environmental Quality Act (CEQA, 2016), the Project would have a significant impact with regard to hydrology and water quality if it would:

- Violate any water quality standards or waste discharge requirements.
- Substantially deplete groundwater supplies or interfere substantially with groundwater recharge such that there would be a net deficit in aquifer volume or a lowering of the local groundwater table level (e.g. the production rate of pre-existing nearby wells would drop to a level which would not support existing land uses or planned uses for which permits have been granted).
- Substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, in a manner which would result in substantial erosion or siltation on- or off-site.
- Substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, or substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off-site.
- Create or contribute runoff water which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff.
- Otherwise substantially degrade water quality.
- Place housing within a 100-year flood hazard area as mapped on a federal Flood Hazard Boundary or Flood Insurance Rate Map or other flood hazard delineation map.
- Place structures within a 100-year flood hazard area which would impede or redirect flood flows.
- Expose people or structures to a significant risk of loss, injury or death involving flooding, including flooding as a result of the failure of a levee or dam.
- Inundation by seiche, tsunami, or mudflow.

9.2 Flooding and Discharge to Existing Receiving Waters

The increase in impervious surfaces on the Project site that is typically associated with development generally leads to increases in the rate and volume of stormwater runoff (Rantz, 1971). Depending on the characteristics of the watershed, such changes in runoff rates can have a number of adverse impacts without incorporation of the appropriate mitigation measures. The magnitude of the impacts is often directly related to the nature of the receiving waters into which the runoff is discharged. In the receiving waters, increases in runoff could increase possibility of flooding, particularly when increased runoff also increases flood peak discharges. Increased total runoff also has the potential to alter channel morphology through enhanced erosion and transport of sediments.

Both Ulatis Creek and Horse Creek are a FEMA regulated floodway and are heavily engineered channels. The channels have been straightened and lined with erosion control measures where necessary. Old Ulatis Creek originates at the Project site and joins Horse Creek approximately 2 miles downstream of Leisure Town Road. The channel has been straightened, but otherwise largely un-engineered; the channel banks have been colonized by local vegetation, including young trees, shrubs, and grasses. We have estimated that the channel capacity of Old Ulatis Creek downstream of Leisure Town Road to be approximately 110 cfs[8]. See Table 8-3 for modeled pre- and post-project peak discharges into Old Ulatis Creek.

The proposed Project would redirect a portion of the Project site runoff that previously drained to Old Ulatis Creek via the existing pond system to Horse Creek.

Table 9-1 summarizes the proposed changes in drainage area for each of the project receiving waters. Currently, only an estimated 32% of the drainage area (project area and contributing off-site drainage area) drains to Horse Creek and 44% to Old Ulatis Creek. Under the proposed stormwater drainage patterns, 41% of the total drainage area would drain to Horse Creek and 36% would drain to Old Ulatis Creek. This serves multiple purposes, including, but not limited to:

- Decreasing the effects of hydromodification on Old Ulatis Creek, which is not as heavily managed as Horse Creek; and

[8] Assumptions include bottom width of 5 ft., side slopes of 0.57:1, channel slope of 0.0025, roughness of 0.08, and height of 6.5 ft.

- Increasing runoff into Horse Creek may mitigate for overall increases in runoff by releasing more stormwater water prior to the peak discharge, which can lag rainfall in the relatively large watershed.

Although difficult to quantify because of overland flows which are routed to Old Ulatis Creek during high tailwater conditions, the total primary drainage area that drains to Ulatis Creek would not change under the proposed project.

Table 9-1 Pre-project and proposed post-project drainage areas for each of the receiving waters.

	Pre-Project Area		**Post-Project Area**	
	Drainage Area *(acres)*	Percent *(%)*	Drainage Area *(acres)*	Percent *(%)*
Horse Creek	227.8	32%	284.9	41%
Ulatis Creek	167.1	24%	167.1	24%
Old Ulatis Creek	308.1	44%	251.0	36%
Total	**703.0**		**703.0**	

The proposed Project will increase total runoff with development of Project site, but stormwater basins will be designed and built so that neither the 10- or 100-year flood events increase the peak discharges in either Horse Creek or Old Ulatis Creek, compared with pre-project peak discharges. The stormwater basins will also be designed to comply with 2-year hydromodification requirements.

9.3 Surface Water and Groundwater Quality

The changes in land use proposed by the Project can be expected to affect the types of loadings and constituents in the runoff that are ultimately released into Old Ulatis Creek and Horse Creek and perhaps infiltrate into the groundwater supplies. Urbanized areas tend to produce contaminants such as heavy metals, oils and greases, pesticides, nutrients from landscape fertilizers, and household chemicals that may transport into the creeks.

Both Horse Creek and Old Ulatis Creek eventually drain into Ulatis Creek, which was added to the 303(d) list as an impaired body of water in 2010. Ulatis Creek is impaired by two pesticides, chlorpyrifos and diazinon (California 303(d) List, 2010). Both pesticides were banned for residential uses in the mid-2000's but are still allowed for agricultural uses. During golf course operation, the pond system keeps runoff from smaller storms for use in irrigation. Larger storms are released from Pond 8. Pond network water surface elevation is controlled using flash boards and a flap gate. The ponds were operated as wet ponds, with water in each for visual appeal for the golf course. Existing water quality treatment is not intentionally managed. However, during smaller storms, some amount of fine sediments may be able to settle out of suspension as water is retained in the ponds. Without any specially engineered biofiltration soils, other contaminants are likely not removed from the water.

Under the proposed development, the required biofiltration will be built and maintained so that storm events with rainfall intensity up to 0.2 inches per hour will be treated prior to draining from the basin or bioswale. This water quality treatment will mitigate for suspended sediments as well as other contaminants not otherwise removed with only sediment settling. Groundwater quality is largely dictated by surface water quality, which is the source of groundwater infiltration. Thus, the treatment of the surface water should mitigate for groundwater quality in the event that surface water infiltrates into the subsurface.

9.4 Groundwater Quantity and Infiltration

In addition to increased runoff, the increase in impervious cover associated with the proposed Project may also reduce infiltration and percolation into the subsurface and groundwater aquifers, potentially decreasing groundwater recharge. The increased water demand from newly constructed housing may simultaneously tax the local groundwater supplies.

Because the northern portion of the Project site is largely soil type C, permeability of the soil is generally low (Figure 2-4). The largest aquifer under the project site is the Basal Tehama aquifer, where most of the City of Vacaville drinking water is sourced. Above the Basal Tehama is a series of alluvial deposits which is comprised of inter-bedded sands, silts, and clays, likely deposited by the channel network system as the creeks evolved and avulsed. Therefore, the permeability of the subsurface is likely highly variable, with low permeability floodplain deposits interspersed between higher-permeability sandy deposits. As a result, deep percolation of surface water is not likely to infiltrate to, and recharge, the Basal Tehama; any historical deep percolation and infiltration of surface water would contribute to shallow alluvial aquifers. The water balance model in Section 3.3 shows that the surface soil is only saturated above capacity in exceedingly wet winter months. During normal golf course operation, the Project site-sourced water for turf irrigation from the shallow alluvial aquifers, which was routed through the pond network. The ponds were operated as wet ponds and for visual appeal, but as most ponds are on type C soils, infiltration from the ponds was likely small. If Pond 6 or Pond 8 were constructed with a permeable floor, water stored in these ponds may infiltrate into the type B soils below.

The proposed Project will increase the amount of impervious cover, potentially decreasing the currently low rate of recharge to the shallow alluvial aquifers. As previously stated, city drinking water is sourced from the Basal Tehama and therefore, the proposed project would increase overall demand on the Basal Tehama aquifer while decreasing demand on the shallow alluvial aquifers. The City of Vacaville is required to plan for increases in water demand from population increases as part of the UWMP and GSP processes, and therefore may have sufficient capacity to support the proposed development.

