APPENDIX G

HYDROLOGIC ANALYSIS

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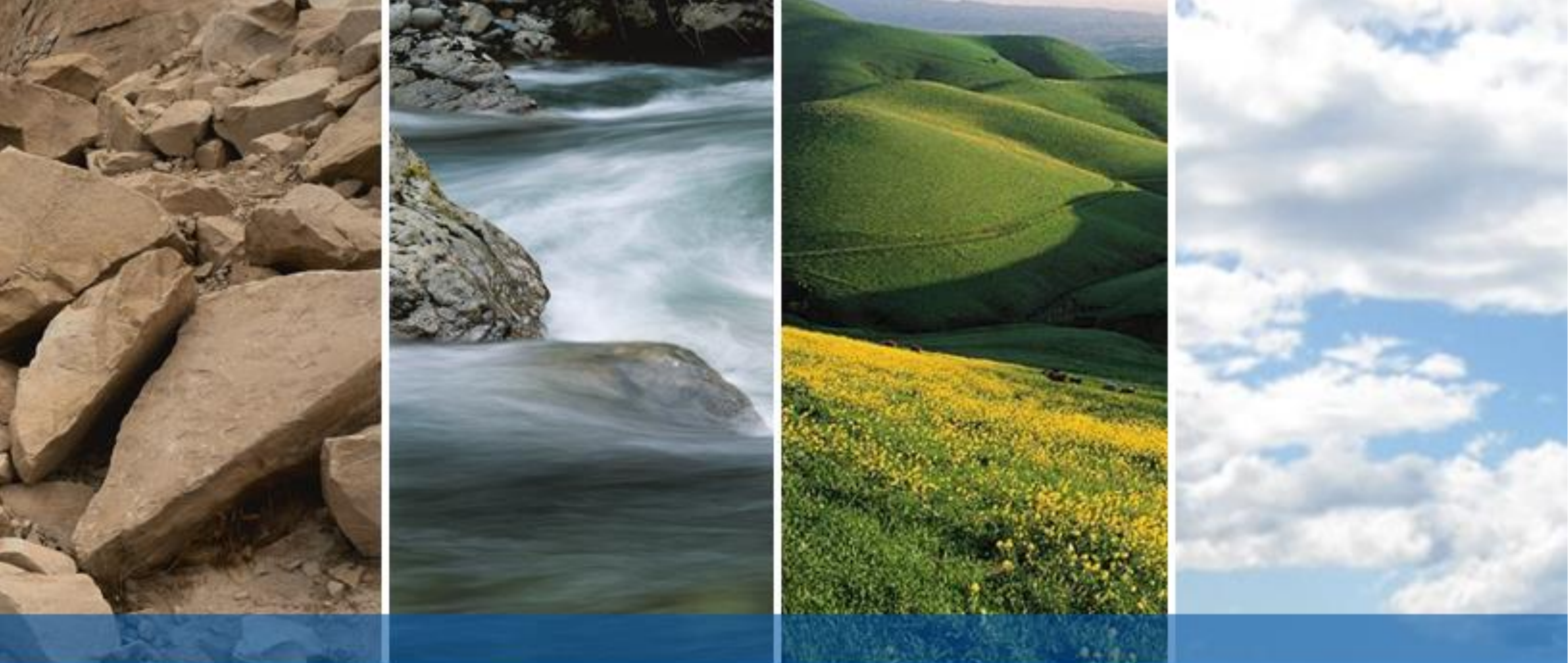

MONTE VISTA MEMORIAL GARDENS DEVELOPMENT
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

HYDROLOGIC ANALYSIS

SUBMITTED TO
Mr. Michael Kliment
Monte Vista Memorial Investment Group, LLC
189 Contractors Avenue
Livermore, CA 94551

PREPARED BY
ENGEO Incorporated

October 22, 2019
Revised November 18, 2019

PROJECT NO.
15426.000.000

ENGEO
Expect Excellence

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15426.000.000

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Mr. Michael Kliment
Monte Vista Memorial Investment Group, LLC
189 Contractors Avenue
Livermore, CA 94551

Subject: Monte Vista Memorial Gardens Development
3656 Las Colinas Road
Livermore, California

HYDROLOGIC ANALYSIS

Dear Mr. Kliment:

We are pleased to present this Hydrologic Analysis for the subject site, located at 3656 Las Colinas Road in Livermore, California (Site). The objectives of this study are to evaluate the effect of development of the project on peak flows within the Arroyo Las Positas watershed and to estimate water usage for the proposed project.

If you have any questions regarding this report, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

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TABLE OF CONTENTS

1.0 PROJECT DESCRIPTION

The proposed site is located in unincorporated Alameda County near the City of Livermore just north of Interstate 580, between the North Livermore Avenue and the North First Street exits. The proposed project would have a funeral home, an extensive cemetery grounds area, as well as a number of associated services, as further described below.

The site is approximately 66 acres and is bisected by Arroyo Las Positas in the southeast. The site generally consists of a relatively flat lowland valley area in the southeast, with gently sloping hills and valleys to the north and west. The localized ridges and valleys are oriented roughly north-south in the northern portion of the property, and roughly east-west in the western portion of the property, with valleys draining toward Arroyo Las Positas. Site slope gradients range from 2.5:1 to 10:1 (horizontal:vertical) in the surrounding hills (with the steepest slopes in the southwest), and the lowland valley area has a slope gradient shallower than 25:1 (horizontal:vertical). Furthermore, the site is bordered by an existing residence to the east and private undeveloped grazing land to the west and north. Currently, the portion of the site to the east of Arroyo Las Positas contains several paved roadways while the area on the west side of the Arroyo is undeveloped and used for grazing and dry farming.

The project proposes to develop 5.8 acres of the southwestern portion of the site into a funeral home, with parking facilities and an associated mortuary, crematorium and other services related to the burial grounds. The proposed project would include two bridges spanning over Arroyo Las Positas to connect the funeral home area to the cemetery grounds in the northwesterly portion of the site. The proposed cemetery grounds also would include several lake features, wetlands, lawn areas, and other landscape elements requiring the installation and maintenance of onsite water irrigation and management systems.

2.0 SITE GEOLOGY

Based on our geologic mapping and subsurface explorations from our 2018 Geotechnical Exploration Report (ENGEO, 2018), the site is underlain by young colluvial and alluvial deposits, as well as older Livermore Gravel deposits. Barlock (1988, 1989) describes the soil onsite as Holocene alluvium, and late-Miocene to early-Pleistocene Livermore Gravels. Furthermore, Dibblee (1980) confirms both the unit geology, and presence of regional folding onsite. We observed these units onsite, as well as Holocene colluvium and residual soil, during our 2018 exploration.

Soil at the site generally consists of interbedded layers of fine- and coarse-grained material associated with alluvial deposits and the Livermore Gravel formation. In general, the upper approximately 2 to 10 feet of soil we encountered in our explorations consisted predominantly of clay. Below the surficial clay layer, we encountered an approximately 5- to 10-foot-thick layer of generally medium dense to very dense coarse-grained material consisting of clayey sand, clayey gravel, silty sand, sand, and gravel. Below this granular layer, we encountered hard lean clay and silty clay with varying amounts of sand and gravel representative of the Livermore Gravel Formation.

Soil mapping of the watershed prepared by the National Resource Conservation Service (NRCS) for the Monte Vista Memorial Project (NRCS, 2019) indicates that surficial soil materials are primarily comprised of Altamont Clay (AaC) and Linee clay loam (LaD) with respective hydrologic group ratings of ‘C’ and ‘D’ among other soil groups of Group ‘C’ and ‘D’ rating.

Group 'C' soil is defined as having low infiltration rates or moderately high runoff potential when thoroughly wet. Group 'D' soil is defined as having a very slow infiltration rate or high runoff potential when thoroughly saturated. As a result, the watershed is characterized by moderate to rapid run-off characteristics after saturation has occurred.

