CONCEPTUAL DRAINAGE MASTER PLAN

PROPOSED DEVELOPMENTS WITHIN SPHERE OF INFLUENCE

CITY OF GONZALES

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1 INTRODUCTION

1.1 Background

In 2014, the Local Agency Formation Commission approved the Sphere of Influence (SOI), for the City of Gonzales (City), most of which is contiguous to the northeastern existing City boundary, in Monterey County, California, southeast of Salinas and northwest of Soledad on the State Highway 101 (Exhibit 1). Development in this area is proposed as mostly residential, with supporting commercial, schools, and open space areas.

In 2010, a Conceptual Drainage Plan was prepared for the City by Rick Engineering Company. The plan recommended drainage improvements within the existing City limits. As part of the study, an inventory of the existing drainage infrastructure was conducted.

1.2 Purpose of Report

The purpose of this report is to develop a conceptual level, backbone stormwater masterplan for Vista Lucia, D'Arrigo Brothers, and Puente del Monte, proposed developments within the SOI, that establishes baseline conditions for each of the developments, provides conceptual level storm drain sizes, structures and channels for the new development, and determine impacts the new development will have on existing infrastructure. Since this is a planning level analysis, only conceptual level pipe and inlet geometry and basin design is provided. Final sizing and configuration of the system will be determined at a later date as each development moves forward in the design process.

1.3 Study Area

The study area encompasses a portion of the SOI that includes the developments referred to as Vista Lucia, Puente del Monte, and D'Arrigo Brothers and the drainage tie-ins to the City's existing drainage system (Exhibit 1). The existing land within the study area is primarily agriculture. See Exhibits 4 and 5 for existing and proposed land use designations.

Drainage generally flows in the southwesterly direction, originating in the hills to the northeast of the City. Within the City, the drainage system is comprised of a combination of streets, street inlets, and storm drains that collect and route the flow to Gonzales Slough, which bisects the City and flows northwesterly, originating just upstream of the intersection of Gloria Road with US 101.

Under existing conditions, stormwater within the Vista Lucia, D'Arrigo Brothers, and Puente del Monte study areas flows as shallow overland flow or within existing agricultural ditches. As these areas are developed, runoff volumes are anticipated to increase.

1.4 Drainage, Stormwater, and Floodplain Development Criteria and Constraints

The City's storm drain design criteria as provided in Part II, Section II, of the City of Gonzales Design Standards dated 1995 are as follows:

- Use of the Rational Method for sizing stormwater facilities. Drainage pipes should be sized for the 20-year design storm for commercial, industrial, and major trunklines. (Note: since this is a conceptual plan for the backbone system, the 25-year design storm was used for conservativeness).
- Retention facilities are required to mitigate increase in runoff for the 100-year design storm.

Stormwater Quality Design Criteria - Projects within the City are subject to Central Coast Regional Water Quality Control Board (CCRWQCB) Resolution No. R3-2013-0032 post-construction stormwater management requirements. An Amendment was adopted by the State Water Board on December 19, 2018, however, the Amendment is not official until the State Water Board clerk certifies the Amended Small MS4 General Permit which was not done by the date of this report (https://www.waterboards.ca.gov/water_issues/programs/stormwater/phase_ii_municipal.html#phase). The City is located mostly in Watershed Management Zone 4 with a small portion in the north located in Zone 1. For large projects (greater than 22,500 square-feet impervious) the stormwater treatment requirements require, along with site design measures, the following:

- Treatment of at least 0.2 inches per hour intensity,
- Retention of the 95th-percentile rainfall event, and
- Peak flow management by applying mitigation measures that mitigate post-development peak flows, discharged from the site, to at least pre-project peak flows for the 2- through 10-year storm events (this is the hydromodification requirement).

Portions of the study area lie within a FEMA Zone A (Exhibit 14), areas with a 1% annual chance of flooding for which no base flood elevations have been determined (FEMA Effective Map Numbers 06053C0414G, 06053C0418G, 06053C0425G, and 06053C0581G dated April 2, 2009). FEMA is in the process of conducting a remap for the area, however, the study was not completed by the date of this report. Floodplain Development Standards (Title 14 Flood Control, Chapter 14.04 Floodplain Management) for the City require:

- A Floodplain Development Permit to be obtained before construction or other development,
- Construction standards in all areas of special flood hazards, as listed in Section 14.04.160, include but are not limited to:
    - For residential construction in Zone A, all new construction shall have the lowest floor, including basement, elevated to or above the base flood elevation. In Zone A, base flood elevations may be obtained using methods from the FEMA publication FEMA 265, “"Managing Floodplain Development In Approximate Zone A Areas - A Guide For Obtaining And Developing Base (100-Year) Flood Elevations" dated July 1995.

- For nonresidential construction, all new construction shall be elevated to conform with the residential requirements or meet Section 14.04.160.C.2 – Elevation and Floodproofing for Nonresidential Construction.

1.5 Methods

Hydrologic and hydraulic modelling for the proposed project site was conducted using Computational Hydraulics, Inc's (CHI) PCSWMM version 7.2. PCSWMM is an interface used to set up and run the U.S. Environmental Protection Agency (USEPA) Stormwater Management Model (SWMM). HEC-HMS was used to estimate the majority of the offsite hydrology upstream of the Study area (Exhibit 2). PCSWMM was used to develop existing and proposed conditions hydrology and hydraulic models within the Study Area and for a small portion of the offsite area (Exhibit 2).

Existing storm drain system horizontal data was obtained from the City of Gonzales Conceptual Drainage Plan prepared by Rick Engineering Company, dated February 2010, and as-built data provided to Kimley-Horn and Associates by the City of Gonzales.

Topographic data used to delineate basins was obtained from the following sources:

- USGS National Elevation Dataset as 1 arc-second (approximately 30 meter) Digital Elevation Model (DEM), and
- 2010 Lidar Data (approximately 10-foot resolution) provided by Association of Monterey Bay Area Governments (AMBAG)

Existing conditions land uses were developed from aerial photography and the City of Gonzales Master Plan data provided by Monterey County GIS download and were categorized based on NRCS Land Use classification system. Proposed conditions land uses within the study area were provided by Kimley-Horn and Associates. (Exhibits 4 and 5)

Rainfall depths and intensities used to model the hypothetical design storms were developed using data from the NOAA Atlas 14, Volume 1, Version 5, revised 2011 (Bonin et al., 2011).

2 OFFSITE HYDROLOGIC MODELING

Hydrologic modeling was conducted using HEC-HMS 4.2.1 with Green and Ampt watershed abstractions, the SCS Unit Hydrograph rainfall transformation, and the Muskingum-Cunge hydrograph routing.

2.1 Subbasin Characterization

The offsite hydrology (HEC-HMS) study area is summarized in Exhibit 2. A total of 15 subbasins contribute drainage to the Study Area. The basins were delineated using USGS topography. GIS software was used to develop model input parameters, such as subbasin area, Green and Ampt parameters, and percent impervious. GIS layers representing subbasin boundaries, soil data, and land use data were used to calculate area-weighted averages for each desired parameter for each subbasin.

2.2 Watershed Abstractions

Watershed abstraction is a term used to describe the collective precipitation losses throughout the watershed that occur during a storm. These losses play a significant role in rainfall-runoff modeling as they determine the amount of rainfall excess, or direct runoff, produced by the storm within the model. Typical losses abstracted from rainfall include: *soil infiltration, landscape interception, depression storage (aka: surface storage), evaporation, and evapotranspiration.* The rainfall volume attributable to these losses is not converted to direct runoff.

2.2.1 Interception, Evaporation, and Transpiration

For this study, the design storms represent single storm events of relatively short duration that typically occur during the fall and winter months. Therefore, losses such as interception, evaporation, and transpiration by vegetation are considered minor and were not considered in this study.

2.2.2 Infiltration

Infiltration is the process by which precipitation is abstracted by seeping into the soil below the land surface. Soil infiltration was estimated using the Green and Ampt method, which applies Darcy's law and principle of conservation of mass to estimate infiltration. The method works under the assumption that water enters the soil as a sharp, vertical wetting front that travels as a function of the hydraulic conductivity. The following parameters, which are estimated as a function of soil texture (Table 1), are used:

- Saturated hydraulic conductivity (Ksat) – The infiltration rate of the soil once it is saturated, in inches per hour.
- Wetting front capillary suction (PSIF) – Characteristic suction head of the soil, in inches.
- Soil moisture – Initial soil water deficit is represented as the difference between the Saturated Moisture Content and the Initial Moisture Content. These dimensionless values represent the initial soil moisture conditions.

2.2.2.1 Soils Data – HEC-HMS Model Area

Soils data, used for offsite hydrology modeling (HEC-HMS) of infiltration rates, were obtained in digital format from the NRCS Soil Survey Geographic (SSURGO) Database for the Monterey County Area, California (NRCS, 2009) and compiled into GIS shapefiles. There are 27 distinct soil classes, referred to as Map Unit Keys (MUKEYs), within the HEC-HMS model area. Each MUKEY is composed of one or more soil components that are further categorized based on soil texture. Dominant soil textures within the HEC-HMS model area are summarized in Exhibit 3.

Soil parameters were determined based on weighted soil texture within each MUKEY using the parameters in Table 1. The capillary suction (PSIF), initial soil moisture, saturated soil moisture content, and hydraulic conductivity (Ksat) are based on published values in Rawls and Brakensiek (1983). Effective porosity from Rawls and Brakensiek (1983) is used to estimate the saturated moisture content. The initial moisture content was set to the more conservative starting moisture condition, represented by the field capacity (Rawls and Brakensiek, 1983; and GSSHA Wiki, accessed 2018).

Table 1. Soil Texture and Green and Ampt Parameters

Soil Surface Texture	Initial Content	Saturated Content	PSIF (in)	Ksat (in/hr)
Sand	0.07	0.42	1.95	9.27
Sandy Loam	0.17	0.41	4.33	0.86
Loam	0.22	0.42	3.75	0.62
Sandy Clay Loam	0.19	0.33	8.6	0.12
Clay Loam	0.24	0.39	8.22	0.08
Course Sandy Loam	0.17	0.41	4.33	0.86

The Green and Ampt parameters were estimated in ArcGIS using the SSURGO soils data and parameters summarized in Table 1 and Appendix B.

2.2.3 Impervious Area

Impervious area reduces infiltration capacity. In this case, offsite impervious areas are associated with agriculture and undeveloped areas, which have zero percent impervious. Percent impervious for each land use are summarized in Table 2.

2.3 SCS Unit Hydrograph – Hydrograph Transformation

The transformation method used in the HEC-HMS model was SCS Unit Hydrograph Method which requires lag time calculations. Lag time is 60% of time of concentration. Time of concentration was calculated using USDA's TR-55 for sheet flow (or overland flow), shallow concentrated, and channel flow. Lag Time calculations can be found in Appendix B.

2.4 Muskingum-Cunge – Channel Routing

HEC-HMS provides the option of using one of several different hydrograph routing methods. Given the predominantly natural terrain and limited offsite land uses, the Muskingum-Cunge eight-point routing method was selected. The Muskingum-Cunge routing method is a combination of the conservation of mass and the iterative diffusion of the conservation of momentum at every time step within the channel (USACE, 2009). HEC-HMS requires the following routing parameters for each Muskingum-Cunge routing reach:

- Channel length,
- Channel slope,
- Manning's *n* roughness coefficient for the channel and overbank areas,
- Eight-point cross-section of natural channel flow areas, and
- Trapezoidal cross-section to represent proposed ditches.

ArcGIS was utilized to determine average reach cross-sections, channel lengths, and routing reach slopes for each of the offsite routing reaches (offsite reaches are labeled on Exhibit 2). Manning's *n* values are selected to reflect average conditions throughout the entire routing reach.

The Manning's *n* values selected for the routing reaches in the HEC-HMS model range from 0.05 to 0.06 for natural channels, 0.08 for overbank areas, and 0.02 for ditches.

2.4.1 Rainfall Data

The rainfall depths used for the modeled hypothetical design storms in HEC-HMS were developed using data from the NOAA Atlas 14, Volume 1, Version 5, revised 2011 (Bonin et al., 2011).

ArcGIS was used to calculate the area weighted average NOAA Atlas 14 precipitation depth for the 5, 10, 15, 30, 60-minute and 2, 3, 6, 12, and 24-hour rainfall depths for the 10-, 25-, and 100-year storm events (Appendix B). The data were used to develop custom, balanced design storm hyetographs for each subbasin in HEC-HMS.

3 STUDY AREA HYDROLOGIC AND HYDRAULIC MODELING

Hydrologic and hydraulic modelling for the study area and a small portion of the offsite area (Exhibit 2) was conducted using PCSWMM version 7.2 using Rational Method Calculations, Synthetic Unit Hydrographs, and storm system layout. PCSWMM utilizes the dynamic wave method, which solves the one-dimensional Saint-Venant equations for continuity and momentum for model conduits. This method enables the simulation of pressurized flow in closed conduits. Dynamic wave routing accounts for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow.