The higher permeability soils in the southern region of the Project site (type B soils) have the potential to be used in conjunction with stormwater basins to mitigate for reductions in shallow alluvial aquifer recharge by focusing managed recharge efforts. Most of the proposed open space, including 2.4 acres of public park, is planned for the area south of Sequoia Drive where the type B soils within the project area are located, which will help maintain existing infiltration into the shallow alluvial aquifer. The proposed bioretention basins, BCT-BIO and SDET-BIO, are also located in the Type B soil area which will help maintain infiltration.

10 LIMITATIONS

This report was prepared in general accordance with the accepted standards of practice in surface-water hydrology and stormwater management existing in Northern California for projects of similar scale at the time the investigations were performed. No other warranties, expressed or implied, are made.

Concepts, findings and interpretations contained in this report are intended for the exclusive use of the Project proponents, at the site and for the purposes discussed therein, under the conditions presently prevailing except where noted otherwise. Their use beyond the boundaries of the site could lead to environmental or structural damage, and/or to noncompliance with policies, regulations or permits. They should not be used for other purposes without great care, updating, review of analytical methods used, and consultation with Balance staff familiar with the Project site.

As is customary, we note that readers should recognize that the interpretation and evaluation of factors affecting the hydrologic context of any site is a difficult and inexact art. Judgments leading to conclusions and recommendations are generally made with an incomplete knowledge of the conditions present. More extensive or extended studies, including hydrologic baseline monitoring, can reduce the inherent uncertainties associated with such studies. We note that many factors affect local and regional issues related to the management of stormwater from both a quantity and quality perspective. We have used standard environmental information -- such as rainfall, topographic mapping, and soil mapping -- in our analyses and approaches without verification or modification, in conformance with local custom. New information or changes in regulatory guidance could influence the plans or recommendations, perhaps fundamentally. As updated information becomes available, the interpretations and recommendations contained in this report may warrant revision.

To aid in revisions, we ask that readers or reviewers who have additional pertinent information of new plans, data or other information, who have observed changed conditions, or who may note material errors should contact us with their findings at the earliest possible date, so that timely changes may be made.

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APPENDICES

APPENDIX A

Inundation Flood Maps

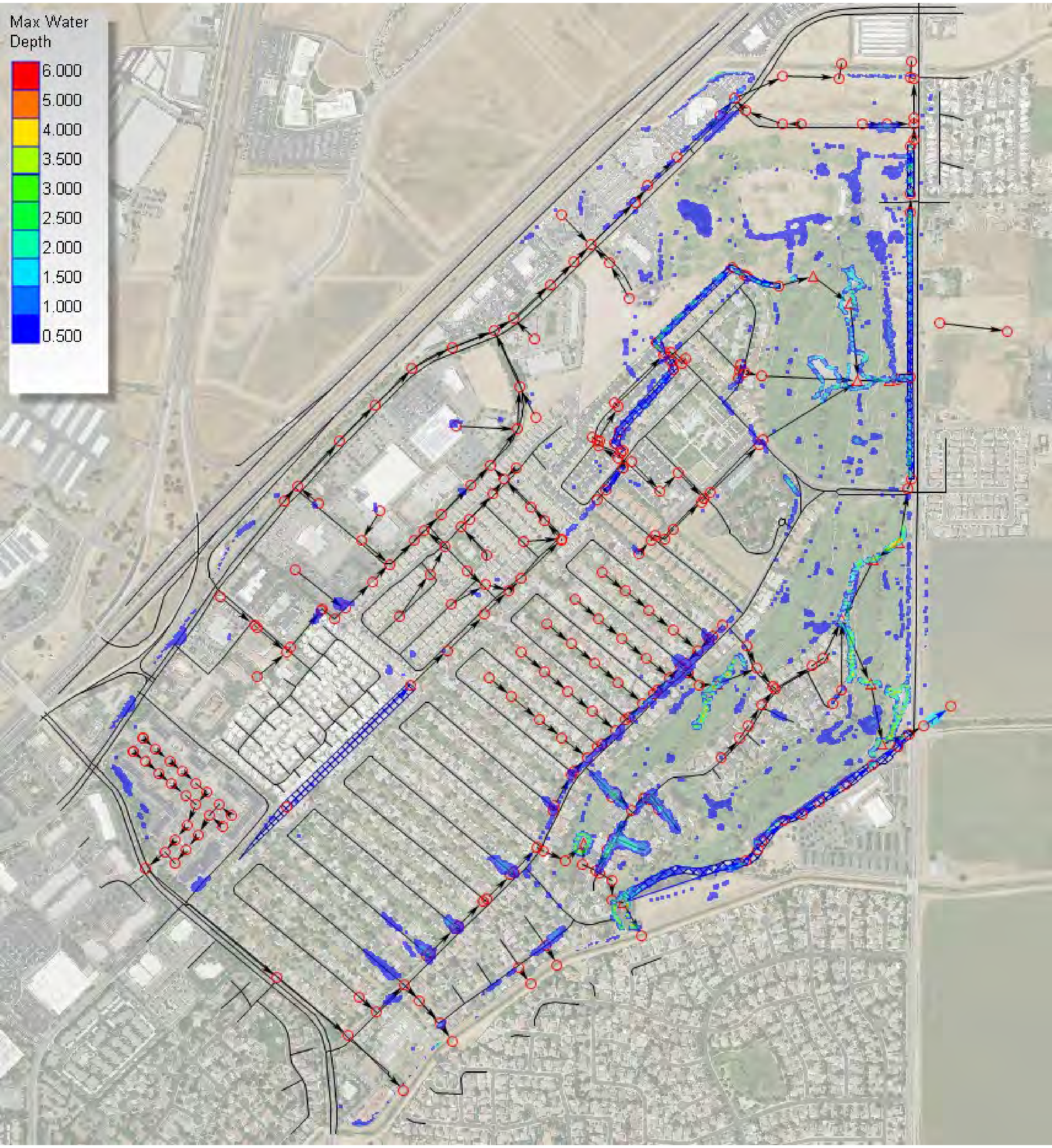

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Figure A1. Pre-project 10-year flood inundation. Inundation plotted for water depth > 0.5 feet.