3.0 HYDROLOGIC CALCULATIONS

For the Monte Vista Memorial project, we evaluated the use of the two proposed lakes to attenuate peak flows from the project before discharging into the Arroyo Las Positas Creek. We conducted our hydrologic calculations in accordance with methodologies set forth by Alameda County Flood Control and & Water Conservation Districts (Alameda County).

3.1 HYDROLOGIC METHODS

We used the Synthetic Unit Hydrograph Method as described in the Alameda County Hydrology and Hydraulic Manual (Alameda County, 2018) to develop peak flow hydrographs within the Site tributary watersheds of Arroyo Las Positas Creek. The Synthetic Unit Hydrograph Method is used for drainage areas greater than 0.5 square miles, for evaluation of detention basin design, or for situations where a hydrograph is required. The Synthetic Unit Hydrograph Method transforms a hypothetical rainfall distribution and rainfall depth into a design runoff hydrograph. The intent of this document is to use the Alameda County Synthetic Unit Hydrograph methodology input files within the framework of the Hydrologic Engineer Center-Hydrologic Modeling System program (HEC HMS). The Synthetic Unit Hydrograph method is dependent on the following parameters for each sub-basin.

1. Size of subwatershed.
2. Subwatershed infiltration rate.
3. Current/Proposed Land Use.
4. Overall watershed slope.
5. Lag Time – which is a function of longest channel length within the subwatershed as well as the subwatershed geometry.
6. Basin Peaking Factor.

We based the delineation of sub-basins for the study of the site contributing drainage areas on input from ACS Consulting Engineers tentative site development files (ACS, 2018) as well as Lidar topography provided by U.S. Geological Survey (USGS). Figures depicting the watershed in terms of infiltration rates and proposed development are included as Figures 2 and 3. Infiltration calculations weighted specifically for each hydrologic subwatershed as well as excerpts from the Alameda County Hydrology and Hydraulics Manual are included in Appendix A.

3.2 REOCCURRENCE INTERVAL

For this report, we analyzed the 24-hour, 10-year and 24-hour, 100-year recurrence interval storm events in conformance with Alameda County Flood Control methods.

These flows represent maximum flows in the pre-project condition and provide a baseline for designing post-construction flow control features for the developed scenario.

3.3 RAINFALL DATA

Published precipitation data provided on the National Oceanic and Atmospheric (NOAA) Atlas 14 Point Precipitation Frequency Estimates database provides precipitation frequency estimates with 90-percent confidence intervals for varied durations and average recurrence intervals. A 24-hour duration is most appropriate for use in evaluating a detention basin design per Alameda County standards.

TABLE 3.3-1: 24-hour Rainfall Depth for Selected Reoccurrence Intervals

	10-YEAR	100-YEAR
Rainfall Depth (inches)	2.91	4.90

Actual rainfall depths used in the hydrologic modeling are summarized in Appendix A for the 24-Hour Design Storm and are based on the Alameda County method of converting the estimated rainfall depth to a rainfall temporal distribution. The total watershed area draining into the local creek segments upstream of the box culvert at Interstate 580 is approximately 0.31 square miles.

3.4 PRE-DEVELOPMENT ASSESSMENT PARAMETERS

As shown in Figure 4, three drainage areas to the Site were subdivided into three Sub-Basins (A-1 through C-1).

TABLE 3.4-1: Pre-Development Basin Characteristics

SUB-BASIN	A-1	B-1	C-1
Area (Square Miles)	0.125	0.174	0.006
Longest Flow Path (feet)	3194	4553	655
Lc (Feet)	1273	2268	365
Initial Infiltration Loss (inches)	1.0	1.0	1.0
Uniform Infiltration Loss (inches/r)	0.067	0.111	0.058
Average Stream Slope (feet/mile)	192.4	123.9	12.7
Weighted Roughness Factor	0.07	0.07	0.07
% Impervious	0.0	0.0	20.9
Distance Factor	21.79	24.58	16.57
Basin Lag Time (hours)	0.27	0.47	0.12
Basin Peaking Factor	0.6	0.6	0.6

Specific lag time calculations are furnished in Appendix A of this document.

3.5 POST-DEVELOPMENT ASSESSMENT PARAMETERS

We modified the existing watersheds in our model to reflect the post-development conditions. As shown in Figure 5, contributing drainage areas to the site were subdivided into six Sub-Basins (A-2 through F-2). In general, the project is characterized by existing rural conditions, new landscaping, and low-density buildings (Sub-Basin E-2).

TABLE 3.5-1: Post-Development Basin Characteristics

SUB-BASIN	A-2	B-2	C-2	D-2	E-2	F-2
Area (Square Miles)	0.005	0.080	0.121	0.023	0.006	0.041
Longest Flow Path (feet)	470	2547	3730	1911	655	1245
Lc (Feet)	540	1079	1978	1087	365	788
Initial Infiltration Loss (inches)	1.0	1.0	1.0	1.0	1.0	1.0
Uniform Infiltration Loss (inches/r)	0.101	0.115	0.115	0.091	0.070	0.082
Average Stream Slope (feet/mile)	655.5	197.3	112.5	548.0	12.7	119.0
Weighted Roughness Factor	0.07	0.07	0.07	0.07	0.43	0.07
% Impervious	6.2	2.3	0.1	12.0	40.0	17.5
Distance Factor	16.19	20.46	22.89	19.15	16.57	17.78
Basin Lag Time (hours)	0.06[1]	0.22	0.39	0.15	0.72[2]	0.14
Basin Peaking Factor	0.69	0.60	0.60	0.68	0.60	0.60

[1]Modeled with basin lag time of 0.1 hours as this is the minimum criteria to run hydrologic model.
[2]Modeled with basin lab time of 0.1 hours to represent directly connected impervious areas within Sub-Basin E-2.

We used a basin lag time of 0.1 hours for Sub-Basin E-2 to demonstrate use of proposed directly connected impervious areas within the majority of the Sub-Basin. It is our experience that this assumption is typical for the type of proposed building use and drainage characteristics of this area. We assumed 40 percent imperviousness for the proposed Site use of Sub-Basin E-2 based on the proposed buildings, pervious pavers, and landscaping within the footprint.

3.5.1 Post Development Hydrologic Routing

A proposed creek will route discharge from the upper lake (Lake 2 on Figure 5) to the lower lake (Lake 1 on Figure 5). To maintain static or desired lake water levels as well as an equilibrium creek flow during the dry season, water from the lower lake will be re-circulated to the upper lake by use of a pump. This will help minimize water demand from groundwater or municipal sources to the maximum extent practicable. We used the Muskingum Cunge method as the routing method for this hydrologic model. The Muskingum Cunge is a widely accepted approach that uses reach length, channel slope and cross-sectional geometry to attenuate the flood hydrograph as it moves through each reach. We assumed a typical trapezoidal channel with the parameters shown in Table 3.5.1-1. We selected these parameters based upon our understanding of the proposed topography and creek routing.

TABLE 3.5.1-1: Muskingum-Cunge Routing Coefficients – Reach from Upper to Lower Lake

REACH	1	2
Length (ft)	543	635
Slope (ft/ft)	0.01	0.01
Manning's n	0.035	0.035
Index Flow (cfs)	75	75
Bottom Width (ft)	5	5
Side Slope (H:V)	3:1	3:1

We selected roughness coefficients based on Manning's method. We used a Manning's 'n' value of 0.035 to represent open channel roughness for a clean, windy creek with no pools or major rifts (Chow, 1959). We selected an index flow of 75 cfs based upon the average values of the respective hydrograph.

We understand sub-drains will capture and re-direct runoff surrounding proposed crypts. In our hydrologic model, we assumed runoff would be directed from Node 2 to the lower lake by a pump with an insignificant lag time associated with such.