3.1 PCSWMM Model Layout

The existing PCSWMM model layout within the City was determined using existing storm drain system horizontal data obtained from the City of Gonzales Conceptual Drainage Plan prepared by Rick Engineering Company in February of 2010, as-built data provided to Kimley-Horn and Associates and the City of Gonzales, and AMBAG LiDAR, and aerial imagery.

The proposed model layout of the backbone system for Vista Lucia, D'Arrigo Brothers, and Puente del Monte was designed based on proposed grading and land use provided by Kimley-Horn and Associates. Although the City's design standards suggest a 20-year design storm for main trunklines, since this is a planning level analysis, a design storm of 25-years was used for conservativeness.

3.2 Peak Flow Calculations

Onsite flows were calculated using the Rational Method per City of Gonzales Design Standards, dated May 1995. Intensity was determined from NOAA Atlas 14 Precipitation (Appendix B). Duration was estimated using time of concentration. Time of concentration was calculated using the methods outlined USDA's Urban Hydrology for Small Watersheds, TR-55. Subbasins were delineated with GIS software using a combination of AMBAG LiDAR data and proposed grading provided by Kimley-Horn and Associates. The minimum time of concentration was assumed to be 15 minutes. Subbasin areas, time of concentrations, weighted runoff coefficients, intensities, and peak flows for both existing and proposed conditions and for the 10-, 25- and 100-year storm events are provided in Appendices C and D, respectively. Runoff coefficients and percent impervious for each land use type are summarized in Table 2 and on Exhibit 4.

Table 2. Runoff Coefficients and Percent Impervious for each Land Use Type

Land Use	% Impervious	Runoff Coefficient
Parks	0	0.2
Promenades and Greenbelts	0	0.2
Drainage, Retention, Buffers, Other Open Space	0	0.2
Agriculture	0	0.2
Water	0	0.2
High Density Residential	90	0.7
Medium High Density Residential	75	0.55
Medium Density Residential	65	0.45
Low Density Residential	60	0.4
Out Parcel (Low Density Residential)	60	0.4
School	33	0.4
Road	95	0.95
Light Industrial	72	0.7
Community Commercial Mixed Use	95	0.95
Retail Commercial Center	95	0.95
Urban Reserve	0	0.2

3.3 Synthetic Universal Rational Method Hydrograph

Runoff hydrographs for the 10-, 25- and 100-year storm events were estimated for each onsite subbasin using a synthetic Universal Rational Method Hydrograph. The ordinates of the Universal Rational Method Hydrograph are summarized in Table 3. The peak flows, Q, for both existing and proposed conditions, along with inflow node locations, were obtained from Appendices C and D, respectively. Inflow node locations were determined based on the onsite subbasin delineations.

Table 3. Synthetic Universal Rational Method Hydrograph Ordinates [Tc = time of concentration (min), Qp = Peak Flow (cfs)]

Time (min) Texture	Q (cfs)
0	0
Tc	0.21Qp
2Tc	0.30Qp
3Tc	Qp
4Tc	0.54Qp
5Tc	0.39Qp
6Tc	0.25Qp
7Tc	0.18Qp
8Tc	0.15Qp
9Tc	0.14Qp
10Tc	0.13Qp
11Tc	0

Source:http://files.carlsonsw.com/mirror/manuals/Carlson_2014/source/Hydrology/Watershed/Watershed_Hydrograph_Rational_Method/Rational_Method.html

3.4 Manning's *n* Channel Flow Roughness Coefficient

All natural channels were given a Manning's n values between 0.02 and 0.05 and all reinforced concrete pipes and concrete channels were given a value of 0.013. Although the City's design standards require a value of 0.015 for reinforced concrete pipes (RCPs) with diameters of 2-feet or less, since a design storm of 25-years was used for conservativeness and this is a planning level analysis, a value of 0.013 was used for all RCPs. City's design standards should be used during the design phase.

4 RESULTS

4.1 HEC-HMS Results

The offsite peak flows for the 24-hour 10-, 25-, and 100-year storm events are summarized in Table 4. HEC-HMS input parameters are summarized in Appendix B. These flows were validated using the USGS Regional Regression Equations for California (Gotvald et. al., 2012). Gonzales is located in Hydrologic Region #4. As seen in Chart 1, the offsite peak flows fall within the accepted range of values from the regional regression results.

Table 4. Summary of offsite peak flows

Location	Area (mi2)	Q (cfs)		
		10-Year	25-Year	100-Year
J-1-1	0.68	21	42	79
J-7-1	1.30	1.2	39	91
J-1-2	1.37	29	74	142
J-1-4	2.33	38	98	190
J-5-3	3.35	117	224	434
J-6-4	5.98	365	654	1250

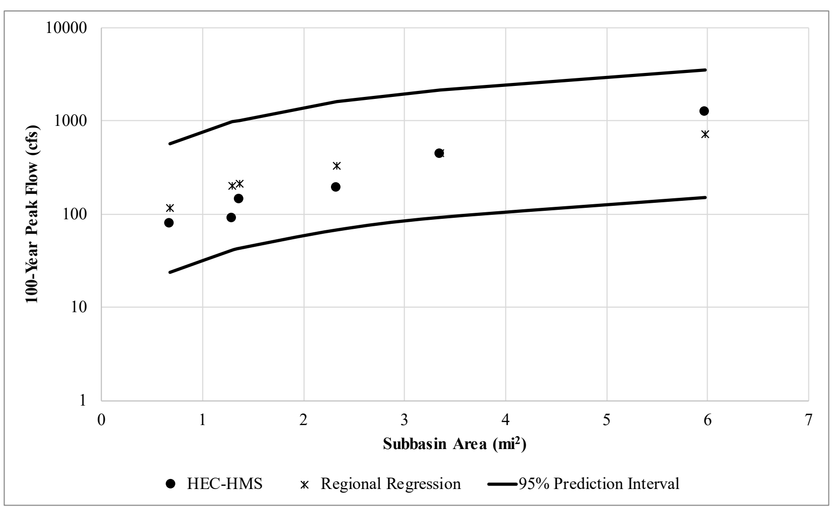

Chart 1. Offsite peak flows compared to regional regression results for Hydrologic Region #4.

4.2 PCSWMM Results

Existing and proposed conditions models were developed for the study area, a small portion of the offsite area, and where the study area connects to the City's existing storm drain system. The existing conditions model layout is summarized on Exhibits 6, 8, 10, and 12. The conceptual proposed backbone systems for Vista Lucia, D'Arrigo Brothers, and Puente del Monte were developed based on the proposed land uses (Exhibit 5) and are summarized in Exhibits 7, 9, 11, and 13. PCSWMM input parameters, system details, and output results are summarized in Appendices C and D. As discussed above, flows were determined using the Rational Method and flow hydrographs were determined using the Universal Rational Method hydrograph.

The proposed conceptual drainage system within each of the development areas has been designed based on the 25-year storm event. The system has been designed mostly using Reinforced Concrete Pipes, with some box culverts where slopes flatten out, and compact dirt trapezoidal channels, except between proposed nodes J807 and J811, where a concrete channel is recommended (Exhibit 9). Within Vista Lucia, Reinforced Concrete Pipe (RCP) sizes range between 2- and 4.5-foot in diameter. Within D'Arrigo Brothers, pipe sizes range between 2.5- and 3-feet in diameter. Within Puente del Monte, pipe sizes range between 2- and 3.5-feet in diameter. See Exhibits 9, 11, and 13 and Appendix D for system details.

The system has been designed, where possible, to route offsite flows around the proposed developments. Ultimately, as with existing conditions, all flows are routed into Gonzales Slough. Storage basins have been incorporated into the proposed drainage network to mitigate post-project peak flows to pre-project levels.

The location, sizing and final configuration of the culverts shown are conceptual only. The final sizing, configuration and locations of the culverts will be determined during the final design of individual projects.

4.2.1 Storage Basin Design

The City requires new developments to mitigate increases in runoff for the 100-year design storm event. Storage basins within Vista Lucia, D'Arrigo Brothers, and Puente del Monte have been sized to meet these requirements. Storage basin locations have been chosen based on topography and, where possible, within land use areas designated for drainage.

Due to topography and proposed land uses, it was not possible to place all basins within the proposed drainage areas as shown on Exhibit 5. However, basin sizes presented here have been designed to maximize those in areas set aside for drainage and minimize the size of basins outside of these areas. The proposed basin locations and overall footprints are provided on Exhibits 9, 11, and 13. All basins have been sized to include 1-feet freeboard, 3:1 side slopes, and 30-feet buffer for maintenance roads and fencing. A summary of the required surface area, footprints, and storage volumes are provided in Table 5, details are provided in Appendix E. A discussion of how the storage basins were sized is provided below.

The SSURGO provides soil column information for each MUKEY. Soil at depths of over about 4.5-feet in the study area is comprised of gravelly sandy loams which have high infiltration rates. In sizing the basins, the infiltration parameters for sand (Table 1) were used with a factor of safety of four (4) applied to the Ksat value. This should be conservative; however, in-situ soil testing is recommended to validate infiltration rates.

LIDs (e.g., bioswales, infiltration trenches, etc.) will be designed on a site by site basis and incorporated into the project during the design phase. The location, sizing and final configuration of all storm water basins shown are representational only. The final sizing, configuration and locations of storage basins will be determined with the final design of individual projects.

Table 5. Summary of Storage Basin Areas and Volumes

Basin ID	Side Slope	Buffer (ft)	Freeboard (ft)	Length (ft)	Width (ft)	Depth (ft)	Surface Area (AC)	Volume (AC-FT)
VL-1	3:1	30	1	341	341	6	2.7	11.8
VL-2	3:1	30	1	290	290	7	1.9	8.1
VL-3	3:1	30	1	290	290	5.5	1.9	7.5
VL-4	3:1	30	1	216	216	6	1.1	3.9
						Total	**7.6**	**31.1**
DAB-1	3:1	30	1	285	285	4.5	1.9	6.0
DAB-2	3:1	30	1	305	305	7	2.1	10.4
DAB-3	3:1	30	1	196	196	6	0.9	3.0
						Total	**4.9**	**19.5**
PDM-1	3:1	30	1	167	167	5	0.6	1.7
PDM-2	3:1	30	1	235	235	5	1.3	4.2
PDM-3	3:1	30	1	440	260	6	2.6	11.5
PDM-4	3:1	30	1	215	215	6	1.1	2.8
PDM-5	3:1	30	1	225	225	5	1.2	3.7
						Total	**6.8**	**23.9**

4.2.1.1 Vista Lucia Storage Basins

Vista Lucia was split into four overall drainage areas based on topography and the proposed road network. Drainage flow direction tends towards the west and southwest. The areas that drain to VL-1 and VL-2 (Exhibit 9) ultimately discharge to proposed node J1004 (Exhibit 9) located at the northwestern corner of the Urban Reserve area. Node J1004 corresponds to existing node J701B (Exhibit 8). The existing flows into J701B and the flows from the proposed drainage areas into the storage basins for the 10-, 25-, and 100-year events are summarized in Table 6.

To mitigate the post-project flows to pre-project conditions, VL-1 has been sized to retain up to and including the 100-year event. VL-2 has been sized to detain flows such that the peak flow at J1004 does not exceed the pre-project peak flow at J701B. VL-2 has a combination pipe (5-ft RCP) and weir outlet that meter out flows to at or below pre-project levels.

Table 6. Vista Lucia Storage Basin VL-1 and VL-2 Peak Flows

Location	Proposed Inflow (cfs)			Proposed Flow at J1004 (cfs)		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
VL-1	111	135	174	35	46	60
VL-2*	129	158	207			
Existing Flow at J701B*				39	47	61

*Excludes offsite flows. Comparison with offsite flows provided in Table 14.

The areas that drain to VL-3 and VL-4 ultimately discharge to proposed node J811 (Exhibit 9) which corresponds to existing node J484 (Exhibit 8). However, there is a portion of D'Arrigo Brothers that also contributes runoff to J483 that needs to be accounted for. The existing peak flows attributed to this portion of the Vista Lucia area are the difference between the flows at existing nodes J482 and J483 (Exhibit 8), which are 4.5-, 5.5-, and 7.2-cfs for the 10-, 25-, and 100-year events, respectively. The flows from the proposed drainage areas into the storage basins are summarized in Table 7. To mitigate the post-project flows to pre-project conditions, VL-3 has been sized to retain up to and including the 100-year event. VL-4 has been sized to detain flows such that the peak flow at J811 does not exceed the pre-project peak flow between J482 and J483. VL-4 has a pipe outlet (1-ft RCP) that meters out flows to at or below pre-project levels. Appendix E

Table 7. Vista Lucia Storage Basin VL-3 and VL-4 Peak Flows

Location	Proposed Inflow (cfs)			Proposed Flow at J811* (cfs)		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
VL-3	69	84	103	25	29	36
VL-4	45	55	71			
Existing Flow at J483*				26	32	41

*Excludes offsite flows. Comparison with offsite flows provided in Table 14.