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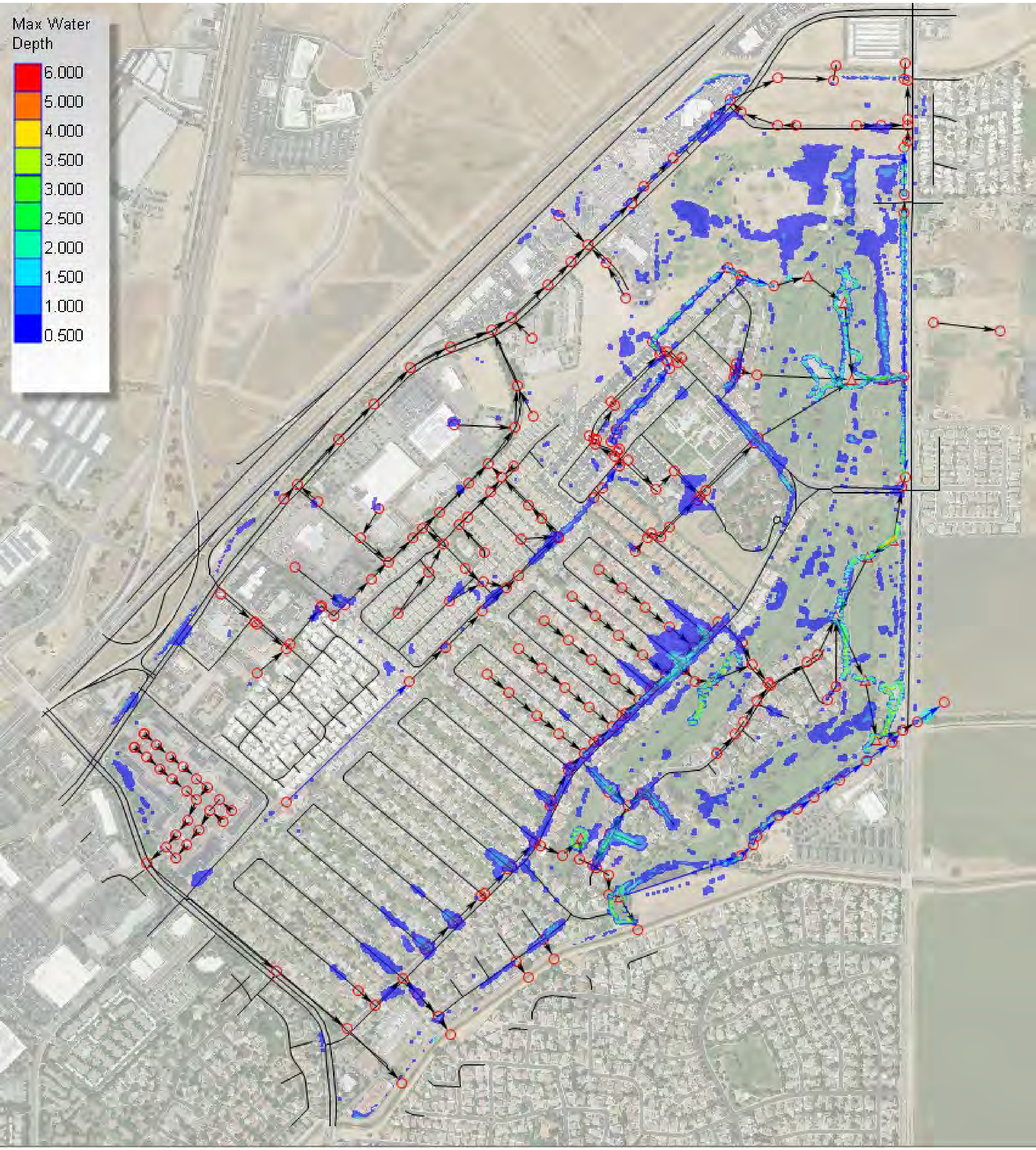

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Figure A2. Pre-project 100-year flood inundation. Inundation plotted for water depth > 0.5 feet.

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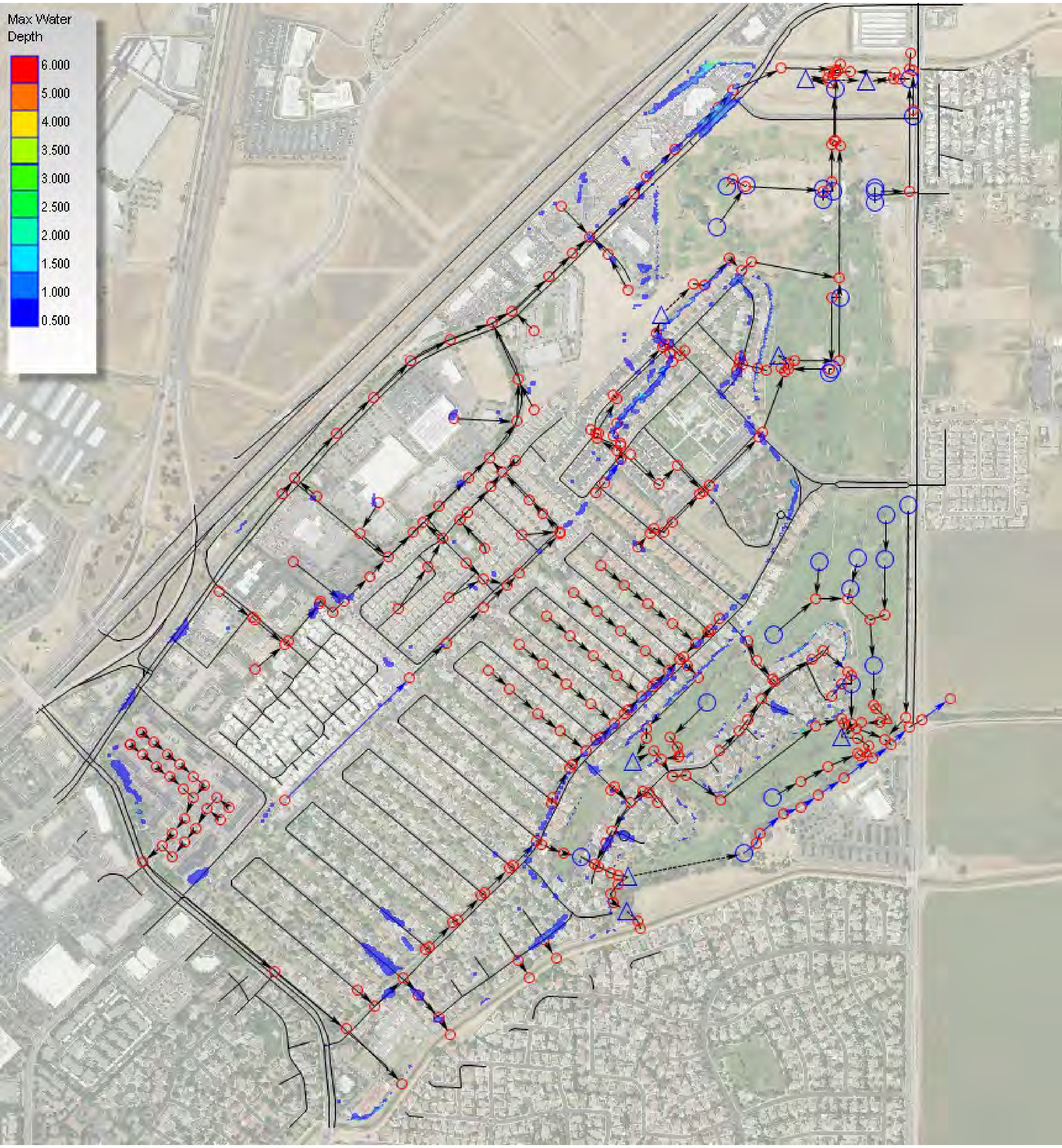

Note: Post-project model results include increased pipe sizes on Yellowstone Drive

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Figure A3. Post-project 10-year flood inundation. Inundation plotted for water depth > 0.5 feet.