3.6 POST-DEVELOPMENT PEAK DISCHARGE COMPARISON TO EXISTING CONDITIONS

We understand there will be two proposed discharge points from the site to Arroyo Las Positas. These locations are at the base of the proposed wetland and bio-filter (Figure 5). We modeled these two discharge points as Node 1 and Node 3, respectively. Below is a summary of the pre-and post- development discharges and volumes for the discharge points modeled. As discussed earlier, Node 2 is re-directed to the lower lake to capture additional runoff from crypt sub-drains within Sub-Basin F-2 as well as Sub-Basins D-2 runoff for the purpose of maintaining lake volume. The lower lake is to be directed to Node 3 in the event of overflow from the lower lake. Node 2 therefore is reflected within Node 1 in the post-development model.

TABLE 3.6-1: 10-Year Peak Flow Comparison of Pre- and Post- Development

NODE	1	2	3
Pre-Development Discharge (cfs)	56.2	62.5	2.9
Post-Development Discharge (cfs)	48.8	N/A	2.9
Pre-Development Volume Runoff (ac-ft)	7.2	7.7	0.5
Post-Development Volume Runoff (ac-ft)	18.4	N/A (0)	0.6

The above table demonstrates that the lakes will require additional storage of 3.5 acre-feet (ac-ft) to match pre-development flow during the 10-year rain event (Nodes 1 and 2). An additional 0.1 ac-ft detainment is required for Node 3.

TABLE 3.6-2: 100-Year Peak Flow Comparison of Pre- and Post- Development

NODE	1	2	3
Pre-Development Discharge (cfs)	98.5	114.6	5.0
Post-Development Discharge (cfs)	222.9	N/A	5.0
Pre-Development Volume Runoff (ac-ft)	18.8	20.9	1.1
Post-Development Volume Runoff (ac-ft)	45.0	N/A (0)	1.2

The above table demonstrates that the lakes will require additional storage of 5.3 ac-ft to match pre-development flow during the 100-year rain event (Nodes 1 and 2). An additional 0.1 ac-ft detainment is required for Node 3.

4.0 DETENTION SIZING

The project intends to decrease post-project peak flows by detaining water in the proposed lake features, which would offset increases in peak flow created by addition of impervious surfaces and modifications to existing surface drainage paths.

Node 3 accepts runoff from Sub-Basin C-1 in the pre- development model and Sub-Basin E-2 in the post- development model. Node 3 shows an additional 0.1 ac-ft discharge for the post-development conditions as compared to existing for the 100-year rain event. Additional runoff will be mitigated by use of a proposed bio-filter (approximate location shown on Figure 5). This bio-filter will accept drainage from the Site through drainage inlets or pervious pavers, treat this runoff, and ultimately discharge to Arroyo Las Positas Creek (Node 3).

As previously mentioned, the post-development model routes Node 2's discharge to the lower lake along with crypt sub-drain runoff from Sub-Basin F-2 to maintain lake water levels. We estimated approximately 5.3 ac-ft capacity required from the proposed lakes to detain the 100-year storm event. We estimated this volume from the ultimate discharge to Node 1 in the post- development model as compared to the discharge to Nodes 1 and 2 in the pre-development model. The lakes would be designed to provide adequate storage to collect excess stormwater and to meter the detained water through an engineered outfall structure. We understand an approximately 2.6-acre wetland mitigation area, as shown in Figure 5, will be implemented to handle discharge from the lakes during high flow events. This wetland mitigation area would discharge at the study 10-year and 100-year pre-development flow rates into the Arroyo Las Positas Creek.

It is our opinion that this estimate is considered conservative as the infiltration potential for soil within the burial areas and new landscaping will increase due to the disturbance of soil.

5.0 WATER BALANCE – PROPOSED WATER FEATURES

The proposed lakes and creeks will be operated as a water feature on the Monte Vista Memorial project. We conducted a water balance analysis on the proposed system to determine the availability of water for the system and the amount of additional water that would be needed, if any, to support the lakes and creek. The water budget defines and quantifies the important input and output parameters, such as precipitation, evapotranspiration, and flow into or out of a given body of water. We analyzed each of these parameters individually to develop expected numerical value flux estimates, and the sum of the parts provides an estimate of available water at a given time step. For this project, this summation analysis approximates the volumes of available surface water expected to flow through, or be retained in, the lakes and creek each month.

The proposed system includes two separate lakes with a creek connecting them. The upper lake consists of two different pools, an upper and lower pool. The upper pool flows into the lower pool via a waterfall feature as detailed in the provided Landscaping plan. The lower pool of the upper lake drains into the creek, which flows into the lower lake. The upper lake's water supply can be supplemented by an onsite groundwater well. A pump powered by solar panels recirculates water from the lower lake to the upper pool of the upper lake. The lower lake will act as a reservoir for irrigation water needed for the landscaping on the site.

The lower lake consist of a steeper portion in the deepest parts of the lake that level out to form a shelf. This shelf portion of the lake is sized to retain storm events but will not typically hold water throughout the year.

5.1 TOPOGRAPHY, WATERSHEDS, AND PROPOSED LAKES AND CREEK

All proposed lakes are located in the Arroyo Las Positas Watershed. We used the same sub-basins used for the study of the hydrologic study for the water balance study. We based the locations and initial sizes of the proposed ponds and creek on the input from ACS Consulting Engineers tentative site development files (ACS, 2018).

5.2 HYDROLOGIC INPUTS

5.2.1 Precipitation and Water Year Types

We used data from The National Climatic Data Center (NCDC) of the National Oceanic and Atmospheric Administration (NOAA) which records daily precipitation for Livermore, California (NCDC,2019) Station GHCND : USC00044997 and extends from 1903 through 2018. However, in order to maintain consistency between different water budget data sets, we performed our analyses utilizing data from Water Year 2 (WY) 1969 through 2017 (October 1968 through September 2017), as this time period correlates with the available pan evaporation data, discussed later. The long-term (WY1969-2017) average annual rainfall estimate from these data is 14.06 inches. The value agrees well with the USGS estimate for mean annual rainfall of 15.0-inches for this site location (Rantz, 1971).

The USGS defines a water year as the 12-month period from October 1, for any given year, through September 30, of the following year. The water year is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the year ending September 30, 1999, is called the "1999" water year and includes 9 of the 12 months.

We used the monthly data set to determine the rainfall-runoff year-type probability analysis, described below.

We summed and ranked monthly average rainfall values for WY 1969 – 2017 (NCDC, 2019) by water year. The exceedance probability ranking of the annual rainfall values suggests the long-term (1969-2017) average value of 14.06 inches has about a 40 percent probability of occurring any given year. Thus, the statistical average value does not equal the median value. The median year in the data set, or that with a 50 percent probability of occurring within any given year, is WY 2004, and we used data for this year for the median year-type analysis. WY2004 generated 13.07 inches of rainfall annually. We selected a water year-type with an 85 percent probability of occurring within any given year was selected as the representative dry year-type. The Water Year exhibiting the 85 percent probability range is WY 1990; we used this water year for the dry year-type analysis. WY1990 generated 9.35 inches of rainfall annually. Average, median, and dry year-type monthly rainfall totals are presented in Tables 5.2.3-1, 5.2.3-2 and 5.2.3-3, respectively.

5.2.2 Runoff

We could not identify a suitable local area stream flow gauges to estimate surface water runoff from the site. Therefore, we calculated runoff contributing to the Monte Vista Memorial project using the Natural Resource Conservation Service (NRCS) Runoff Curve Number (CN) Method (NRCS, 1986). The CN method approximates volume of direct surface runoff as a function of daily (24-hour) rainfall (P), the potential maximum retention after runoff begins (S), the initial abstraction (Ia), and the curve number (CN). Estimated as 20 percent of the value for S, Ia accounts for the total water losses before runoff begins and includes depression storage, interception, evaporation, and infiltration. S is directly related to CN, a function of hydrologic soil group (HSG), cover type, treatment, hydrologic condition, and antecedent runoff condition.