4.2.1.2 D'Arrigo Brothers Storage Basins

D'Arrigo Brothers was split into three overall drainage areas based on topography and the proposed road network. The majority of D'Arrigo Brothers drain towards the west and the remaining drains towards the southwest. The areas that drain to DAB-1 ultimately discharge to proposed node J807 (Exhibit 13) which corresponds to existing node J482 (Exhibit 12). The existing flows into J482 and from the proposed drainage areas into storage basin DAB-1 for the 10-, 25-, and 100-year events are summarized in Table 8. To mitigate the post-project flows to pre-project conditions, DAB-1 has been sized to detain flows such that the peak flows at J807 do not exceed the pre-project peak flows at J482. DAB-1 has a pipe outlet (1.25-ft RCP) that meters out flows to at or below pre-project levels.

Table 8. D'Arrigo Brothers Storage Basin DAB-1 Peak Flows

Location	Proposed Inflow (cfs)			Proposed Flow at J807* (cfs)		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
DAB-1	44	54	70	21	25	31
			Existing Flow at J482*	21	26	34

*Excludes offsite flows. Comparison with offsite flows provided in Table 14.

The areas that drain to DAB-2 ultimately discharge to proposed node J811 (Exhibit 13) which corresponds to existing node J743 (Exhibit 12). The existing flows into J743 and the flows from the proposed drainage areas into basin DAB-2 for the 10-, 25-, and 100-year events are summarized in Table 9.   To mitigate the post-project flows to pre-project conditions, DAB-2 has been sized to detain flows such that the peak flows at JDAB-2 do not exceed the pre-project peak flows at J743. DAB-2 has a pipe outlet (2.75-ft RCP) that meters out flows to at or below pre-project levels.

Table 9. D'Arrigo Brothers Storage Basin DAB-2 Peak Flows

Location	Proposed Inflow (cfs)			Proposed flow at JDAB-2 (cfs)		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
DAB-2	98	119	153	41	49	60
			Existing Flow at J743	41	51	61

The areas that drain to DBA-3 ultimately discharge to node J725B-2 (Exhibits 10 and 11), the same node for both existing and proposed conditions.   However, there is a portion of Puente del Monte that also contributes runoff to J725B-2 that needs to be accounted for. Peak flows attributed to D'Arrigo Brothers and Puente del Monte were estimated based on overall contributing area (Table 10). DBA-3 has been sized to detain flows such that the peak flow out of DAB-3 does not exceed the pre-project flows attributed to the area draining to DAB-3. DAB-3 has a combination pipe (1.25-ft RCP) and weir outlet that meter out flows to at or below pre-project levels (Table 11).

Table 10. Area weighted flows for portion of D'Arrigo Brothers and Puente del Monte that drain to existing node J725B-2

Location	Area (AC)	Peak Flow (cfs)		
		10-Year	25-Year	100-Year
Existing Flow at J725B-2	550	45	55	72
Flow Attributed to D'Arrigo Brothers	85	7	8	11
Flow Attributed to Puente del Monte	245	20	25	32

Table 11. D'Arrigo Brothers Storage Basin DAB-3 Peak Flows

Location	Proposed Inflow (cfs)			Proposed Outflow from DAB-3 (cfs)		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
DAB-3	38	47	61	5	6	7
Existing Conditions Flow (from Table 10)				7	8	11

4.2.1.3 Puente del Monte Storage Basins

Puente del Monte was split into five overall drainage areas based on topography and the proposed road network. Puente del Monte drainage flow direction tends towards the west and the remaining drains towards the southwest. The majority of the flows converge at the southern-most corner of the site. Storage basins PDM-1, PDM-2, and PDM-3 have been located within land use areas designated for drainage. These storage basins have been sized to retain all flows up to and including the 100-year event so that PDM-4 and PDM-5 could be minimized. Note that over-detaining offsite flows could mitigate for the increase in flows to PDM-4 and PDM-5, thereby either eliminating or reducing the size of PDM-4 and PDM-5. However, this may necessitate the treatment of offsite flows to meet stormwater standards and the newly developed areas draining to PDM-4 and PDM-5 would still require stormwater treatment.

The area that drains to PDM-1 discharges to proposed node J903 (Exhibit 11) which ultimately drains to existing node J792 (Exhibit 10). To mitigate the post-project flows to pre-project conditions, PDM-1 has been sized to retain flows up to and including the 100-year event. The peak outflows from PDM-1 are zero, up to and including the 100-year event.

To mitigate post-project flows to pre-project conditions, PDM-2 has been sized to retain up to and including the 100-year event. The peak outflows from PDM-2 are zero, up to and including the 100-year event.

To mitigate post-project flows to pre-project conditions, PDM-3 has been sized to retain up to and including the 100-year event. The peak outflows from PDM-3 are zero, up to and including the 100-year event.

The areas that drain to PDM-4 (Exhibit 11) ultimately discharge to proposed node J2004 (Exhibit 11). Node J2004 corresponds to existing node J725-2 (Exhibit 10). The existing flows into J725-2 and flows from the proposed drainage areas into PDM-4 for the 10-, 25-, and 100-year events are summarized in Table 12. To mitigate the post-project flows to pre-project conditions, PDM-4 has a combination pipe (2.0-ft) and weir outlet that meter out flows to at or below pre-project levels.

Table 12. Puente del Monte Basin PDM-4 Peak Flows

Location	Proposed Inflow (cfs)	Proposed Flow at J2004* (cfs)

	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
PDM-4	65	79	89	20	27	31
Existing Flow at J725-2*				25	30	39

*Excludes offsite flows. Comparison with offsite flows provided in Table 14.

The areas that drain to PDM-5 ultimately discharge to node J725B-2 (Exhibits 10 and 11), the same node for both existing and proposed conditions. However, there is a portion of D'Arrigo Brothers that also contributes runoff to J725B-2 that needs to be accounted for. Peak flows attributed to D'Arrigo Brothers and Puente del Monte were estimated based on overall contributing area (Table 10). PDM-5 has been sized to detain flows such that the peak flow out of PDM-5 does not exceed the pre-project flows attributed to the area draining to PDM-5. PDM-5 has a combination pipe (2-ft) and weir outlet that meter out flows to at or below pre-project levels (Table 13).

Table 13. Puente del Monte Basin PDM-5 Peak Flows

Location	Proposed Inflow (cfs)			Proposed Outflow from PDM-5 (cfs)		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
PDM-5	61	74	96	20	21	32
Existing Conditions Flow (from Table 10)				20	25	32

4.2.2 Impacts on Existing Infrastructure

The City requires development projects to provide storage basins to mitigate increases in runoff for the 100-year design storm. Planning level basins have been sized to meet this requirement. No increases in flows are proposed up to and including the 100-year design storm. The entire backbone system has been sized to pass the 25-year design storm, which is more conservative compared to the City's design standard which uses a 20-year design storm. Proposed on- and off-site infrastructure are shown on Exhibits 7, 9, 11, and 13. Table 14 summarizes the 25-year peak flows with offsite flows included.

A comparison of the existing and proposed 25-year peak flows is provided in Table 14. The existing ditch below proposed Vista Lucia is currently under sized. Under proposed conditions it is recommended to make this a trapezoidal concrete ditch with a depth of 4-feet, bottom width of 6-feet, and side slopes of 2:1.

At Puente del Monte, existing offsite runoff flows northwesterly in an existing ditch northwest along Iverson Road and then southwesterly in an existing ditch labeled L-792A and L-792B on Exhibit 10 where it collect at J792 and is then routed under State Route 101 into Gonzales Slough. The remainder of offsite runoff flows southwesterly through the agricultural fields on the western side of Puente del Monte (see L-725-3 on Exhibit 12). Existing drainage ultimately flows to the southern end of the City where it collects within existing nodes J-792, J725B2, and

J725-2 and the existing ditch along Gloria Road and is then routed under State Route 101 into Gonzales Slough. Offsite flows will continue to be routed in this manner (Exhibit 11).

Table 14. 25-Year Existing and Proposed Peak Flow Summary

	10, 25, and 100-Year Peaks - no offsite flows		10, 25, and 100-Year Peaks - with offsite flows	
Location	Existing J701B	Proposed J1004	Existing J701B	Proposed J1004
Peak Flow (cfs)	39, 47, & 61	35, 43, & 48	332, 619, & 1255	350, 637, & 884
Time of Peak	06/13/2009 04:12	06/13/2009 01:13	06/13/2009 13:04	06/13/2009 13:00
Location	Existing J484	Proposed J811	Existing J484*	Proposed J811
Peak Flow (cfs)	41, 49, & 64	25, 29, & 36	120, 258, & 519	108, 211, & 261
Time of Peak	06/13/2009 04:31	06/13/2009 03:28	06/13/2009 13:53	06/13/2009 13:27
Location	Existing J482	Proposed J807	Existing J482*	Proposed J807
Peak Flow (cfs)	21, 26, 34	21, 25, & 31	117, 225, & 435	114, 220, & 428
Time of Peak	06/13/2009 02:42	06/13/2009 03:19	06/13/2009 13:17	06/13/2009 13:22
Location	Existing J743	Proposed JDAB-2	Existing J743	Proposed JDAB-2
Peak Flow (cfs)	41, 51, & 61	41, 49, & 60	41, 51, & 61	41, 49, & 60
Time of Peak	06/13/2009 04:18	06/13/2009 02:18	06/13/2009 04:18	06/13/2009 02:18
Location	Existing J792	Proposed OF-J792	Existing J792	Proposed OF-J792
Peak Flow (cfs)	28, 34, & 44	25, 30, & 39	61, 158, & 313	61, 157, & 312
Time of Peak	06/13/2009 03:40	06/13/2009 02:53	06/13/2009 13:12	06/13/2009 13:08
Location	Existing J725B-2	Proposed OF-J725B-2	Existing J725B-2	Proposed OF-J725B-2
Peak Flow (cfs)	45, 55, & 71	36, 48, & 58	45, 55, & 71	36, 48, & 58
Time of Peak	06/13/2009 04:30	06/13/2009 01:36	06/13/2009 04:30	06/13/2009 01:36
Location	Existing J725-2	Proposed J2004	Existing J725-2	Proposed J2004
Peak Flow (cfs)	25, 30, & 39	20, 27, & 31	25, 36, & 72	20, 35, & 72
Time of Peak	06/13/2009 04:36	06/13/2009 01:26	06/13/2009 04:36	06/13/2009 13:11

*Flows taken from the 25-year Existing Conditions, with offsite flows and proposed improvements, PCSWMM Model

4.2.3 Post Construction Stormwater Requirements

The Central Coast Regional Water Quality Control Board (CCRWQCB) Post-Construction Stormwater Management Requirements for Tier 4 projects (Tier 4 projects are projects that create or replace 22,500 square feet of impervious surface) require projects to treat a rainfall intensity of at least 0.2 in per hour, prevent offsite discharge from events up to the 95th percentile rainfall event, and mitigate post-development peak flows to at least pre-project peak flows for the 2- through 10-year storm events (aka hydromodification requirements).

Design guidance to meet these requirements is provided in the CCRWQCB Monterey County Stormwater Technical Guide for Low Impact Development dated March 2015 (LID Guide). The LID Guide provides guidance for the sizing and design of LID facilities that are distributed throughout a site. These facilities are sized and located on a site-by-site case during the design phases of a project. Conceptual level sizing, based on land use, is presented here.

Following the LID Guide, a 4% sizing factor (area = 0.04 times tributary impervious area) is used to estimate the minimum surface area required to meet the treatment requirement of 0.2 inches per hour (Table 15). The 95th percentile retention requirements for each land use type are determined as the impervious area multiplied by the runoff factor and storm depth. The 95th percentile rainfall event for the City is 1.1 inches as determined from the County's 2013 95th percentile rainfall depth map (Appendix F). The 95th percentile retention volumes, assuming pervious areas are landscaped areas, are summarized in Table 16.

The storage basins VL-1, VL-3, PDM-1, PDM-2, and PDM-3 have been sized to retain all flows up to and including the 100-year event and, therefore, meet the hydromodification requirements. The regional storage basins VL-2, VL-4, DAB-1, DAB-2, DAB-3, PDM-4, and PDM-5 are all located in areas that are ideal based on topography but within land use areas not designated for drainage. As such, these storage areas have been sized to minimize the required surface area and for the 10- through 100-year events. To meet the hydromodification requirements, either localized management could be incorporated as part of LID design, or these regional storage basins could be upsized and designed to limit outflow up to the 10-year event. Either can be incorporated during the design phase as drainage mitigation areas are identified.