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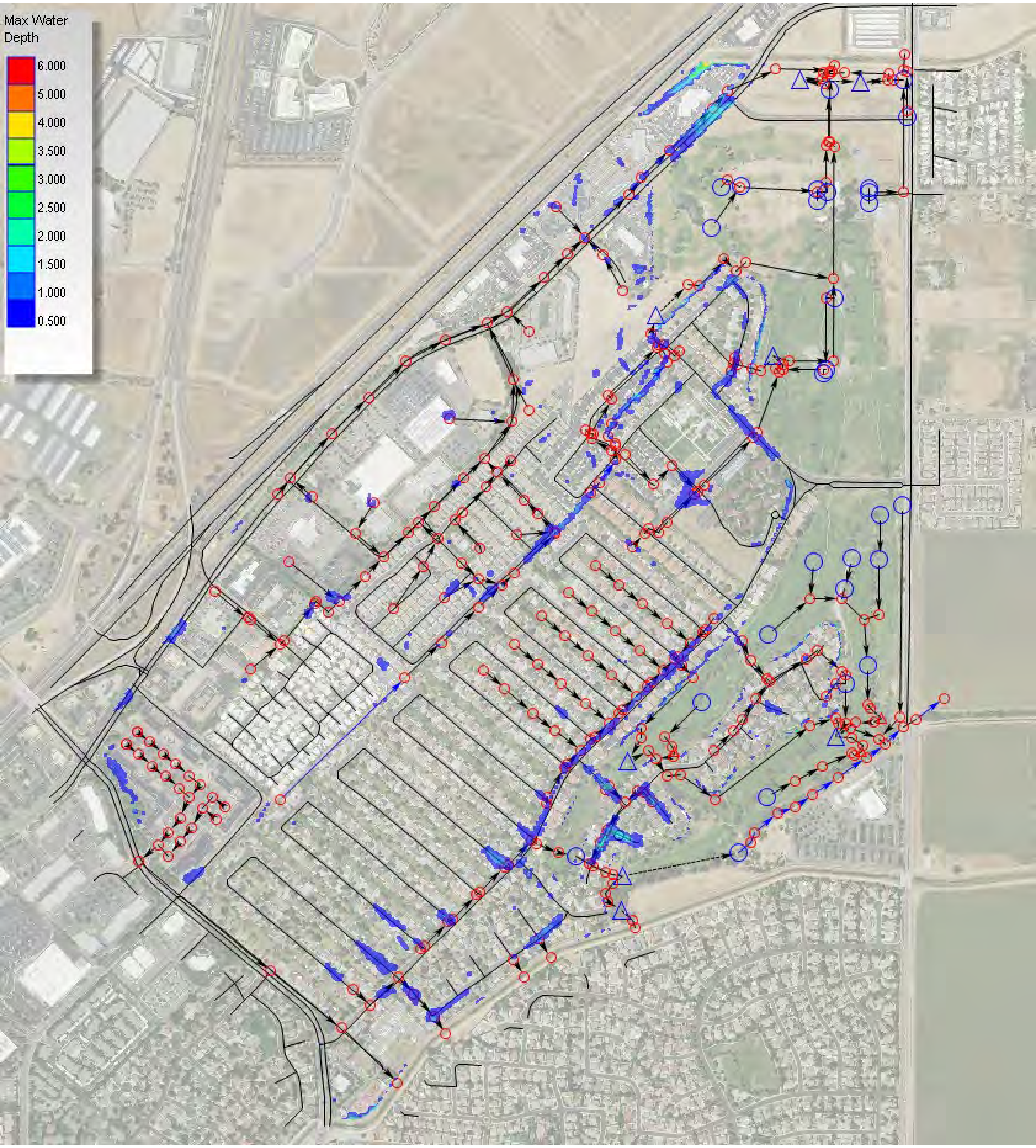

Note: Post-project model results include increased pipe sizes on Yellowstone Drive

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Figure A4. Post-project 100-year flood inundation. Inundation plotted for water depth > 0.5 feet.

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APPENDIX B

Yellowstone Drive Pipe Sizes

	P6-24 - P6-23	P6-23 - P6-22	P6-22 - P6-21	P6-21 - P6-20	P6-20 - P6-19	P6-19 - P6-18	P6-18 - P6-17	P6-17 - P6-16
Scenario	10-year	10-year	10-year	10-year	10-year	10-year	10-year	10-year
Link Name	P6-24 - P6-23	P6-23 - P6-22	P6-22 - P6-21	P6-21 - P6-20	P6-20 - P6-19	P6-19 - P6-18	P6-18 - P6-17	P6-17 - P6-16
Diameter (Hei	2.500	3.000	3.000	3.000	3.000	3.000	3.000	3.000
Length	42.380	267.840	36.200	152.420	112.240	37.560	152.440	112.510
Upstream Inve	89.100	88.925	87.821	87.672	87.043	86.691	86.574	86.096
Downstream I	88.925	87.821	87.672	87.043	86.691	86.574	86.096	85.747

	P6-16 - P6-15	P6-15 - P6-14	P6-14 - P6-13	P6-13 - P6-10	P6-10 - P6-09	P6-09 - P6-03	P6-03 - P6-02	P6-02 - P6-01-OF
Scenario	10-year	10-year	10-year	10-year	10-year	10-year	10-year	10-year
Link Name	P6-16 - P6-15	P6-15 - P6-14	P6-14 - P6-13	P6-13 - P6-10	P6-10 - P6-09	P6-09 - P6-03	P6-03 - P6-02	P6-02 - P6-01-OF
Diameter (Hei	3.000	3.000	3.500	3.500	3.500	3.500	4.500	4.500
Length	37.200	152.720	112.300	37.150	137.550	125.820	74.510	132.000
Upstream Inve	85.747	85.632	85.158	84.816	84.703	84.284	83.960	83.830
Downstream I	85.632	85.158	84.816	84.703	84.284	83.960	83.830	83.410

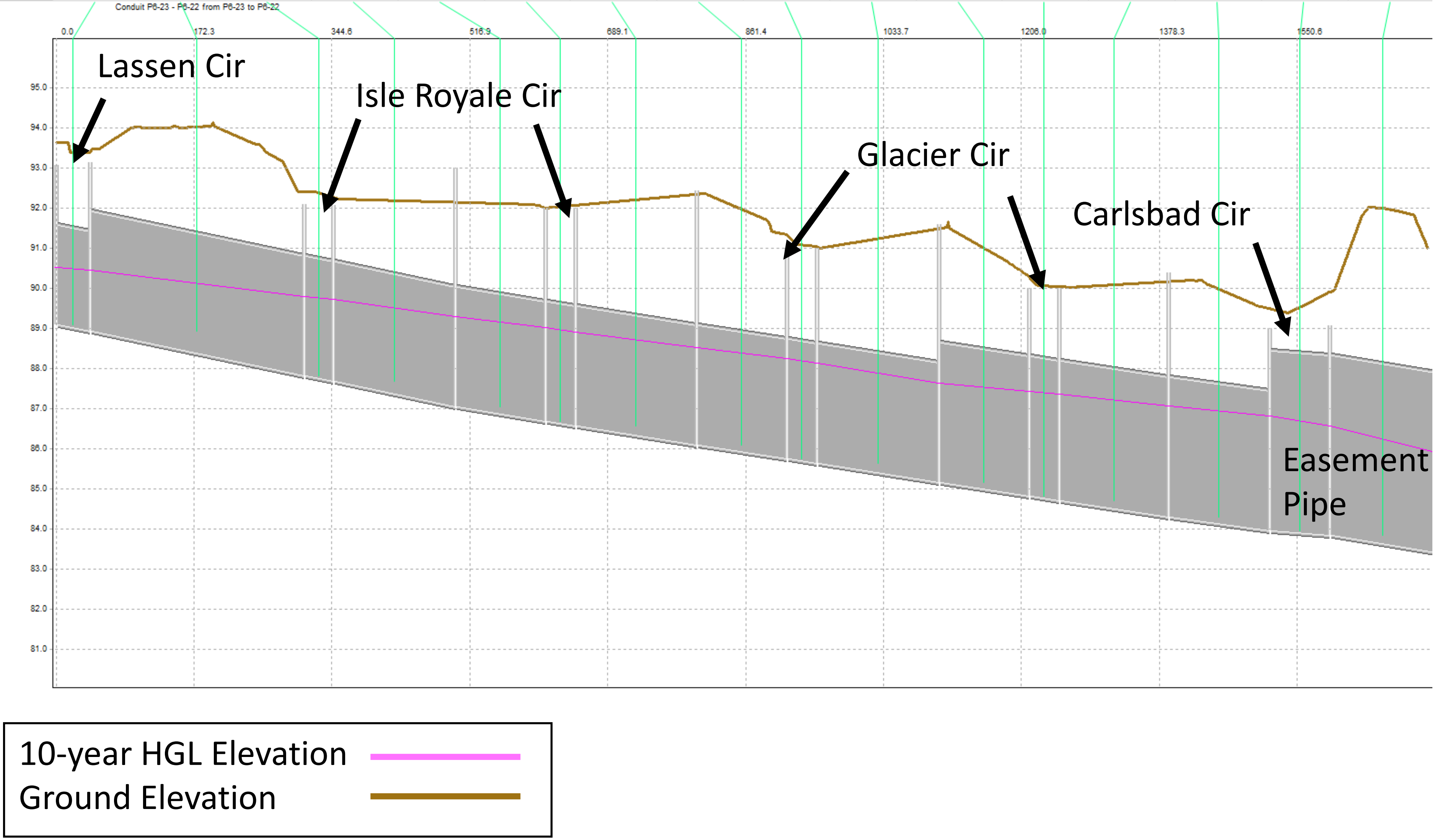

Figure B1. Increased pipe sizes on Yellowstone Drive from Lassen Circle through the storm drain easement adjacent to Carlsbad Circle. One possible pipe size increase; pipes could be additionally optimized to accommodate other underground infrastructure.