Soil data (NRCS, 1966, 1977, 2007, 2010) overlaid onto contributing watersheds resulted in HSG coverage of type C and type D. Both HSG C and HSG D soil has high runoff potential and low infiltration rates when thoroughly wetted (Figure 4). To calculate the composite CN for the site, a weighted average of the soil types is calculated. Assuming cover type is pasture, grassland, or range, with 50 to 75 percent ground cover and not heavily grazed (fair condition), with approximately 43 percent HSG C and 57 percent HSG D, the universal CN calculated from Table 2-2c (NRCS, 1986) was 79 for HSG C and 84 for HSG D. The composite CN is 81.85.

Based on a CN value of 81.85, we estimated the parameters to the runoff equations as:

S = (1000 / CN) – 10 = 2.2 inches
Ia = 0.2 * S = 0.44 inches

These data indicate that within a 24-hour period, the initial 0.44 inch of rainfall goes towards depression storage, interception, evaporation, and infiltration. Below this initial rainfall total, no runoff occurs. Rainfall in excess of 0.44 inch generates surface water runoff (Q) by the equation:

$Q = ((P - 0.2S)^2) / (P + 0.8S)$

Using the daily rainfall total data and runoff equation discussed above, daily runoff totals for the entire analysis period (WY1969-2017) were calculated. The average monthly values over the entire analysis period were used in the average water year type water budget analysis and are presented in Table 5.2.3-1. The long-term (WY1969-2017) average annual rainfall estimate of 14.16 inches generates an average annual runoff value of nearly 4.81 inches per year.

We calculated median and dry year type runoff totals using the runoff equation and daily rainfall totals for WY2004 (median year type) and WY1990 (dry year type). The resulting annual runoff totals for WY2004 and WY1990 were 5.99 and 2.64 inches, respectively (Tables 5.2.3-2 and 5.2.3-3).

5.2.3 Evaporation

We performed this analysis considering monthly pan evaporation using data at Lake Del Valle in Livermore for Water Year 1969 - 2017 obtained from Zone 7 Water Agency (Zone 7 Water Agency, 2018). We converted pan evaporation data to an open-water evaporation rate by multiplying by a coefficient of 0.6402 (Zone 7 Water Agency, 2018). Average, median, and dry year-type monthly evapotranspiration values are presented in Tables 5.2.3-1, 5.2.3-2 and 5.2.3-3, respectively.

TABLE 5.2.3-1: Average Year-Type Annual Input Values WY 1969-2017

MONTH	PRECIP. (INCHES)	RUNOFF (INCHES)	ETO (INCHES)
October	0.81	0.05	3.28
November	1.62	0.41	1.55
December	2.51	1.00	0.96
January	2.83	1.24	0.92
February	2.55	1.03	1.25

MONTH	PRECIP. (INCHES)	RUNOFF (INCHES)	ETO (INCHES)
March	2.06	0.68	2.32
April	1.04	0.13	3.50
May	0.38	0.00	5.01
June	0.10	0.06	6.02
July	0.02	0.10	6.85
August	0.06	0.08	6.23
September	0.19	0.03	5.04
Annual	**14.16**	**4.81**	**42.92**

TABLE 5.2.3-2: Median Year-Type (50th Percentile of Being Equaled or Exceeded) for WY 1969-2017 is WY 2004

MONTH	PRECIP. (INCHES)	RUNOFF (INCHES)	ETO (INCHES)
October	0.02	0.10	4.30
November	2.02	0.66	1.10
December	3.57	1.83	0.72
January	2.19	0.77	0.69
February	4.01	2.20	1.42
March	0.39	0.00	3.19
April	0.18	0.04	4.72
May	0.11	0.06	5.54
June	0	0.11	6.06
July	0	0.11	6.50
August	0	0.11	6.33
September	0.58	0.01	5.61
Annual	**13.07**	**5.99**	**46.18**

TABLE 5.2.3-3: Dry Year-Type (85th Percentile of Being Equaled or Exceeded) for WY 1969-2017 is WY 1990

MONTH	PRECIP. (INCHES)	RUNOFF (INCHES)	ETO (INCHES)
October	1.13	0.16	3.11
November	1.02	0.12	1.89
December	0.1	0.06	1.12
January	1.54	0.36	1.01
February	2.46	0.96	1.17
March	0.87	0.07	2.33
April	0.37	0.00	3.67
May	1.78	0.50	5.03
June	0	0.11	5.88
July	0.02	0.10	6.52
August	0	0.11	5.90
September	0.06	0.08	4.54
Annual	**9.35**	**2.64**	**42.17**

5.3 INFILTRATION

The site consists of HSG C and HSG D, which indicates extremely low rates of vertical infiltration. Thus, for this analysis, we considered the infiltration to be negligible.

5.4 LANDSCAPING DEMANDS

The provided landscaping plan detailed a variety of plantings that would be used for the landscaping. This included different cover grasses as well as shrubs and trees. In this analysis, the landscaping water demand was split into planting demand and lawn maintenance demand.

We estimated the cover area of plantings using the provided Landscaping Plan (Camp & Camp, 2018). We based the water requirements for the plantings on the Water Use Classification of Landscape Species (WUCOLS), which determines the plant water requirements based on a percentage of the reference evapotranspiration in the area, shown in Table 5.4.1 along with the coverage area of the plantings. The water requirement for the plantings are calculated based on these classifications and the reference evapotranspiration (ETo) for Livermore taken from the Model Water Efficient Landscape Ordinance (Table 5.4.2). The resulting plant water demand on the site is shown in Table 5.4-3

TABLE 5.4-1: Planting Water Requirements

PLANT NAME	WUCOLS CLASSIFICATION	PERCENTAGE OF EVAPOTRANSPIRATION	COVERAGE AREA, ACRES
Aesculus californica	Very Low	10	0.193
Lagerstroemia ‘Muskogee’	Low	30	0.270
Laurus nobilis ‘Saratoga’	Low	30	0.533
Platanus racemose	Moderate	60	1.174
Quercus agrifolia	Very Low	10	1.336
Quercus lobata	Low	30	2.268
Arcstostaphylos ‘Howard McMinn’	Low	30	0.103
Ceanothus ‘Dark Star’	Low	30	0.093
Cercis occidentalis	Low	30	0.047
Olea europaea ‘Arbequina’	Low	30	0.296

TABLE 5.4-2: Reference Evapotranspiration for Livermore

MONTH	REFERENCE EVAPOTRANSPIRATION, IN/MONTH
October	3.2
November	1.5
December	0.9
January	1.2
February	1.5
March	2.9
April	4.4
May	5.9
June	6.6

MONTH	REFERENCE EVAPOTRANSPIRATION, IN/MONTH
July	7.4
August	6.4
September	5.3
Annual	**47.2**

We calculated the lawn maintenance demand based off of a crop coefficient given by a publication by the University of California Division of Agriculture and Natural Resources of 0.8 (Harivandi et al., 2009) and the reference evapotranspiration value shown in Table 5.4-2. We converted these values to a lawn maintenance demand by multiplying by the lawn coverage area.

The results of the landscaping water requirements are shown below in Table5.4-3.

TABLE 5.4-3: Landscaping Water Demand

MONTH	PLANTING DEMAND, AF	LAWN MAINTENANCE DEMAND, AF
October	0.20	1.62
November	0.25	2.03
December	0.48	3.92
January	0.73	5.94
February	0.98	7.97
March	1.10	8.91
April	1.23	9.99
May	1.07	8.64
June	0.88	7.16
July	0.53	4.32
August	0.25	2.03
September	0.15	1.22
Annual	**7.86**	**63.75**

The irrigation system for the project site will draw water from the lower lake. The lawn areas are underlain by a French drain system that will capture water that has infiltrated the soil and return it back to the lower lake for reuse. We estimated the landscaping return value as 20 percent of the applied water, the larger of the direct precipitation or landscaping demands., based on the CN method's initial abstraction, discussed above. The initial abstraction is the amount of rainfall that goes towards depression storage, interception, evaporation, and infiltration.