Table 15. Surface area needed to meet the Stormwater Treatment Requirements for each Land Use Type

Land Use	Percent Impervious	Treatment Area per Acre (ft^2/Acre)
Low Density Residential	40	700
Medium to High Density Residential	70	1,220
School	50	870
Light Industrial	75	1,310
Commercial/Roads	95	1,655

Table 16. 95th Percentile Stormwater Retention Requirements for each Land Use Type

Land Use	Percent Impervious	Impervious Runoff Coefficient	Pervious Runoff Coefficient	Volume per Acre (ft^3/Acre)
Low Density Residential	40	1	0.1	735
Medium to High Density Residential	70	1	0.1	2,040
School	50	1	0.1	1,100
Light Industrial	75	1	0.1	2,320
Commercial/Roads	95	1	0.1	3,625

5 SUMMARY

The purpose of this report is to develop a conceptual backbone stormwater masterplan for Vista Lucia, D'Arrigo Brothers, and Puente del Monte, proposed developments within the City of Gonzales SOI, and determine impacts the new development will have on existing infrastructure. The study area encompasses these proposed developments along with a small portion of offsite (Exhibit 2). Existing and proposed land uses are summarized on Exhibits 4 and 5.

Offsite flows generated from the hills above the study area were estimated using HEC-HMS. Flows within the study area were estimated using PCSWMM. The City's design standards require drainage pipes to be sized for the 20-year design storm for commercial, industrial, and major trunklines. Since this is a planning level masterplan for the backbone system, the 25-year design storm was used for conservativeness. The conceptual drainage system within each of the development areas has been designed based on the 25-year storm event. The system has been designed, where possible, to route offsite flows around the proposed developments. Ultimately, as with existing conditions, all flows are routed into Gonzales Slough. The existing and proposed drainage systems are summarized on Exhibits 6 through 13.

Although the conceptual layout presented here is based on routing offsite flows around the proposed developments, a developer could, at the approval of the City, consider routing and detaining these flows onsite to mitigate increases in peak flows. This may allow for more flexibility in sizing and locating storage basins. However, any runoff generated from the proposed development would need to meet the post construction stormwater management requirements.

The City requires retention facilities to mitigate the increase in runoff associated with the 100-year design storm. Regional storage basins have been incorporated into the proposed drainage network to mitigate post-project peak flows to pre-project levels for the 10- through 100-year storm events.

Post construction stormwater management requires new developments replacing or creating greater than 22,500 square-feet of impervious surface to treat at least the 0.2 inches per hour intensity, retain the 95^{th} percentile rainfall event, and mitigate post-development peak flows to at least pre-project peak flows for the 2- through 10-year storm events.

The storage basins VL-1, VL-3, PDM-1, PDM-2, and PDM-3 have been sized to retain all flows up to and including the 100-year event and, therefore, meet the hydromodification requirements. The regional storage basins VL-2, VL-4, DAB-1, DAB-2, DAB-3, PDM-4, and PDM-5 are all located in areas that are ideal based on topography but within land use areas not designated for drainage. As such, these storage areas have been sized to minimize their surface area and for the 10- through 100-year events. To meet the hydromodification requirements, either localized management could be incorporated as part of LID design, or these regional storage basins could be upsized and designed to limit outflow up to the 10-year event. Either can be incorporated during the design phase as drainage mitigation areas are identified.

Although the conceptual layout presented here is based on routing offsite flows around the proposed developments, a developer could, at the approval of the City, consider routing and detaining these flows onsite to mitigate increases in peak flows. This may allow for more flexibility in sizing and locating storage basins. However, any runoff generated from the proposed development would still be required to meet the post construction stormwater management requirements.

New development within the FEMA Zone A areas will need to comply with the City's Floodplain Development Standards (Title 14 Flood Control, Chapter 14.04 Floodplain Management). A Floodplain Development Permit is required prior to construction.

The following improvements to the existing infrastructure are recommended:

- The existing ditch below the proposed Vista Lucia is currently under sized. It is recommended to make this a trapezoidal concrete ditch with a depth of 4-feet, bottom width of 6-feet, and side slopes of 2:1.

Note, since this is a planning level analysis, only conceptual pipe, inlet geometry, and storage basin design are provided here. The final sizing, configuration and locations of the basins, pipes, etc., will be determined during the design phases of each, individual project.

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Appendix A – Exhibits

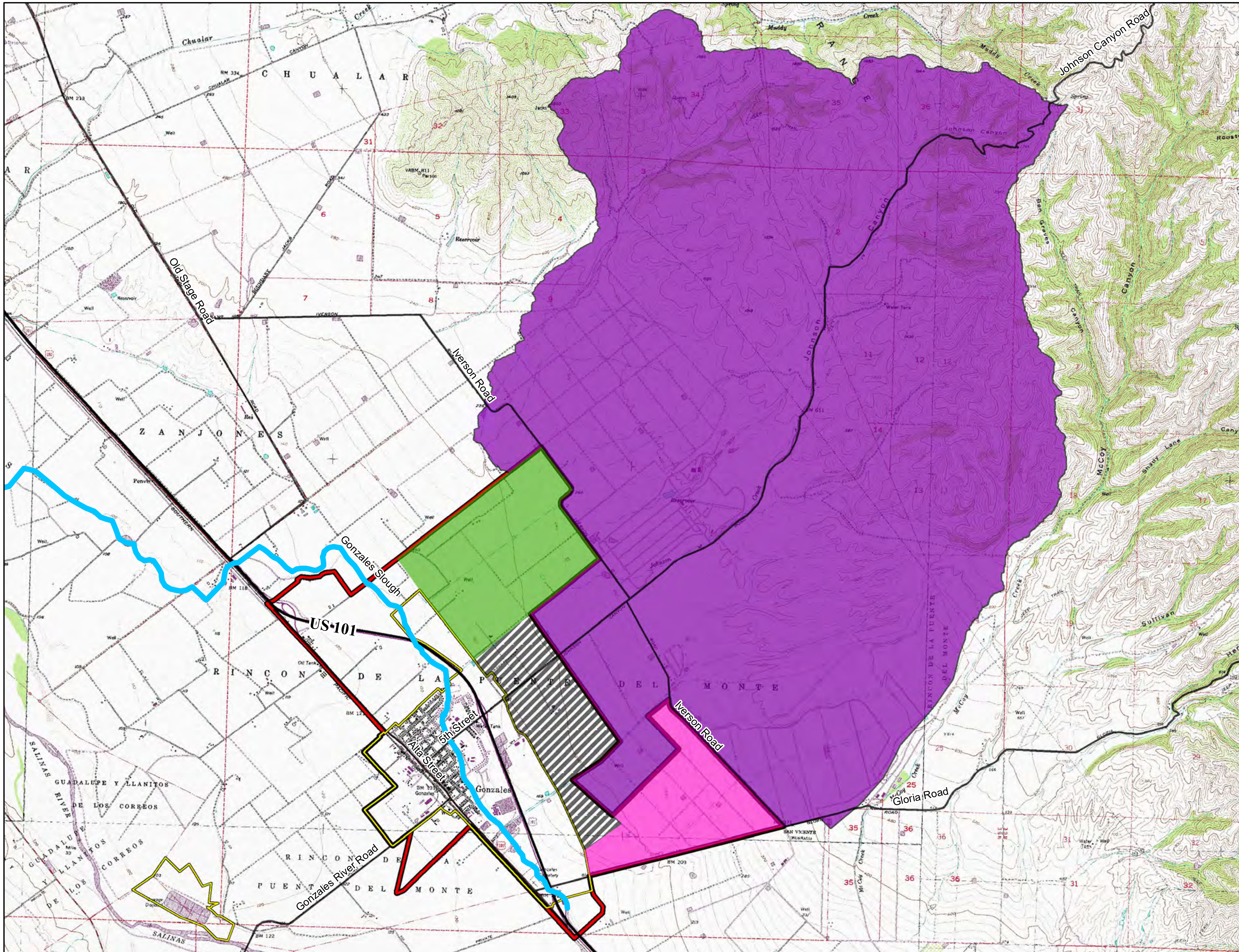

Exhibit 1
Location Map

Prepared By:
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Prepared On: 1/4/2019