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	P6-08 - P6-07	P6-07 - P6-06	P6-06 - P6-05	P6-05 - P6-04	P6-04 - P6-03	P6-03 - P6-02	P6-02 - P6-01-OF
Scenario	10-year	10-year	10-year	10-year	10-year	10-year	10-year
Link Name	P6-08 - P6-07	P6-07 - P6-06	P6-06 - P6-05	P6-05 - P6-04	P6-04 - P6-03	P6-03 - P6-02	P6-02 - P6-01-OF
Diameter (Hei	2.000	2.500	2.500	3.000	3.000	4.500	4.500
Length	112.080	37.180	127.500	137.400	38.950	74.510	132.000
Upstream Inve	85.340	84.766	84.701	84.657	84.170	83.960	83.830
Downstream I	84.816	84.661	84.657	84.244	84.110	83.830	83.410

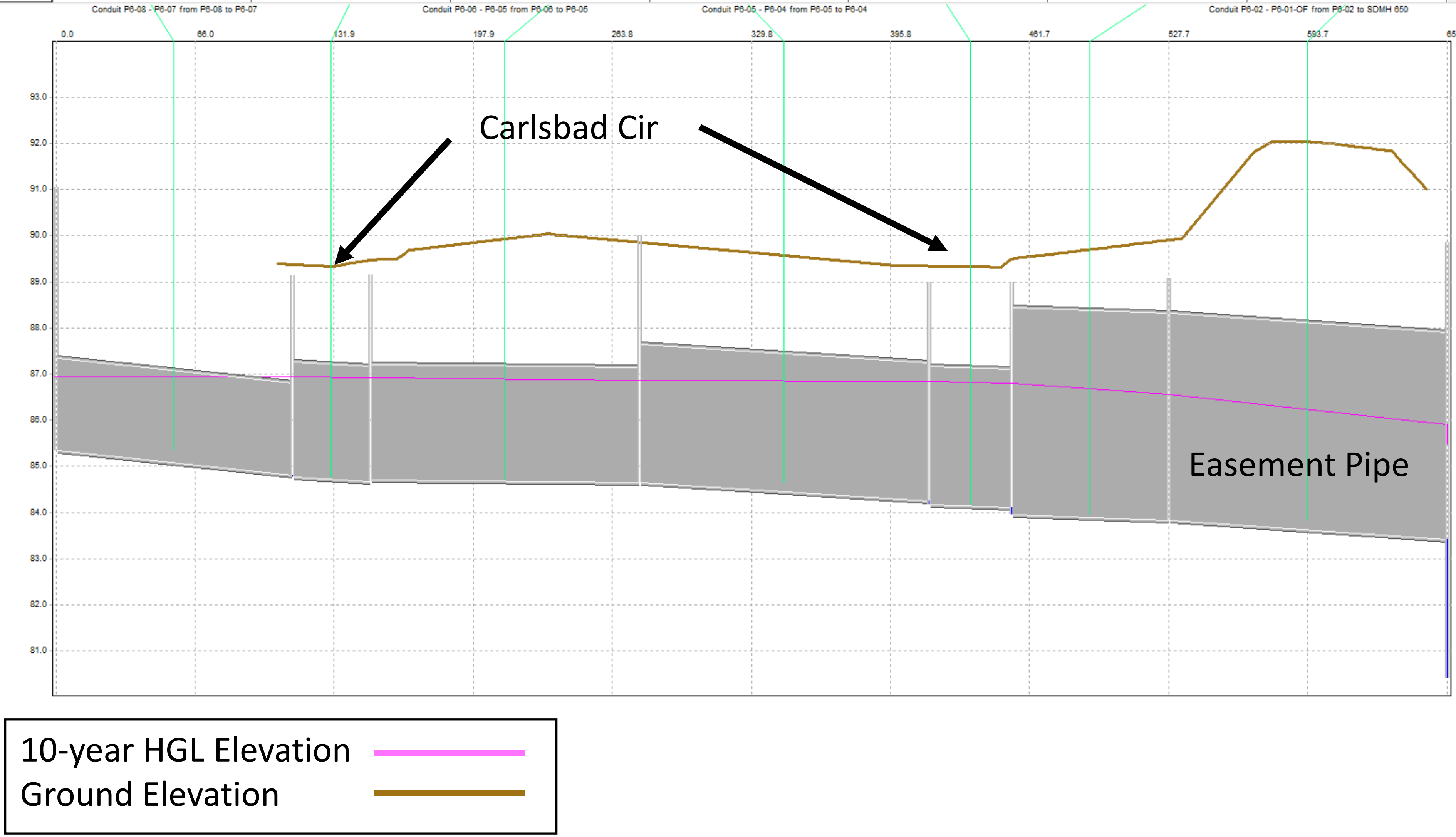

Figure B2. Increased pipe sizes on Yellowstone Drive from Rushmore Drive through the storm drain easement adjacent to Carlsbad Circle. One possible pipe size increase; pipes could be additionally optimized to accommodate other underground infrastructure.

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	LP-26 - LP-25	LP-25 - LP-24	LP-24 - LP-23	LP-23 - LP-22	LP-22 - LP-21	LP-21 - LP-20	LP-20 - LP-19	LP-19 - LP-18	LP-18 - LP-17	LP-17 - LP-16
Scenario	10-year	10-year	10-year	10-year	10-year	10-year	10-year	10-year	10-year	10-year
Link Descripti										
Link Name	LP-26 - LP-25	LP-25 - LP-24	LP-24 - LP-23	LP-23 - LP-22	LP-22 - LP-21	LP-21 - LP-20	LP-20 - LP-19	LP-19 - LP-18	LP-18 - LP-17	LP-17 - LP-16
Upstream Nod	LP-12	LP-11	LP-10	LP-09	LP-08	LP-07	LP-06	LP-05	LP-04	LP-03
Downstream	LP-11	LP-10	LP-09	LP-08	LP-07	LP-06	LP-05	LP-04	LP-03	LP-02
Diameter (Hei	2.500	3.000	3.000	3.000	3.000	3.000	3.000	3.000	3.500	3.500