5.5 WELL PRODUCTION

The project site has a groundwater well that can be used to supplement the water supply to the upper lake. Based on a 24-hour flow test conducted by Pacific Coast Well & Pump Inc. in July 2012, the well has an average production capacity of 200 gallons per minute. This is approximately equal to 0.88 AF of water per day.

The well draws from the Livermore Valley Groundwater Basin. The Livermore Valley Groundwater Basin spans approximately 69,600 acres (109 square miles) and has an approximate capacity of 500,000 AF. The Alameda County Flood Control and Water Conservation District, Zone 7 manages groundwater in the basin. Zone 7 has maintained an annual hydrologic budget. Under average conditions, the groundwater budget has remained in balance with the demands balancing the inflows. The estimated groundwater storage in 1999 was 219,000 AF. Due to higher than usual rainfall in 2017 WY, the groundwater storage increased to 246,000 AF.

The groundwater-bearing materials in the basin include valley-fill materials, the Livermore Formation, and the Tassajara Formation. The valley-fill materials are composed of unconsolidated sand, gravel, silt, and clay. The valley-fill materials yield significant quantities of water to wells in the central and southern portions of the valley. The Livermore Formation is primarily exposed over the south and southwest regions of the Livermore Valley groundwater basin. The Livermore Formation consists of unconsolidated to semi-consolidated beds of gravel, silt, and clay. Limey concretions are common in its lower portion and tuffaceous beds are present at its base. The Livermore Formation supplies water to deep wells in the eastern half of the basin. The Tassajara Formation is exposed in the uplands to the north of the Livermore Valley. The Tassajara Formation consists of beds composed of sandstone, siltstone, shale, conglomerate, and limestone. Wells tapping into the Tassajara Formation yield only sufficient water for domestic or stock purposes.

For management purposes, the Livermore Valley groundwater basin is also split based on varying geologic, hydrogeologic, and groundwater conditions. These are the Main Basin, Fringe Subareas, and Upland Areas. The project site is located within the Upland and is underlain by the Livermore Formation.

For the purpose of sustainable groundwater management, the groundwater well draw was limited to 150 gpm, or approximately 0.66 AF of water per day.

5.6 RECIRCULATION PUMP

A pump will be used to recirculate water from the lower lake to the upper lake to reduce the demand of well water to sustain the system. The pump will be powered by solar panels on site. We modeled the pump with a six-hour operational time each day to account for the amount of time the solar panels would be receiving sunlight. A pump with a capacity of around 1850 gpm would be needed to recirculate enough water within the allotted operational time. The pump would be able to move approximately 60 AF of water per month. If solar power is not sufficient for the operation of the pump, the power supply will be supplemented by another electrical source.

5.7 BUILDING DEMAND

The building area including the main funeral home and the reflecting pool will be supplied by a municipal water connection and the demands from this area were not included in the overall water budget.

5.8 MODELING APPROACH

We applied the following performance criteria to the analysis. First, a constant flow of 1 cubic foot per second was to be maintained through the creek at all times to ensure adequate flow in the creek. The second objective was to design the lakes to be at full depth for the entirety of the year.

5.9 WATER BUDGET ANALYSES

Our water budget analysis consisted of processing monthly inflow, outflow, and storage volume changes for both lakes and the creek that connects them.

A typical water budget for a lake system accounts for the monthly inflows, outflows and changes in lake storage as described below:

- Monthly direct rainfall inflow is converted from inches to volume (acre-feet) by multiplying monthly rainfall by the “total” lake surface area[1].
- Monthly surface water runoff is converted from inches to volume (acre-feet) by multiplying monthly runoff from the contributing watershed area, excluding lake (net drainage area). Sub-catchment C flows into the upper lake, Sub-catchment B flows into the creek, and Sub-catchments A, D, and F flows into the lower lake, shown in Figure 5.
- Monthly evapotranspiration outflow is converted from inches to volume (acre-feet) by multiplying the previous end of month lake surface area by the evapotranspiration.
- Accounting for the rainfall, runoff, evapotranspiration, landscaping demands, and landscaping return volumes for each month produces the monthly lakes-creek inflow balance. The monthly inflow balance is positive if the sum of rainfall, runoff, and landscaping return exceed evapotranspiration and landscaping demand losses. Or, the monthly inflow balance may be zero when evapotranspiration and landscaping demand volume is greater than contributing rainfall, runoff, and landscaping return volumes. Monthly inflow is added cumulatively, month by month, with any negative monthly values converted to zero to account for dry months.
- End-of-month lake storage is calculated during filling and draining sequence based on the stage-area-volume relationships derived for each lake. Outflow or spillage from the lake is quantified should inflows exceed total lake capacity and converting any negative monthly values to zero, accounting for the months where the pond dries out. End of month stage (lake water depth) and lake surface areas are calculated from the lake volume using the stage-area-volume relationships. The end of month lake surface area is used in the water budget to calculate the amount of evaporation occurring in the following month.
- Should spillage from the upper lake occur once capacity is exceeded, the monthly spillage volume is accounted for as an additional inflow volume to receiving creek and lower lake. Should spillage from the lower lake occur once capacity is exceeded, additional spillage would be routed to the proposed wetland area discussed later in this report.

5.10 RESULTS-PROPOSED LAKES AND CREEK SYSTEM

There is sufficient water supply to sustain the proposed lakes and creek system. Results of the water budget analyses indicate that the water inflow into the lakes and creek and the supplemental water from the onsite groundwater well are sufficient in achieving target lake

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[1] All direct rainfall enters the lake area even if it falls on the side slopes of the empty lake shelf, as it is assumed to runoff and pool at the bottom of the lake.

water depths and creek flow throughout the year for the average, median, and dry years. These lakes are also large enough to retain the 100-year storm detailed in Section 3. Table 5.10-1 below shows the proposed lake sizing based on the input from ACS Consulting Engineers tentative site development files (ACS, 2018). The table shows the full lake depth, volume and surface area, along with the additional depth, additional volume, and total surface area created by the shelf. This lake sizing is subject to change upon further analysis.

TABLE 5.10-1: Summary of Lake Sizing

LAKE	"FULL" LAKE DEPTH (FEET)	"FULL" LAKE VOLUME (AF)	"FULL" LAKE SURFACE AREA (ACRES)	ADDITIONAL SHELF DEPTH (FT)	ADDITIONAL SHELF VOLUME (AF)	TOTAL LAKE SURFACE AREA WITH SHELF (ACRES)
Upper Lake, Upper Pool	16	7.06	0.47	N/A	N/A	N/A
Upper Lake, Lower Pool	38	16.86	0.49	N/A	N/A	N/A
Lower Lake	16	19.21	1.36	1	10.04	2.09

For this analysis, we maximized the recirculated water amount based on the amount of water in the lower lake up to a maximum value of 60 AF per month. We assumed the remaining water needs of the system will be supplemented by the well water. The lakes-creek system well-water demand, the recirculation volume, and overflow volume from the lower lake for the average, median, and dry year are shown below in Tables 5.10-2, 5.10-3 and 5.10-4, respectively.