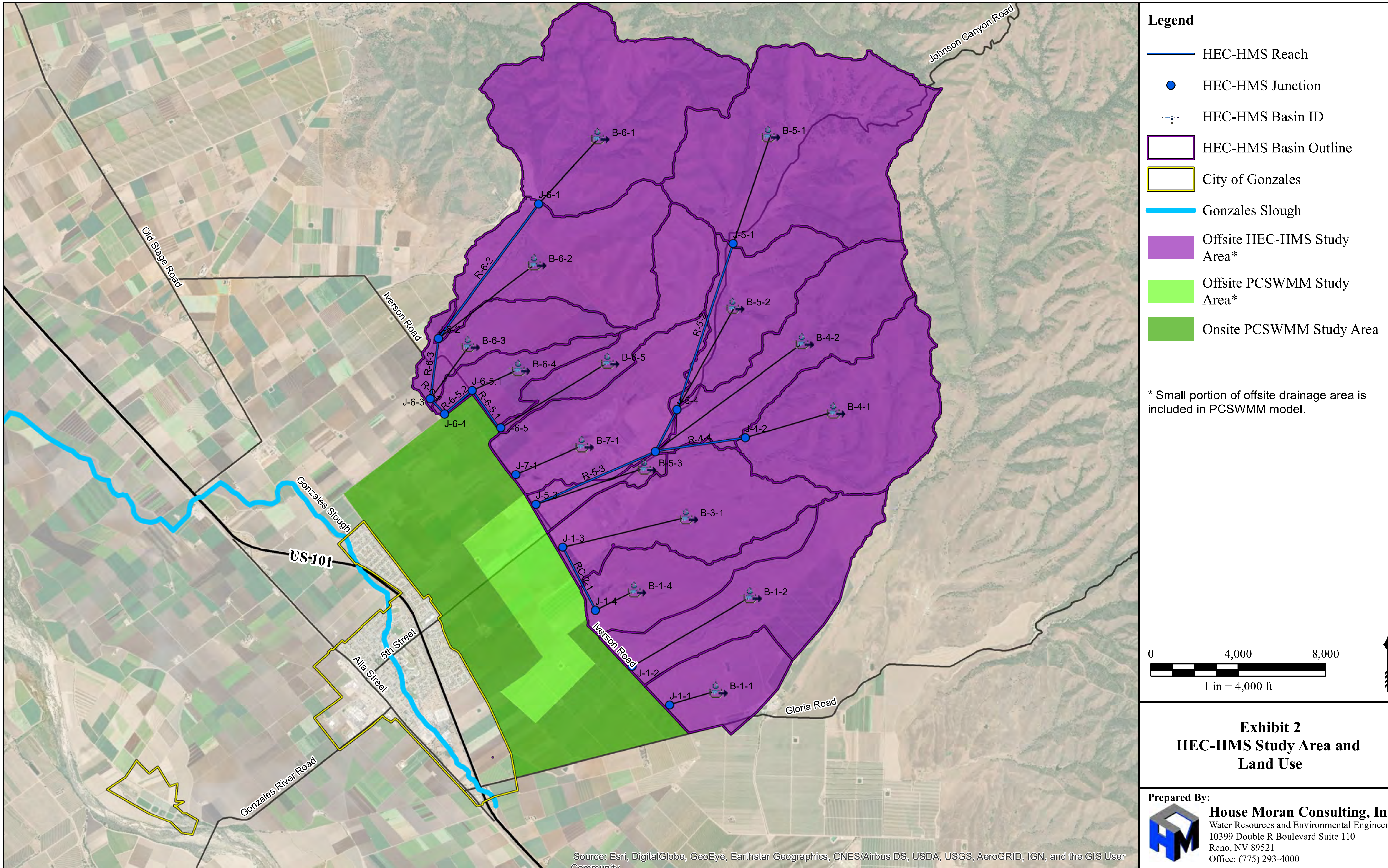

Exhibit 2
HEC-HMS Study Area and Land Use

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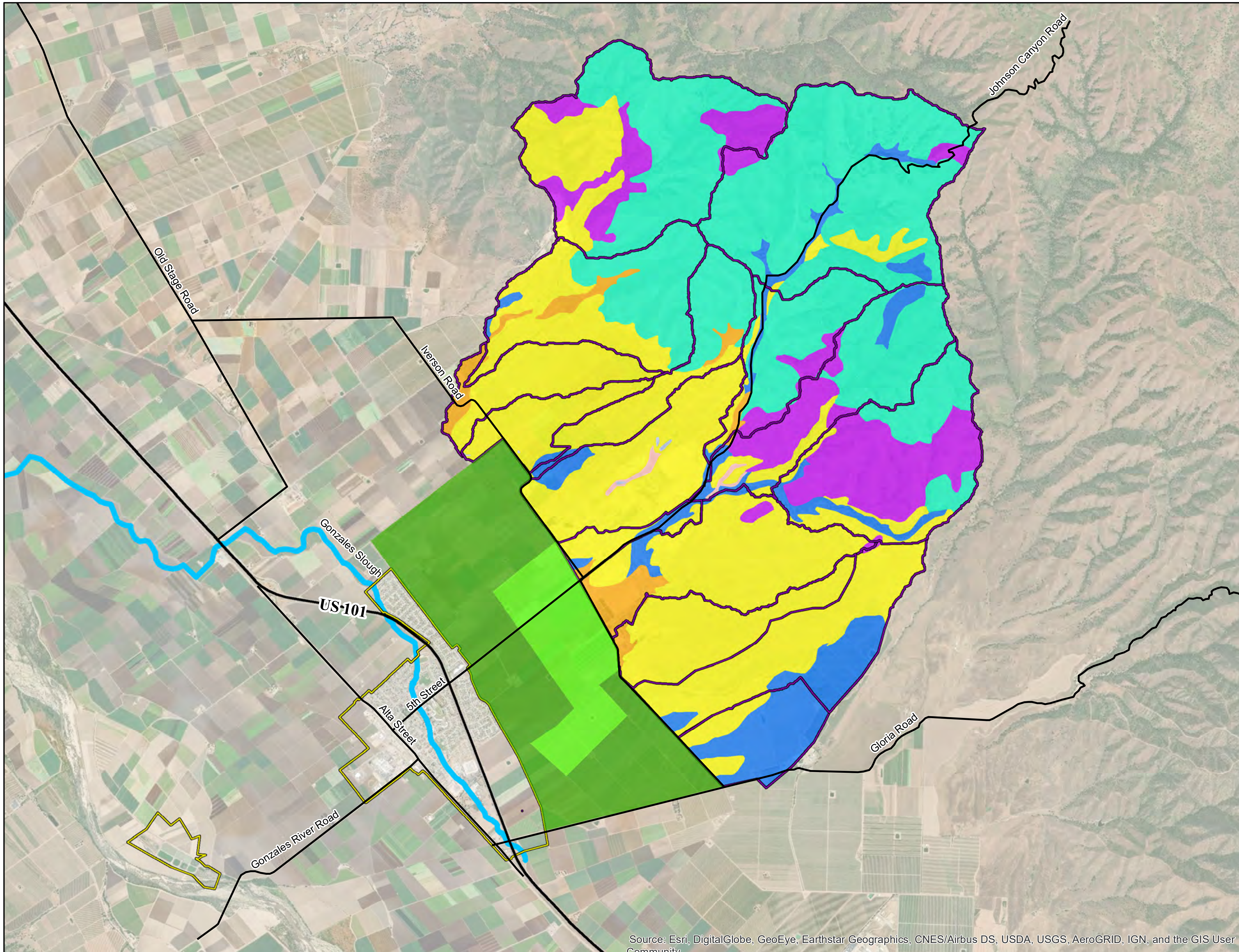

Legend

- Offsite PCSWMM Study Area
- Onsite PCSWMM Study Area
- City of Gonzales
- Gonzales Slough
- HEC-HMS Basin Outline

Dominant Soil Texture

- Sand
- Sandy clay loam
- Sandy loam
- Loam
- Coarse sandy loam
- Clay loam
- Water

0 4,000 8,000
1 in = 4,000 ft

Exhibit 3
HEC-HMS Soils Map

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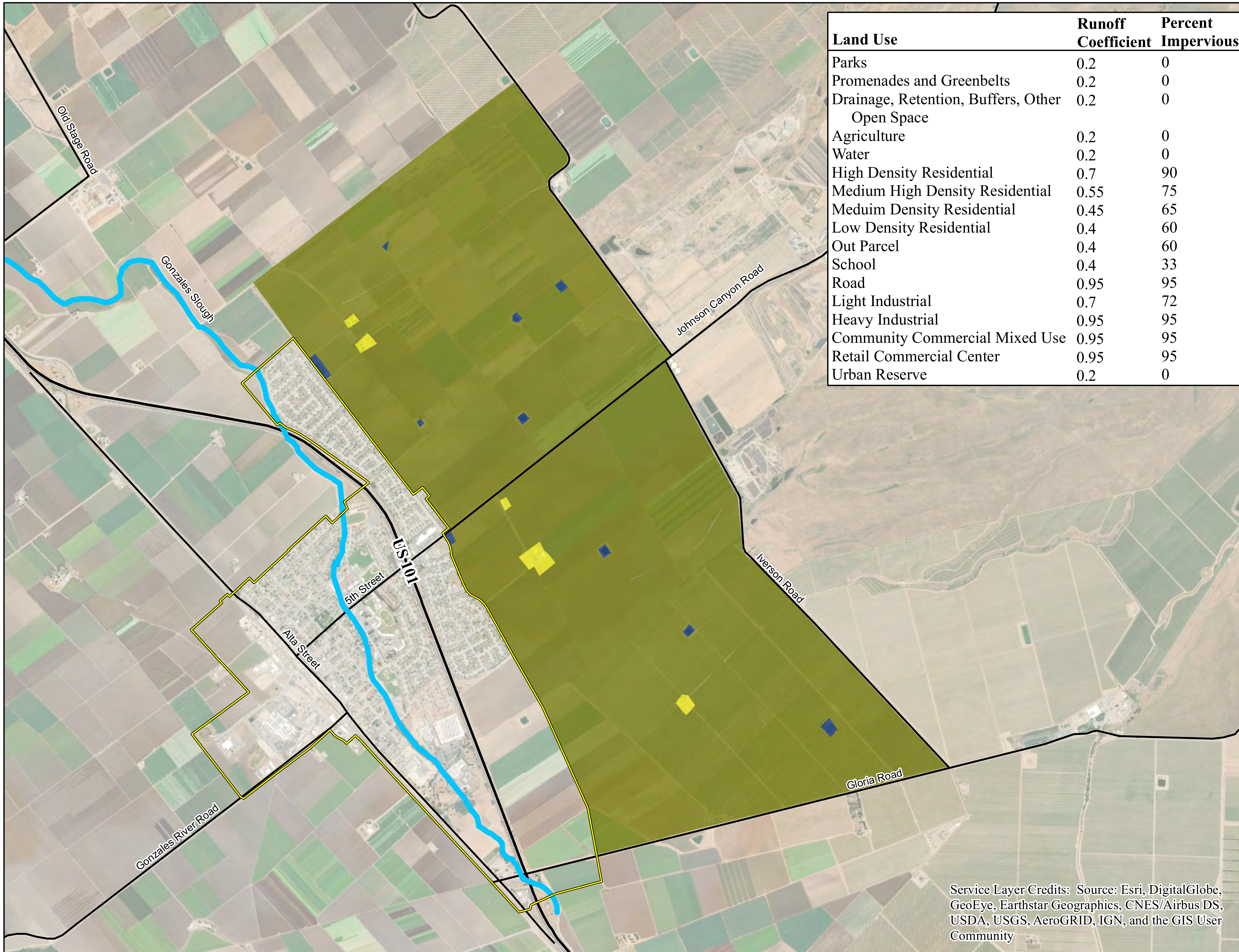
Prepared On: 1/13/2019



Land Use	Runoff Coefficient	Percent Impervious
Parks	0.2	0
Promenades and Greenbelts	0.2	0
Drainage, Retention, Buffers, Other Open Space	0.2	0
Agriculture	0.2	0
Water	0.2	0
High Density Residential	0.7	90
Medium High Density Residential	0.55	75
Meduim Density Residential	0.45	65
Low Density Residential	0.4	60
Out Parcel	0.4	60
School	0.4	33
Road	0.95	95
Light Industrial	0.7	72
Heavy Industrial	0.95	95
Community Commercial Mixed Use	0.95	95
Retail Commercial Center	0.95	95
Urban Reserve	0.2	0

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Legend

City of Gonzales

Gonzales Slough

Existing Land Use

Agriculture

Water

Low Density Residential

0 2,000 4,000

1 in = 2,000 ft

Exhibit 4
Existing Land Use Map
PCSWMM Model Area

Prepared By:

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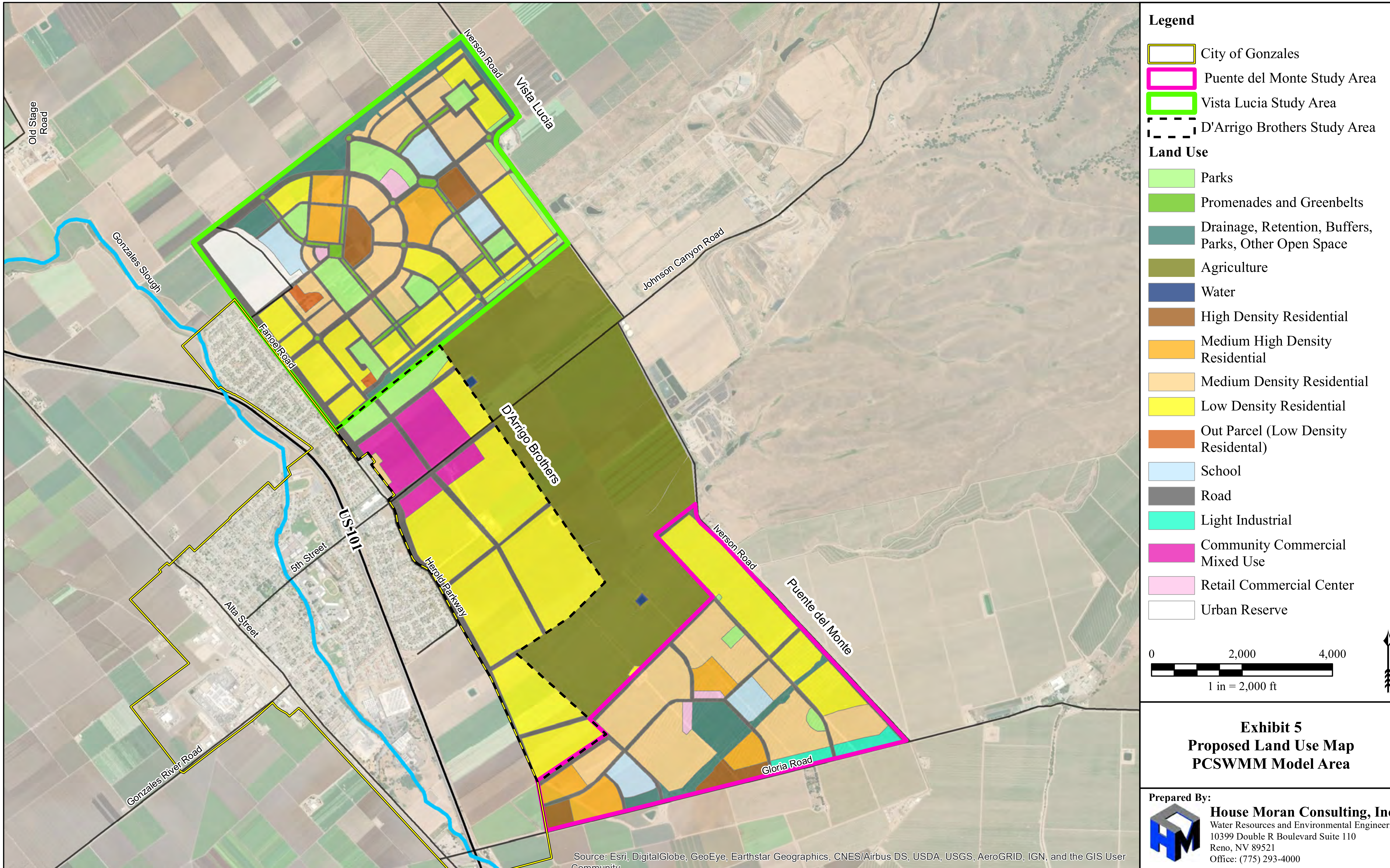
Prepared On: 1/7/2019



Note: All land use outside proposed developments remain the same between existing and proposed conditions.