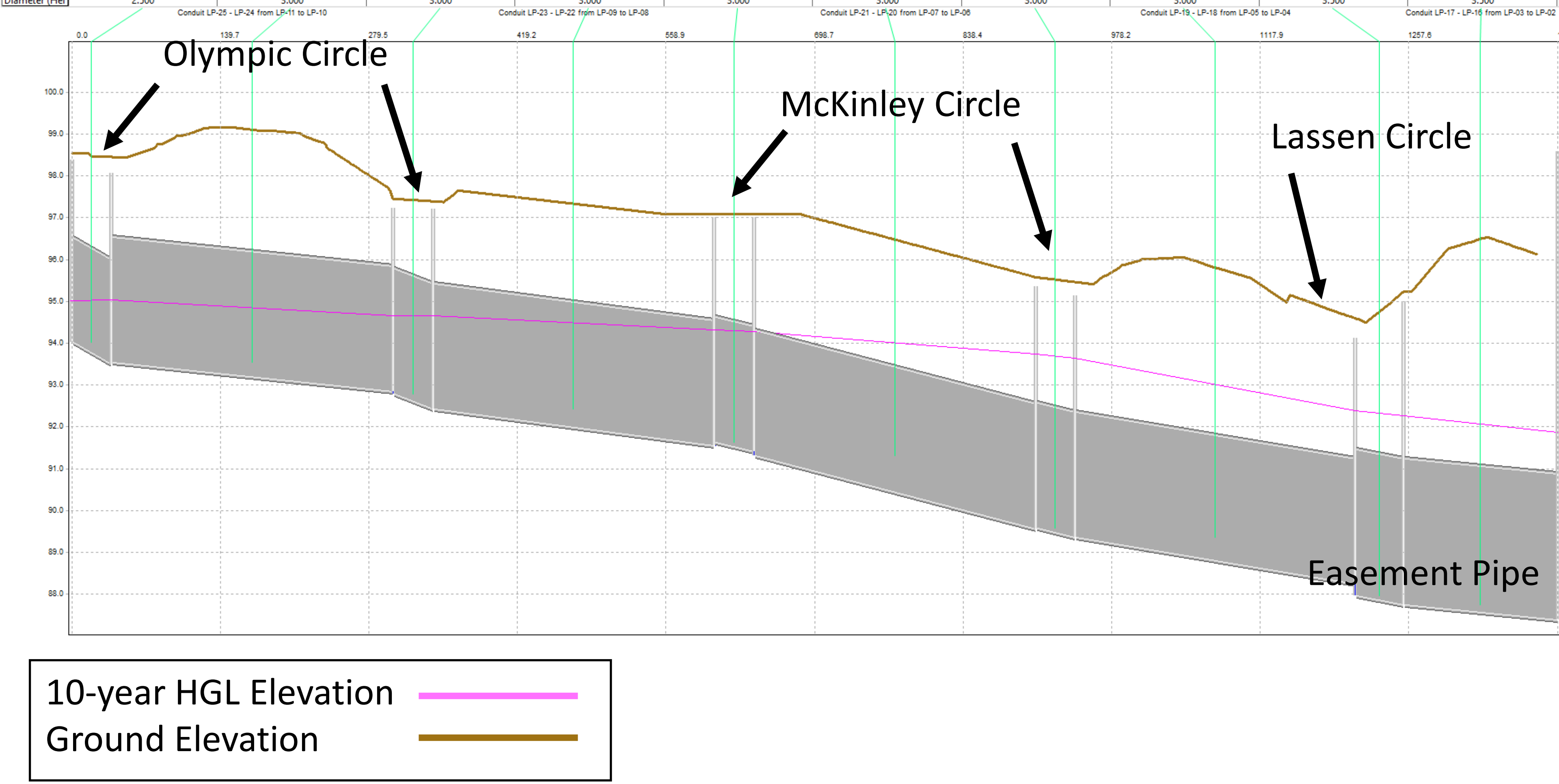

Figure B3. Increased pipe sizes on Yellowstone Drive from Olympic Circle through the storm drain easement adjacent to Lassen Circle. One possible pipe size increase; pipes could be additionally optimized to accommodate other underground infrastructure.

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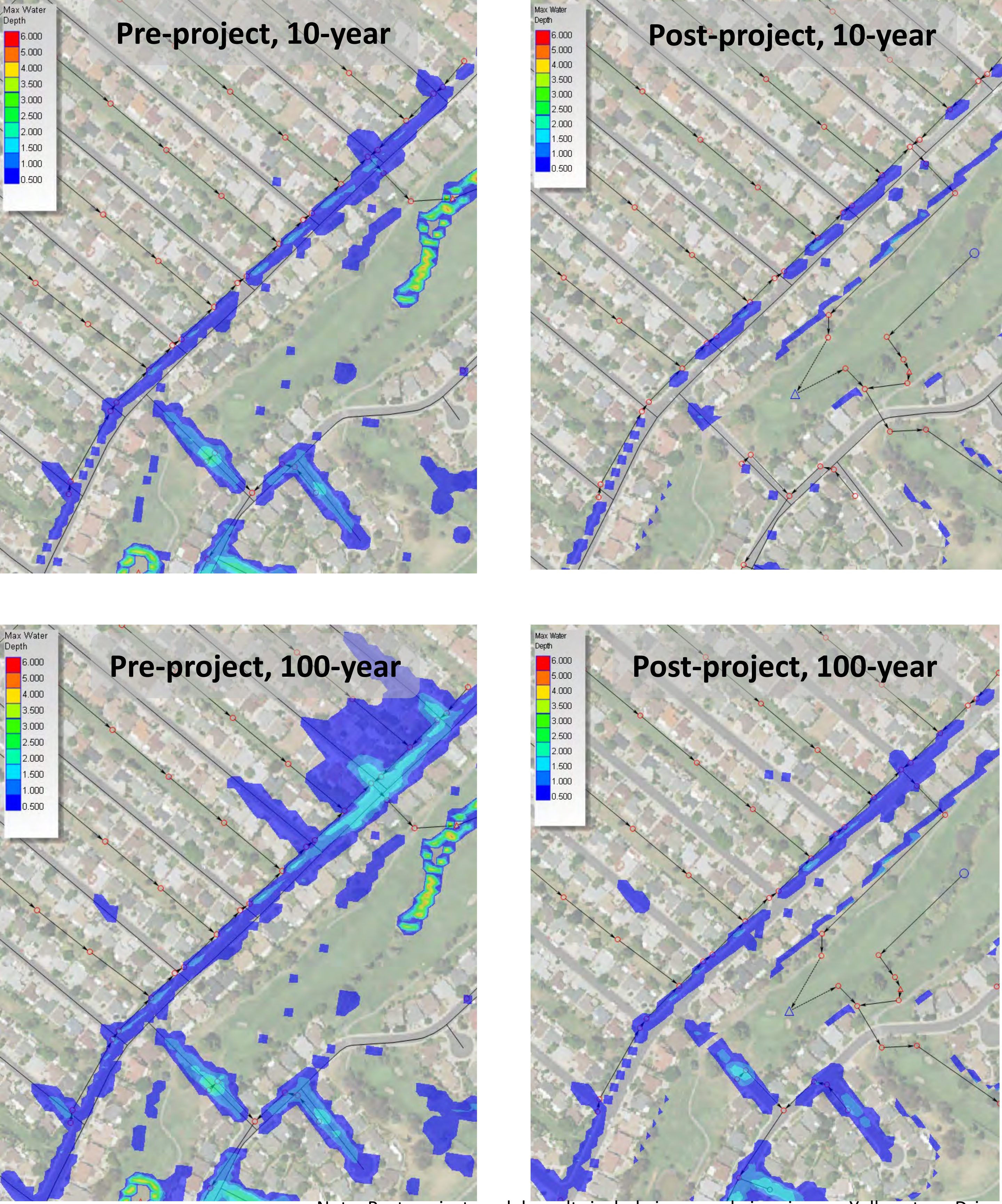
APPENDIX C

Flood Inundation Map –Problem Areas



Note: Post-project model results include increased pipe sizes on Yellowstone Drive

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Figure C1. Flood inundation on Yellowstone Drive with increased pipe sizes, draining to Pond 6. Inundation plotted for water depth > 0.5 feet.