TABLE 5.10-2: Water demand, Recirculation, and Overflow Amount of Lake-Creek System for Average Year

MONTH	WELL-WATER SUPPLY (AF)	RECIRCULATION WATER (AF)	OVERFLOW VOLUME FROM LOWER LAKE (AF)
October	1.55	59.58	0.0000
November	0.00	60	0.0000
December	0.00	60	4.7594
January	0.00	60	8.0859
February	0.00	60	5.1671
March	0.00	60	0.0000
April	1.45	57.45	0.0000
May	6.14	58.86	0.0000
June	6.68	52.91	0.0000
July	7.27	54.14	0.0000
August	6.26	55.21	0.0000
September	5.36	56.39	0.0000
Annual	**34.71**	**694.54**	**18.012**

TABLE 5.10-3: Water demand, Recirculation, and Overflow Amount of Lake-Creek System for Median Year

MONTH	WELL-WATER SUPPLY (AF)	RECIRCULATION WATER (AF)	OVERFLOW VOLUME FROM LOWER LAKE (AF)
October	2.44	58.52	0.0000
November	0.00	60	0.0000
December	0.00	60	16.2164
January	0.00	60	1.3607
February	0.00	60	21.2563
March	3.30	58.45	0.0000
April	5.34	54.66	0.0000
May	7.14	54.51	0.0000
June	7.81	60	0.0000
July	8.65	52.76	0.0000
August	7.41	54.55	0.0000
September	6.17	53.7	0.0000
Annual	**48.26**	**687.15**	**38.833**

TABLE 5.10-4: Water demand, Recirculation, and Overflow Amount of Lake-Creek System for Dry Year

MONTH	WELL-WATER SUPPLY (AF)	RECIRCULATION WATER (AF)	OVERFLOW VOLUME FROM LOWER LAKE (AF)
October	0.5	58.52	0.0000
November	0.00	60	0.0000
December	1.18	60	16.2164
January	0.00	60	1.3607
February	0.00	60	21.2563
March	1.31	58.45	0.0000
April	5.22	54.66	0.0000
May	7.14	54.51	0.0000
June	7.57	60	0.0000
July	8.76	52.76	0.0000
August	7.33	54.55	0.0000
September	6.05	53.7	0.0000
Annual	**37.92**	**685.96**	**3.919**

Tables 5.10-3 and 5.10-4 show that the overall well-water supply to the lakes is greater in a median year than in a dry year. This is due to the distribution of rainfall during the selected years of WY 1990 and WY 2004. Although there is less overall rainfall in the statistically dry year, WY 1990, the rainfall is distributed throughout the year in such a way that the water demands of the site are better met than using the distribution from the statistically median year, WY 2004.

Based on the overflow results, a drawdown schedule would need to be established to reduce the chance of overflow events leading up to the months of December, January, and February.

6.0 WATER BALANCE – PROPOSED WETLAND

We conducted an additional water balance analysis on the proposed direct precipitation wetlands located on the southern part of the project site to determine the surface water supply available to the wetland. These wetlands are approximately 2.6 acres and will be implemented to discharge from the lakes during high flow events.

6.1 HYDROLOGIC INPUTS

6.1.1 Precipitation

We also used the precipitation inputs used in the proposed lakes and creek system analyses in the analyses for the proposed wetland.

6.1.2 Runoff

Based on the final grading of the site, the wetland area will not receive significant runoff. Therefore, we set the runoff inputs for the wetland to zero for all months.

6.1.3 Evapotranspiration

To determine the evapotranspiration of the wetland area, we used the landscape coefficient method per the University of California Cooperative extension California Department of Water Resources (2000). We based the coefficient on the species of plant and the classification under the WUCOLS, the density of the plantings, and the microclimate in which the landscape is located. Each component generates a related coefficient and the landscaping coefficient is the product of the generated component coefficients. The resulting coefficients for the wetland area are shown in the table below.

TABLE 6.1.3-1: Calculation of the Landscape Coefficient

COEFFICIENT TYPE	VALUE
Species	0.2
Density	0.7
Microclimate	1.0
Landscape	0.14

To determine the predicted evapotranspiration for each month, we multiplied the landscape coefficient by the reference evapotranspiration for Livermore shown in Table 5.4-2.

6.2 MODELING APPROACH

The intent of the model was to determine periods of saturation that are likely to occur in the proposed wetlands. Wetlands are considered saturated if hydrologic inputs exceed outputs for any given month.

6.3 WATER BUDGET ANALYSES

The water budget analysis consisted of processing monthly inflow values and outflow values for the proposed wetland area.

The wetland water budget accounts for the monthly inflows and demands as described below:

- Monthly direct rainfall inflow is converted from inches to volume (acre-feet) by multiplying monthly rainfall by the wetland area.
- Monthly evapotranspiration outflow is converted from inches to volume (acre-feet) by multiplying the landscape coefficient by the by the wetland area.
- Accounting for the rainfall and evapotranspiration volumes for each month produces the monthly wetland inflow balance. The monthly inflow balance is positive if the sum of rainfall and runoff exceed evapotranspiration losses; or, the monthly inflow balance may be zero when evapotranspiration volume is greater than contributing rainfall and runoff volumes. Monthly inflow is added cumulatively, month by month, with any negative monthly values converted to zero to account for dry months.

6.4 RESULTS – PROPOSED WETLANDS

Results of the analysis are provided in table 6.4.1 below. In general, there is sufficient water supply to achieve creation of the proposed wetland. The wetland area will be expected to be saturated for an average of 6 months every year.

TABLE 6.4-1: Predicted Wetland Saturation

YEAR TYPE	MONTHS
Average	7
Median	4
Dry	6

Table 6.4-1 shows the wetlands being saturated for a longer period of time during a dry year than in a median year. This is due to the difference in the distribution of rainfall for WY 1990 and WY 2004. Although there is less precipitation during the dry year, it is distributed to better meet the water demands of the site.

7.0 CONCLUSION

Based on the results of our modeling, the Monte Vista Memorial Garden project will not increase peak flows in the Arroyo Las Positas watershed downstream of the project if built in accordance with recommendations made herein. We expect this hydrologic model will need to be updated prior to approval of the final map of the project in order to assess any future modifications to the land plan.

In addition, based on the water balance analyses, there is an adequate water supply to sustain the Monte Vista Memorial Garden Project's proposed water features and proposed wetland.

If you have any questions on any portion of this report, please call and we will be glad to discuss them with you.

SELECTED REFERENCES

ACS Consulting Engineers; Monte Vista Memorial Gardens Plan Set; APN 090-0015-016-03; 3656 Las Colinas Road, Livermore, California; November 28, 2018.

Alameda County Flood Control & Water Conservation District; Hydrology and Hydraulics Manual; 2018.

Barlock, V.E., 1988, Geologic map of the Livermore gravels, Alameda County, California; U.S. Geologic Survey, Open-File Report OF-88-516, scale 1:24,000.

Barlock, V.E., 1989, Sedimentology of the Livermore gravels (Miocene-Pleistocene), southern Livermore Valley, California: U.S. Geological Survey, Open-File Report OF-89-131, scale 1:48,000.

Camp & Camp Associates; Monte Vista Memorial Gardens, Overall Master Plan, Livermore, California; Sheet L-1; May 2018, Job No. 10-017

Chow, V.T., 1959, Open-Channel Hydraulics: New York, McGraw-Hill, 680 p.

Department of Water Resources; Model Water Efficient Landscape Ordinance; September 10, 2009.

Dibblee, T.W., 1980, Preliminary geologic map of the Livermore quadrangle, Alameda and Contra Costa Counties, California: U.S. Geological Survey, Open-File Report OF-80-533-B, scale1:24,000.

Dibblee, T.W., and Minch, J.A., 2006, Geologic map of the Livermore quadrangle, Contra Costa & Alameda Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-196, scale 1:24,000.

ENGEO: Geotechnical Exploration Report; 3656 Las Colinas Road, Livermore, California; December 21, 2018.

Harivandi et al.; Managing Turfgrasses during Drought. University of California Division of Agriculture and Natural Resources Publication 8395; August 2009.

National Resources Conservation Service (NRCS); Urban Hydrology for Small Watersheds, Technical Release 55; 1986.

NRCS; Custom Soil Resource Report for Alameda Area, California; Monte Vista Memorial Project; October 9, 2019.

NOAA; Hydrometeorological Design Studies Center; Precipitation Frequency Data Server (PFDS); Atlas 14 Point Precipitation Frequency Estimates: KS. https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html.