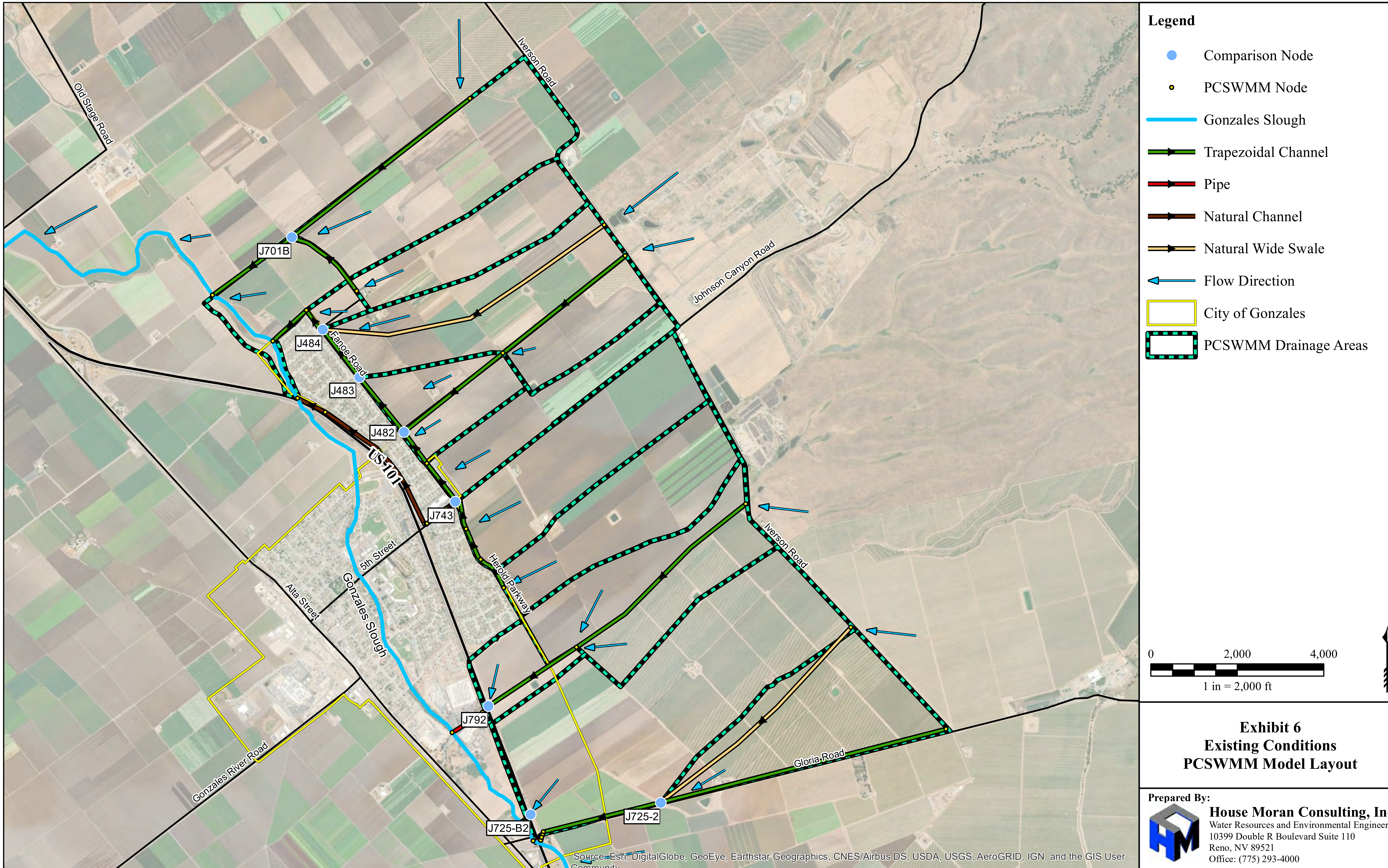


Exhibit 6
Existing Conditions
PCSWMM Model Layout

Prepared By:
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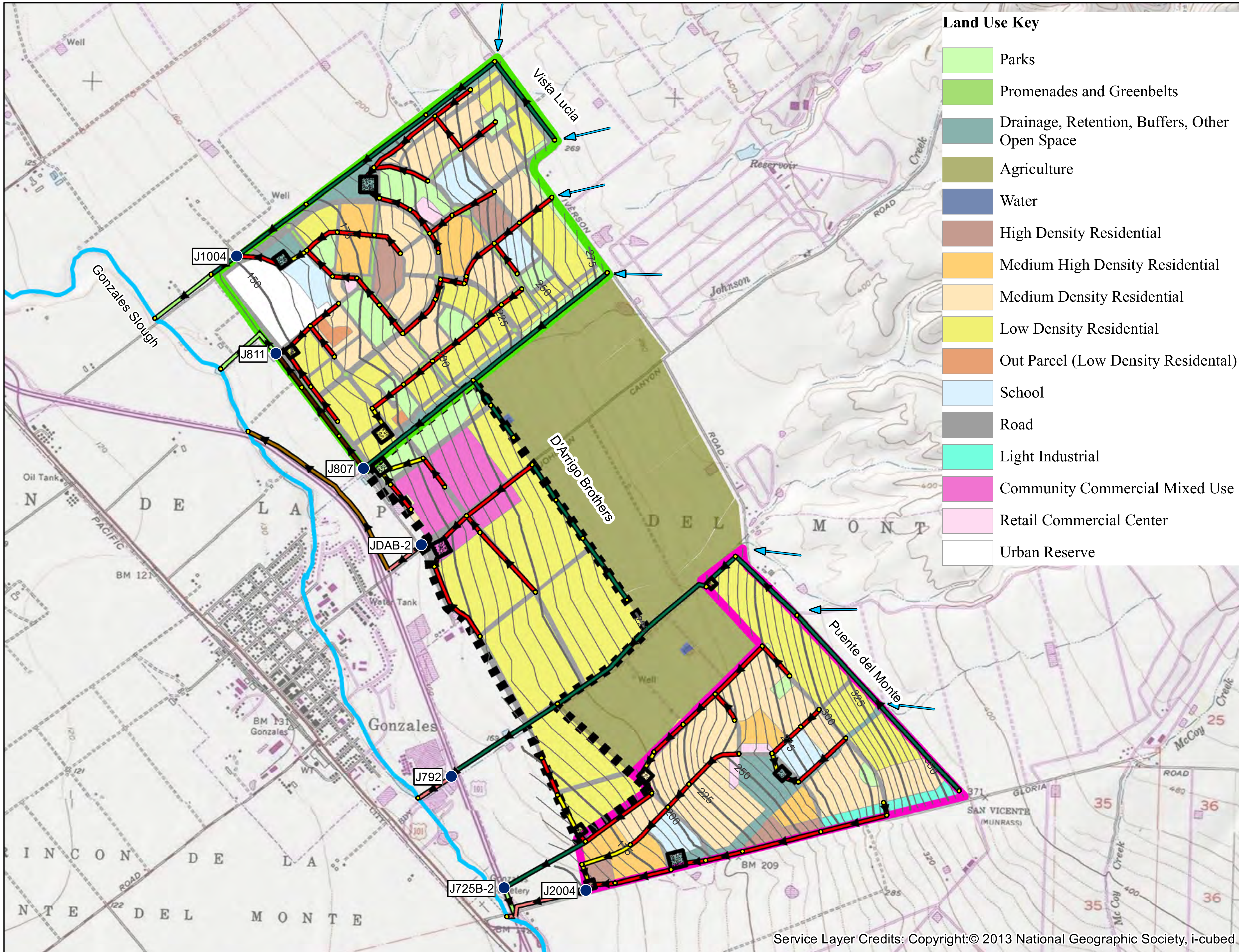
Prepared On: 1/13/2019



Legend

- Comparison Node
- PCSWMM Node
- Proposed Storage Basin
- Existing Pipe
- Existing Natural Channel
- Existing Trapezoidal Channel
- Proposed Circular Pipe - Diameter Labeled
- Proposed Rectangular Pipe - Length and Width Labeled
- Proposed Concrete Channel - Depth; Bottom Width Labeled
- Proposed Compacted Dirt Channel - Depth; Bottom Width Labeled
- Gonzales Slough
- Puente del Monte Study Area
- D'Arrigo Brothers Study Area
- Vista Lucia Study Area
- Proposed Contours - 25'
- Proposed Contours - 5'
- Offsite Flow

0 1,000 2,000

1 in = 4,000 ft

Exhibit 7
Proposed Conditions
PCSWMM Model Layout

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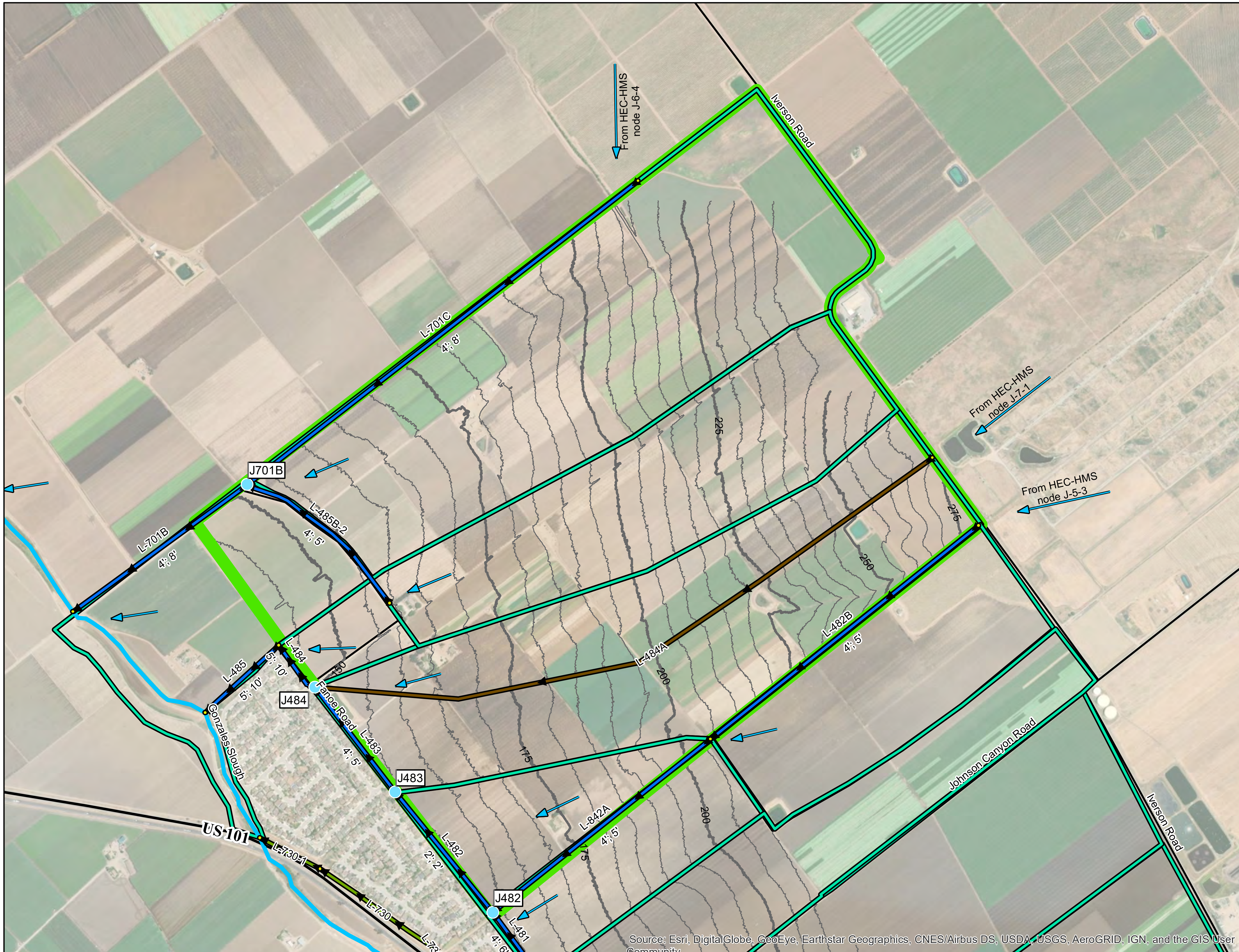
Prepared On: 1/6/2019



Legend

- Comparison Nodes
- FlowDirection_Onsite
- Trapezoidal Channel - Depth; Bottom Width Labeled
- Pipe - Diameter Labeled
- Natural Channel
- Natural Wide Swale
- Gonzales Slough
- PCSWMM Drainage Areas
- Contours - 25 ft
- Contours - 5 ft

0 1,000 2,000

1 in = 2,000 ft

Exhibit 8
Vista Lucia Study Area
Existing Conditions
PCSWMM Model Layout

Prepared By:

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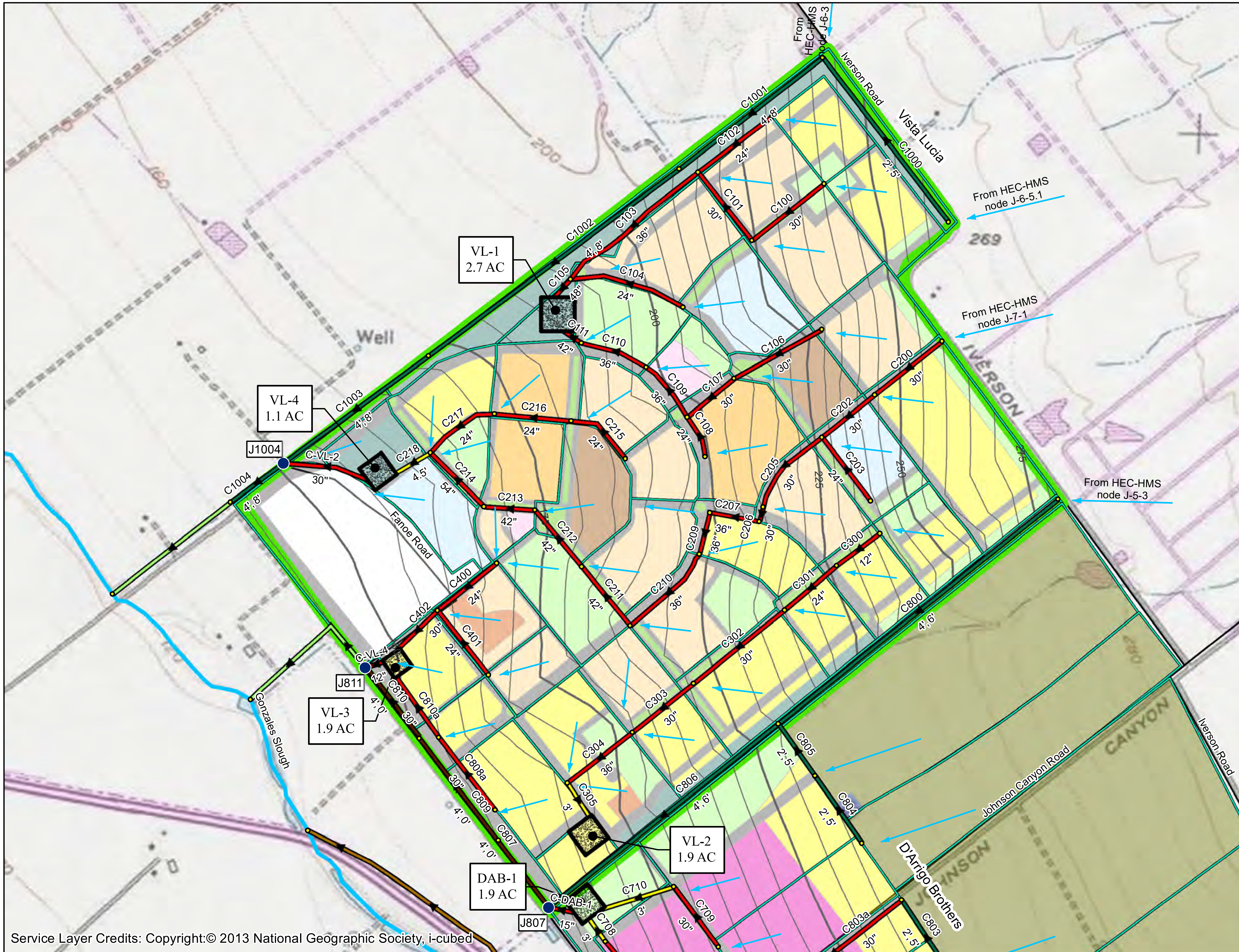
Prepared On: 1/4/2019



Legend

- Comparison Node
- Flow Direction
- Proposed Circular Pipe - Diameter Labeled
- Proposed Rectangular Pipe - Length and Width Labeled
- Proposed Concrete Channel - Depth; Bottom Width Labeled
- Proposed Compacted Dirt Channel - Depth; Bottom Width Labeled
- Existing Pipe
- Existing Natural Channel
- Existing Trapezoidal Channel
- PCSWMM Drainage Areas
- Proposed Storage Basin
- Proposed Contours - 25'
- Proposed Contours - 5'
- Gonzales Slough

Note 1: All proposed trapezoidal channels have a 2:1 side slope.
Note 2: See Exhibit 5 for Land Use Key.

0 1,000 2,000
1 in = 4,000 ft

Exhibit 9
Vista Lucia Study Area
Conceptual Drainage Layout

Prepared By:
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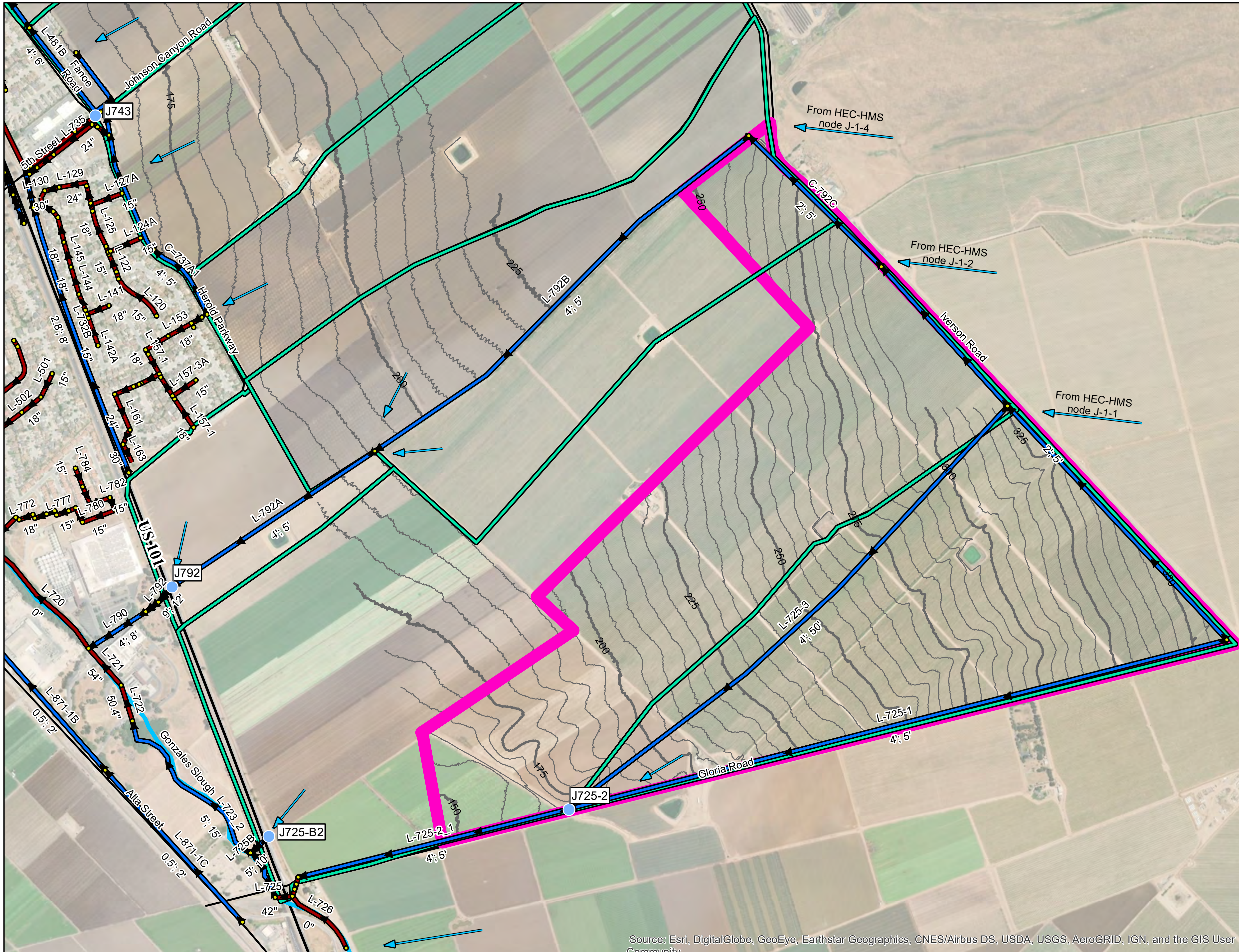
Prepared On: 3/1/2019



Legend

- Comparison Nodes
- Flow Direction
- Trapezoidal Channel - Depth; Bottom Width Labeled
- Pipe - Diameter Labeled
- Natural Channel
- Natural Wide Swale
- PCSWMM Drainage
- Gonzales Slough
- Contours - 25 ft
- Contours - 5 ft

0 1,000 2,000

1 in = 2,000 ft

Exhibit 10
Puente del Monte Study Area
Existing Conditions
PCSWMM Model Layout

Prepared By:
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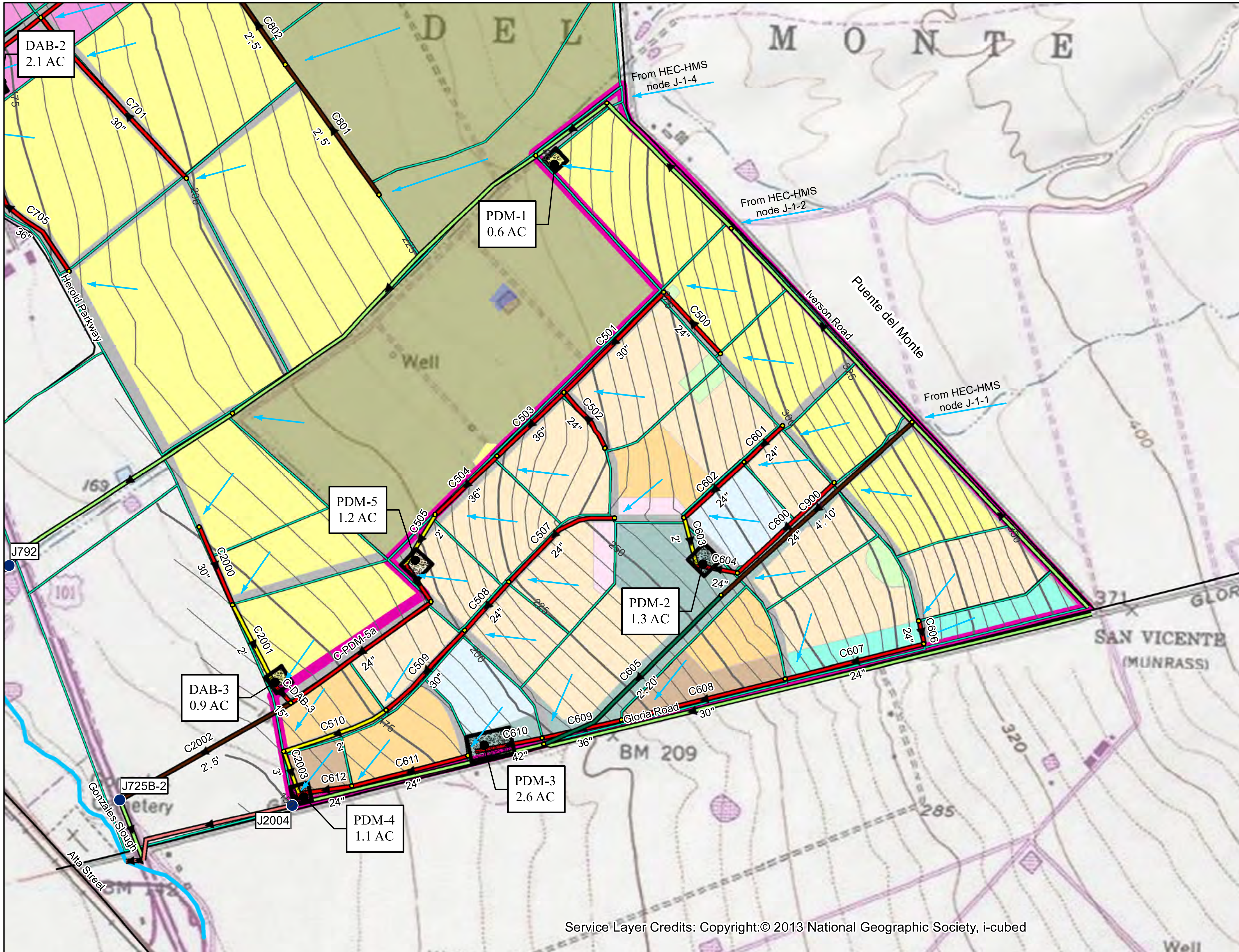
Prepared On: 2/12/2019



Legend

- Comparison Node
- Flow Direction
- Proposed Storage Basin
- Proposed Circular Pipe - Diameter Labeled
- Proposed Rectangular Pipe - Length and Width Labeled
- Proposed Concrete Channel - Depth; Bottom Width Labeled
- Proposed Compacted Dirt Channel - Depth; Bottom Width Labeled
- Existing Pipe
- Existing Natural Channel
- Existing Trapezoidal Channel
- PCSWMM Drainage Areas
- Proposed Contours - 25'
- Proposed Contours - 5'
- Gonzales Slough

Note 1: All proposed trapezoidal channels have a 2:1 side slope.
Note 2: See Exhibit 5 for Land Use Key.