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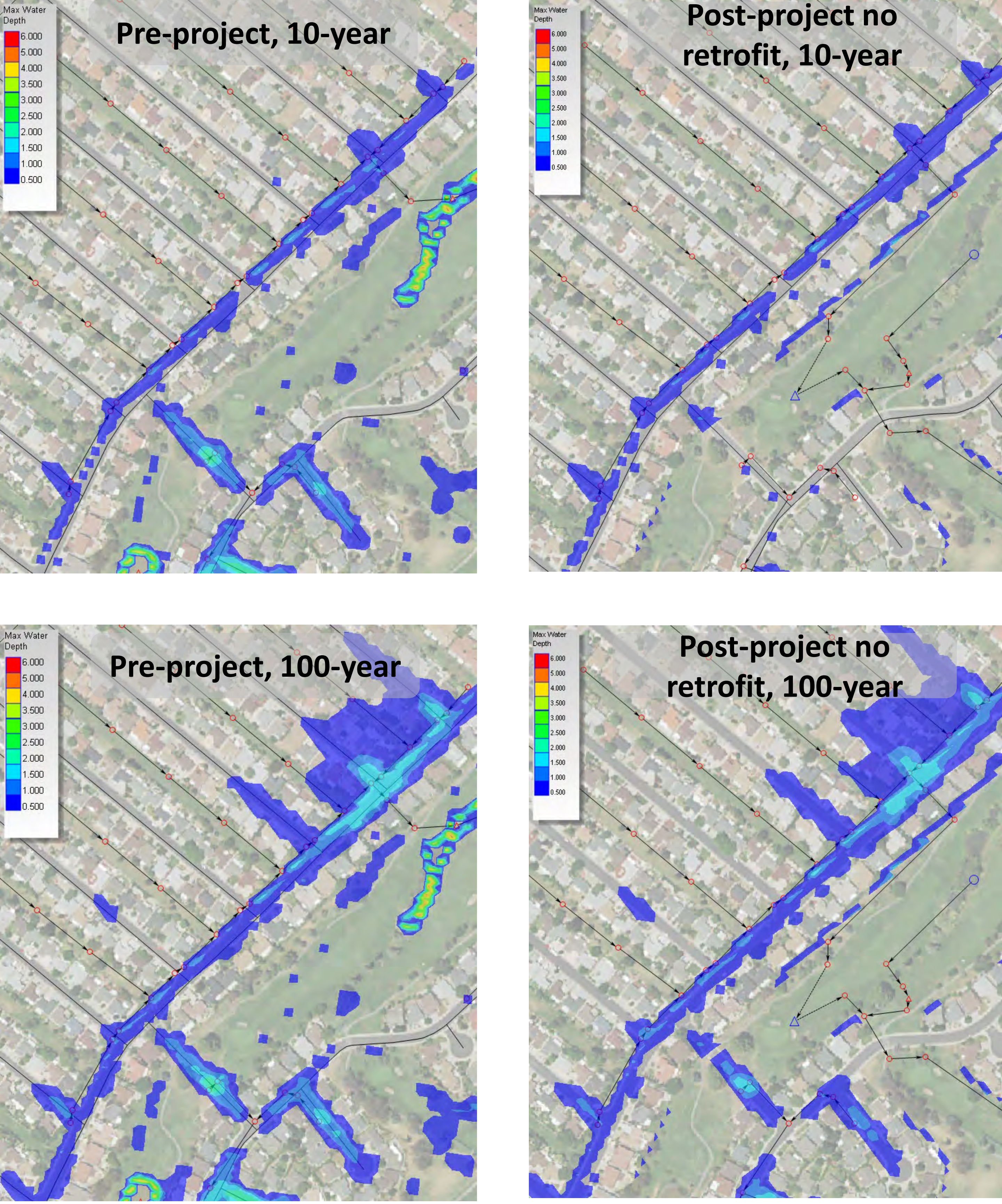

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Figure C2. Flood inundation on Yellowstone Drive with existing pipes, draining to Pond 6. Inundation plotted for water depth > 0.5 feet.

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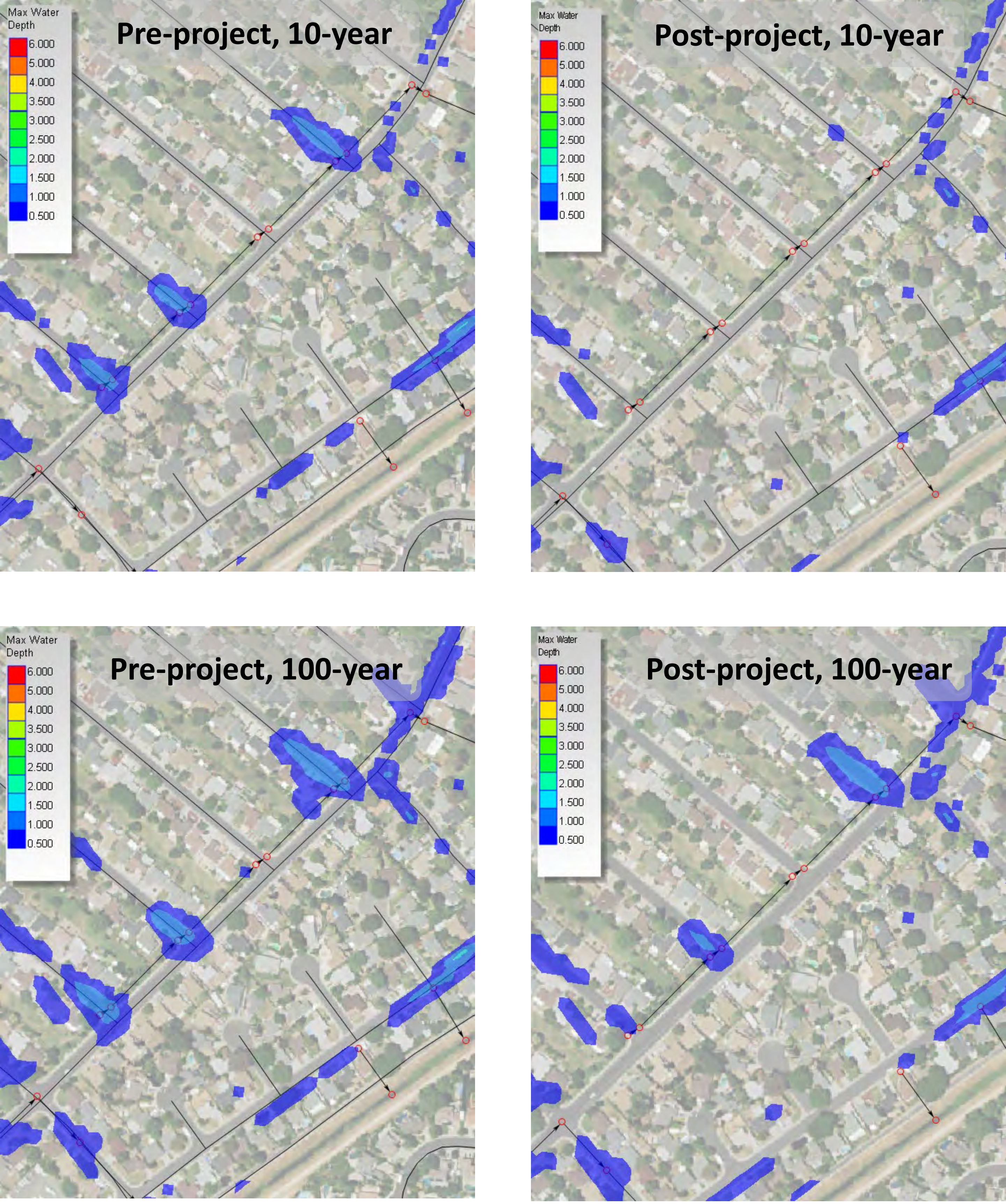

Note: Post-project model results include increased pipe sizes on Yellowstone Drive

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Figure C3. Flood inundation on Yellowstone Drive, draining to Little Pond. Inundation plotted for water depth > 0.5 feet.