University of California Cooperative Extension California Department of Water Resources; A Guide to Estimating Irrigation Water Needs of landscape Plantings in California; 2000

U.S. Army Corp. of Engineers; Hydrologic Modeling System (HEC-HMS); Version 4.3. September 2018.

U.S. Department of Agriculture, Natural Resources Conservation Service; 2007, Soil Survey Geographic (SSURGO) database for Contra Costa County, California, ca013, http://SoilDataMart.nrcs.usda.gov/.

Zone 7 Water Agency; Annual Report for the Ground Water Management Program, Table 4-G; 2008.

FIGURES

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP MONTE VISTA MEMORIAL GARDENS DEVELOPMENT 3656 LAS COLINAS ROAD LIVERMORE, CALIFORNIA	PROJECT NO.: 15426.000.000		FIGURE NO. 1
		SCALE: AS SHOWN		
		DRAWN BY: JV	CHECKED BY: SPC	

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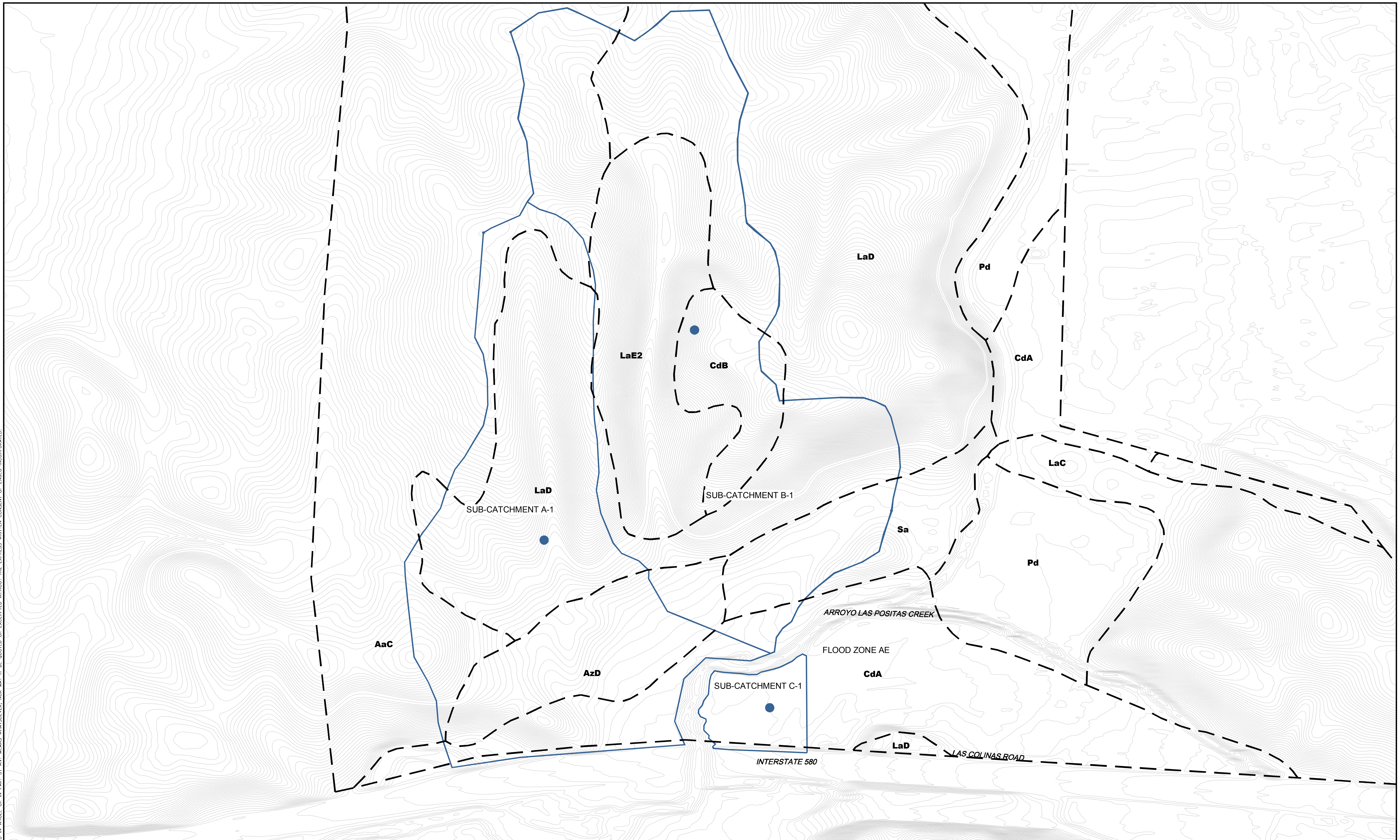

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

WATERSHED BOUNDARY

SOIL LINE

SOIL TYPES

AaC	ALTAMONT CLAY, 3-15% SLOPES, CLASS C SOIL
AzD	AZULE CLAY LOAM, 3-30% SLOPES, MLRA 15, CLASS D SOIL
CdA	CLEAR LAKE CLAY, DRAINED, 0-2% SLOPES, MLRA 14, CLASS D SOIL
CdB	CLEAR LAKE CLAY, DRAINED, 3-7% SLOPES, CLASS C SOIL
LaC	LINNE CLAY LOAM, 3-15% SLOPES, CLASS C SOIL
LaD	LINNE CLAY LOAM, 15-30% SLOPES, MLRA 15. CLASS D SOIL
LaE2	LINNE CLAY LOAM, 30-15% SLOPES, ERODED, CLASS C SOIL
Pd	PESCADERO CLAY LOAM, 0-6% SLOPES, MLRA 14, CLASS C/D SOIL
Sa	SAN YSIDRO LOAM, 0-2%, MLRA 14, CLASS C SOIL

N

0 400 FEET

BASE MAP SOURCE: ACS CONSULTING ENGINEERS AND USDA

ENGEO Expect Excellence

PRE-DEVELOPMENT WATERSHEDS - SOIL MAP
MONTE VISTA MEMORIAL GARDENS DEVELOPMENT
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO.: 15426.000.000
SCALE: AS SHOWN
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FIGURE NO. 2