0 1,000 2,000
1 in = 4,000 ft

Exhibit 11
Puente del Monte Study Area
Conceptual Drainage Layout

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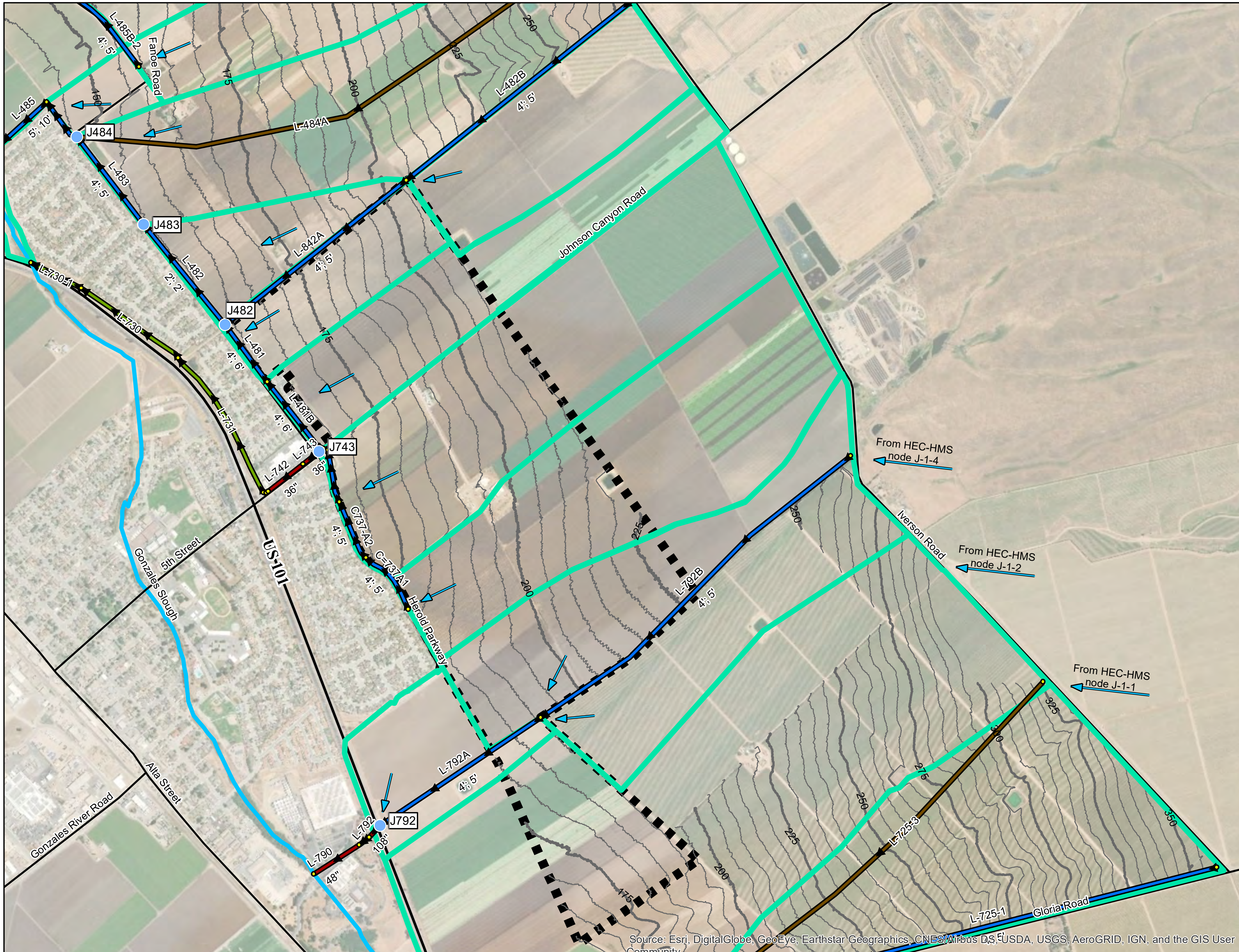
Prepared On: 3/4/2019



Legend

- Comparison Nodes
- Trapezoidal Channel - Depth; Bottom Width Labeled
- Pipe - Diameter Labeled
- Natural Channel
- Natural Wide Swale
- Flow Direction
- PCSWMM Drainage Areas
- Contours - 25 ft
- Contours - 5 ft
- Gonzales Slough

0 1,000 2,000

1 in = 2,000 ft

Exhibit 12
D'Arrigo Brothers Study Area
Existing Conditions
PCSWMM Model Layout

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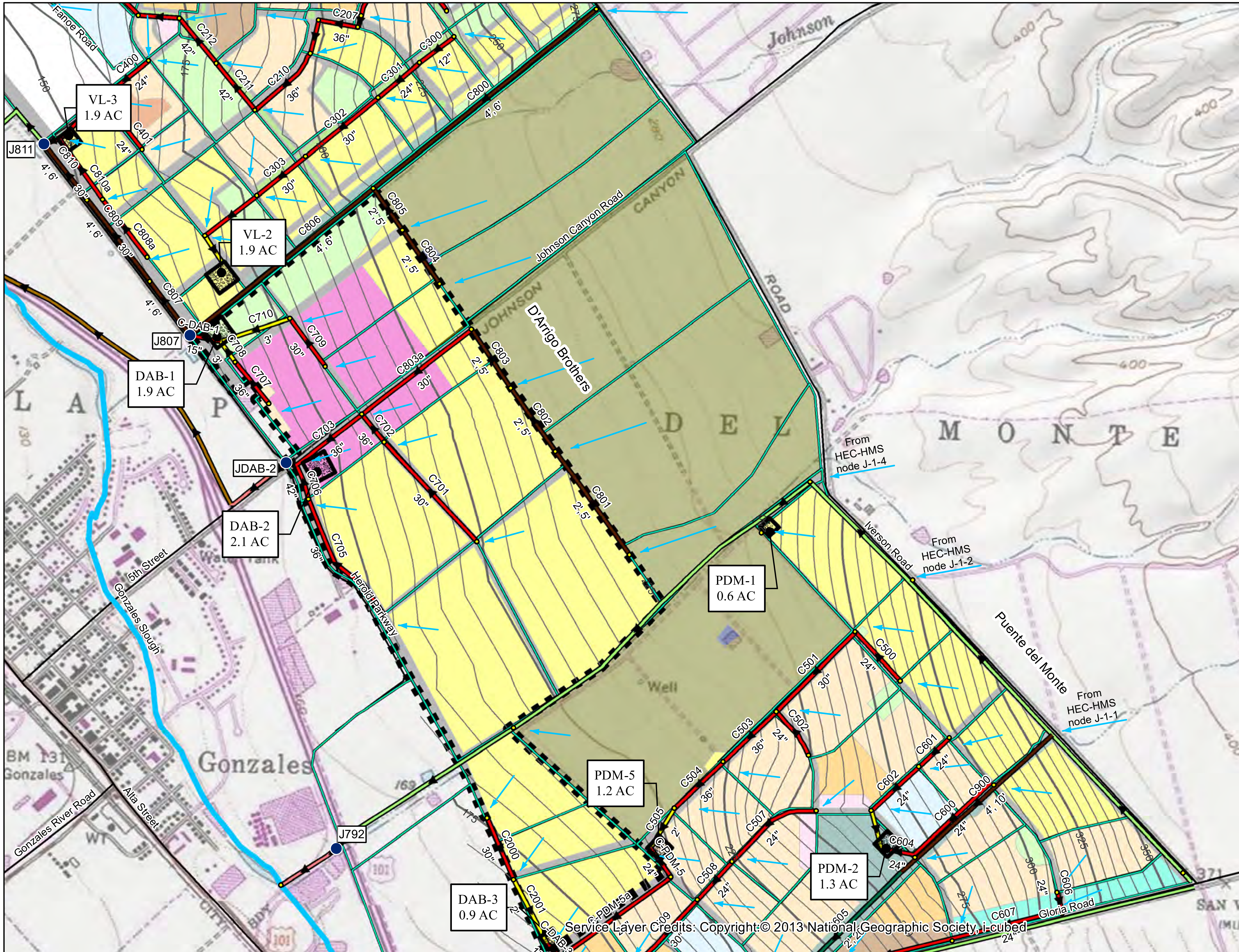
Prepared On: 3/1/2019



Legend

- Comparison Node
- Flow Direction
- Proposed Circular Pipe - Diameter Labeled
- Proposed Rectangular Pipe - Length and Width Labeled
- Proposed Concrete Channel - Depth; Bottom Width Labeled
- Proposed Compacted Dirt Channel - Depth; Bottom Width Labeled
- Existing Pipe
- Existing Natural Channel
- Existing Trapezoidal Channel
- PCSWMM Drainage Areas
- Proposed Storage Basin
- Proposed Contours - 25'
- Proposed Contours - 5'
- Gonzales Slough

Note 1: All proposed trapezoidal channels have a 2:1 side slope.
Note 2: See Exhibit 5 for Land Use Key.

0 1,000 2,000
1 in = 4,000 ft

Exhibit 13
D'Arrigo Brothers Study Area
Conceptual Drainage Layout

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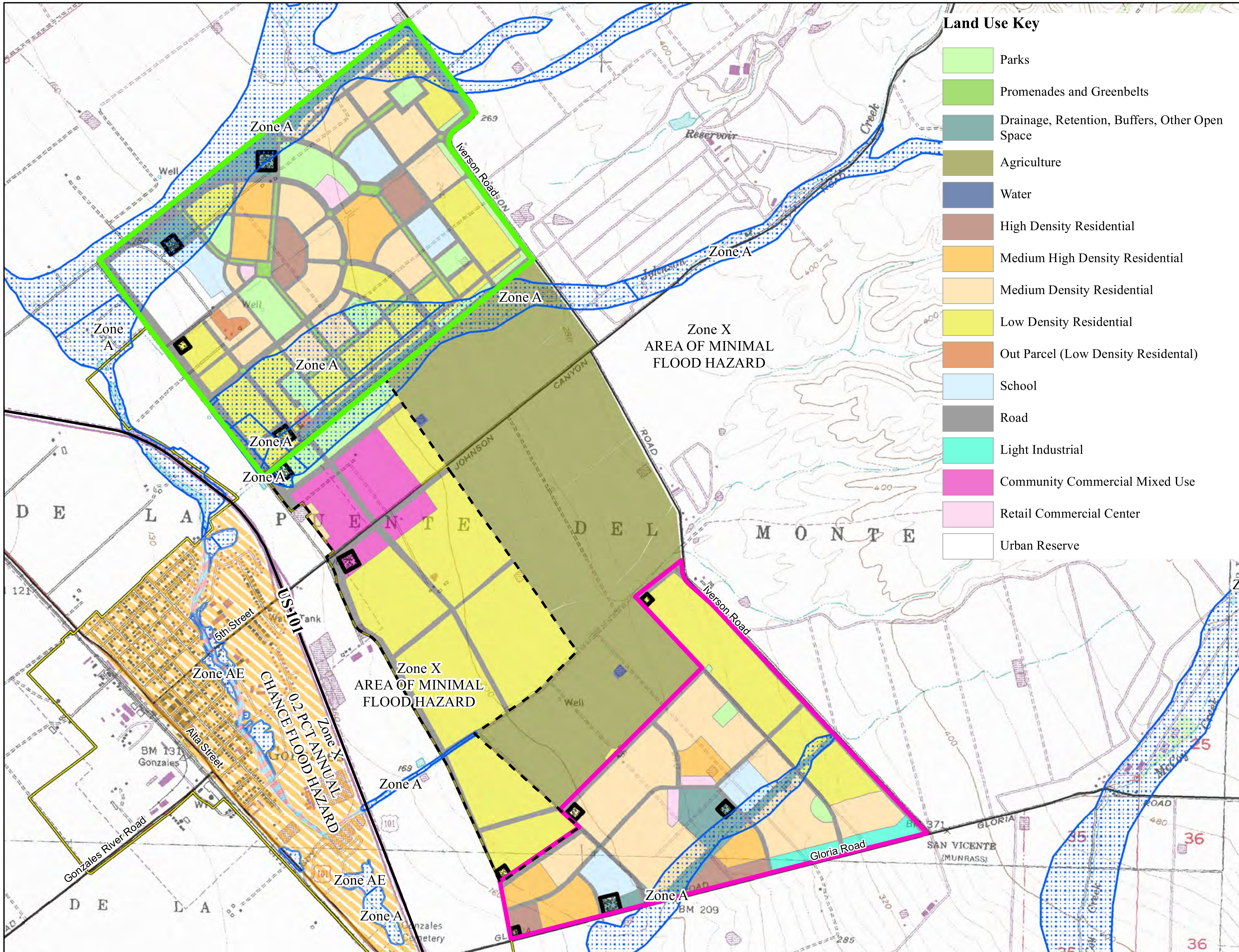

Legend

- Puente del Monte Study Area
- Vista Lucia Study Area
- D'Arrigo Brothers Study Area
- FEMA Flood Zone A
- FEMA Flood Zone AE Floodway
- FEMA Flood Zone X
- Proposed Storage Basin
- City of Gonzales

0 2,000 4,000

1 in = 4,000 ft

Exhibit 14
FEMA Flood Zone Map

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