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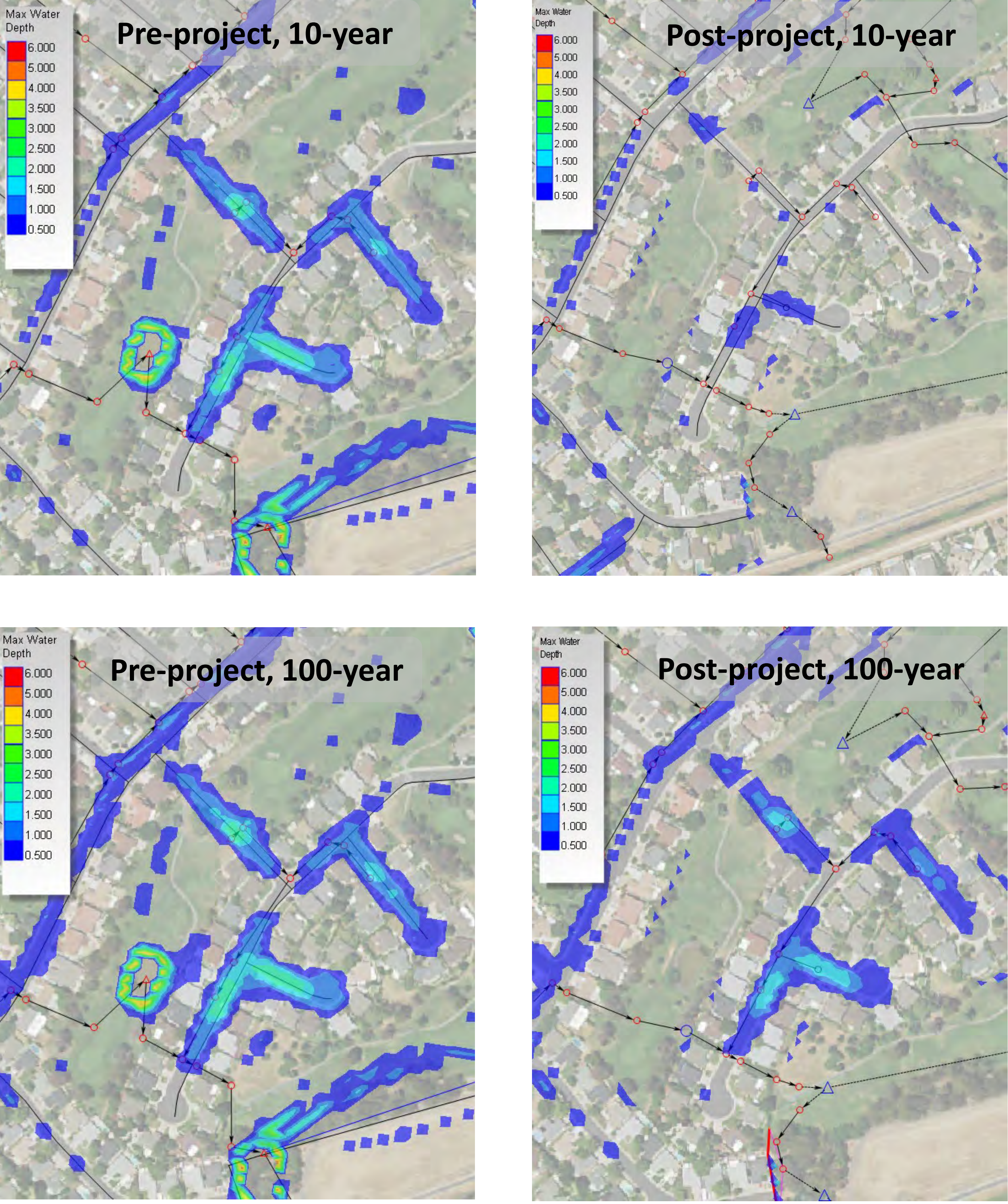

Note: Post-project model results include increased pipe sizes on Yellowstone Drive

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Figure C4. Flood inundation on White Sands Drive.
Inundation plotted for water depth > 0.5 feet.

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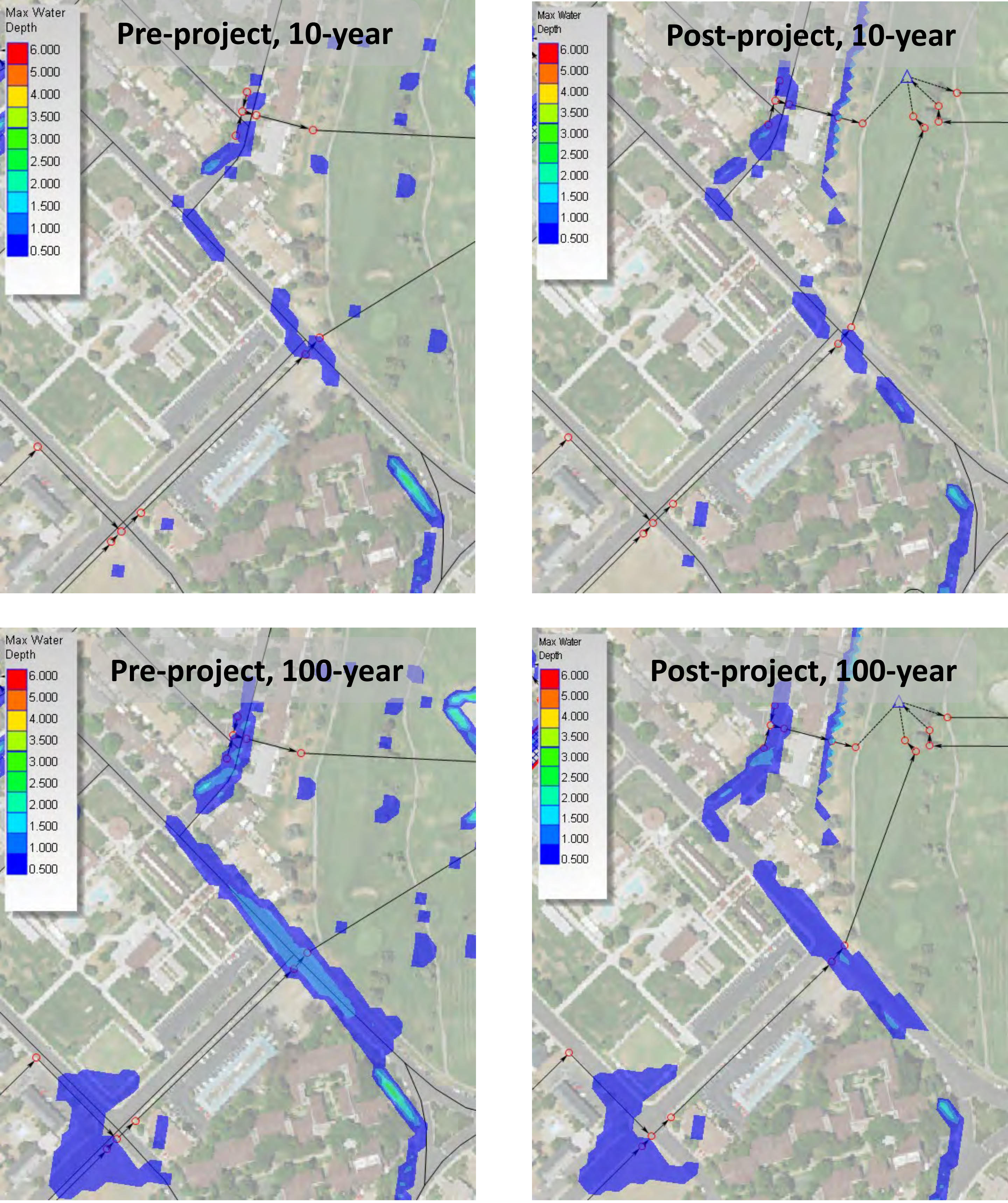

Note: Post-project model results include increased pipe sizes on Yellowstone Drive

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Figure C5. Flood inundation on Sequoia Drive. Inundation plotted for water depth > 0.5 feet.

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