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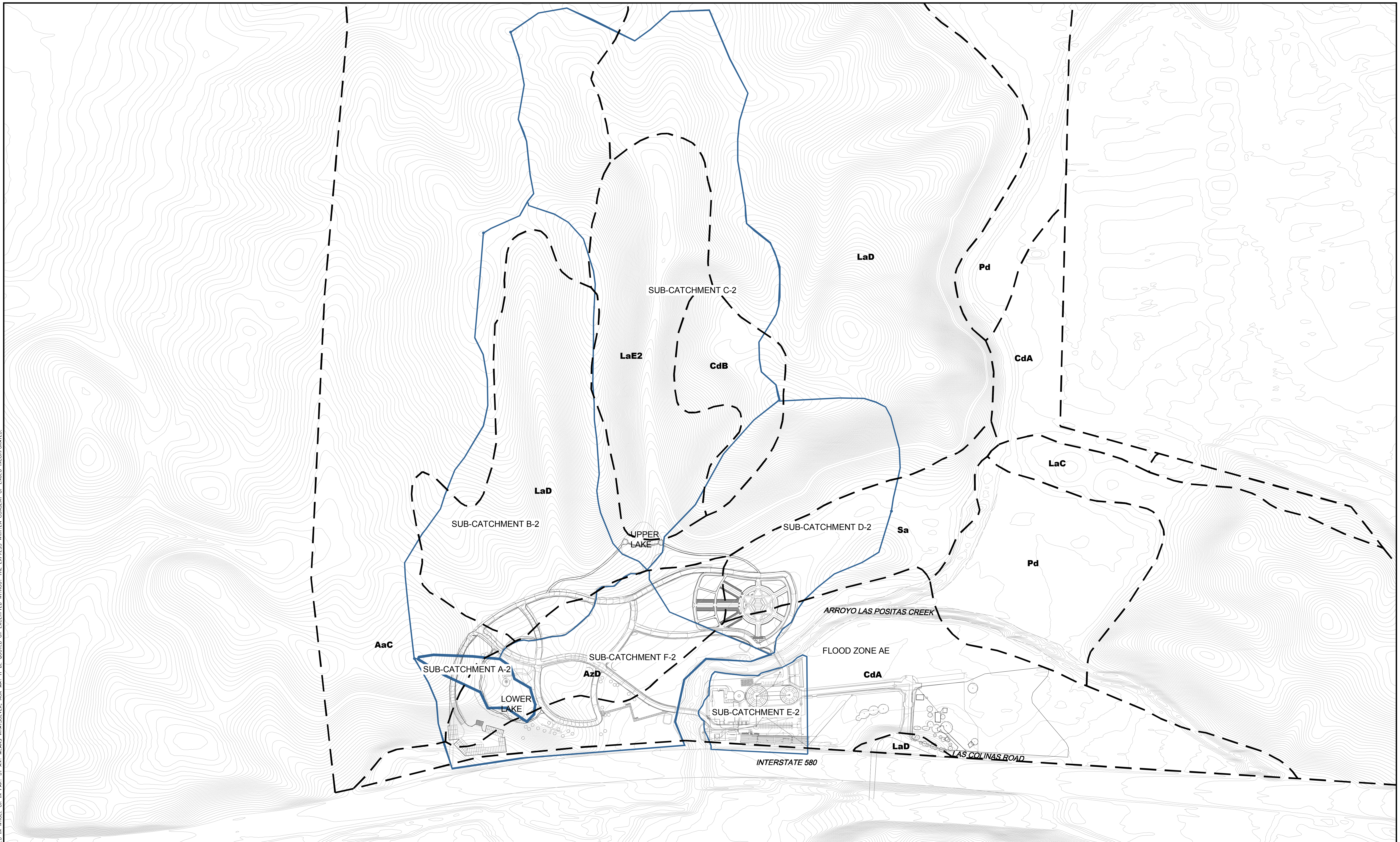

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

WATERSHED BOUNDARY

SOIL LINE

SOIL TYPES

AaC	ALTAMONT CLAY, 3-15% SLOPES, CLASS C SOIL
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Pd	PESCADERO CLAY LOAM, 0-6% SLOPES, MLRA 14, CLASS C/D SOIL
Sa	SAN YSIDRO LOAM, 0-2%, MLRA 14, CLASS C SOIL

N

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FEET

BASE MAP SOURCE: ACS CONSULTING ENGINEERS AND USDA

ENGEO
Expect Excellence

POST-DEVELOPMENT WATERSHEDS - SOIL MAP
MONTE VISTA MEMORIAL GARDENS DEVELOPMENT
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO.: 15426.000.000		FIGURE NO. 3
SCALE: AS SHOWN		
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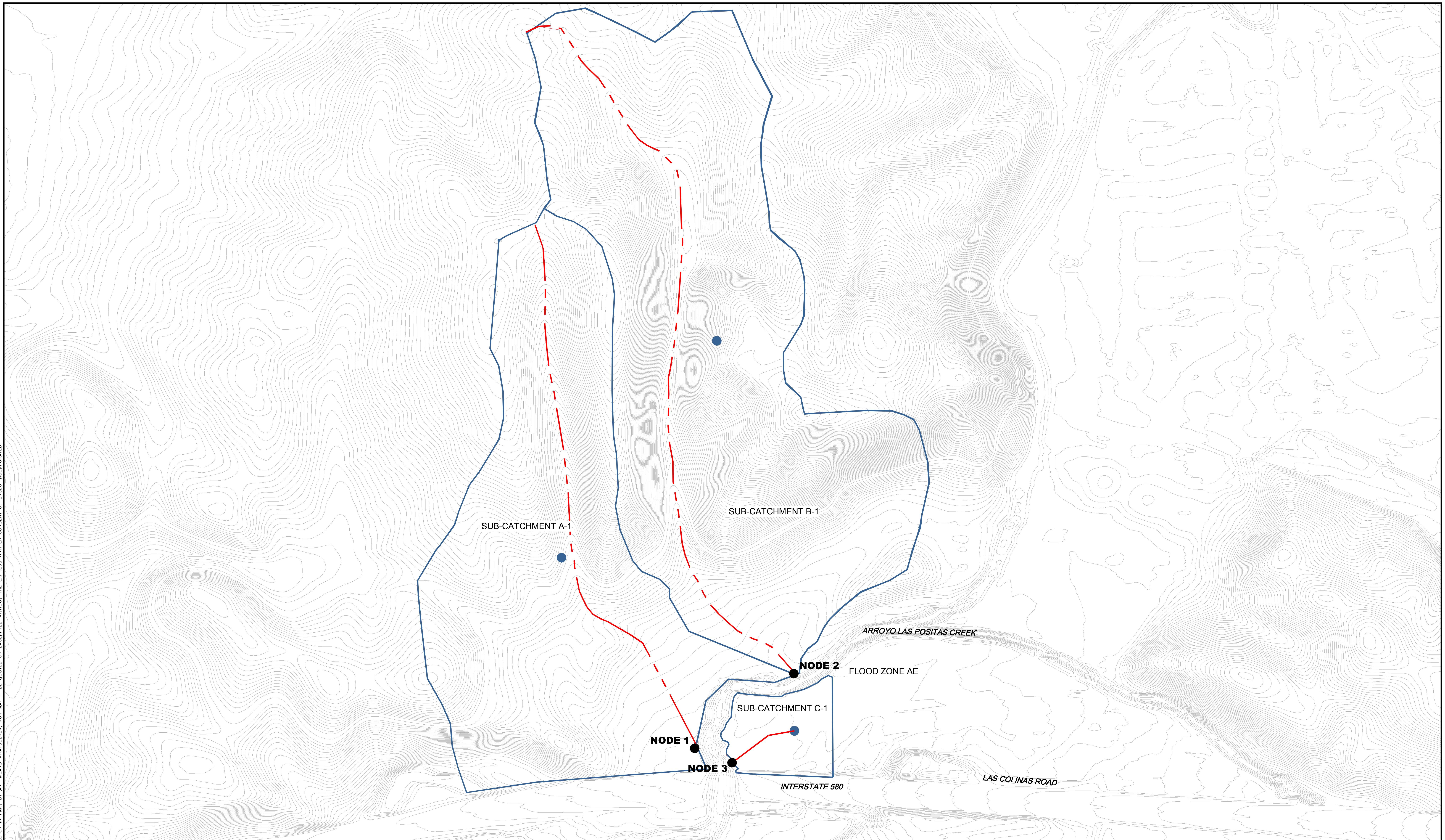
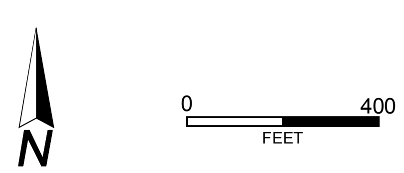


FOR PRE-DEVELOPMENT CONDITIONS

Subcatchment	Area (Square Miles)	Initial Loss (inches)	Uniform Loss Rate (inches)	Longest Flow Path (ft)	Channel Length from Centroid (ft)	Average Stream Slope (ft/mile)	Weighted Roughness Factor (N)	% Impervious	Distance Factor	Basin Lag Time (hr)	Basin Peaking Factor
A-1	0.125	1	0.140	3194	1273	192	0.07	0	21.79	0.27	0.6
B-1	0.174	1	0.050	4553	2268	124	0.07	0	24.58	0.47	0.6
C-1	0.006	1	0.058	655	365	13	0.07	21	16.57	0.12	0.6



EXPLANATION
ALL LOCATIONS ARE APPROXIMATE

- WATERSHED BOUNDARY
- LONGEST DRAINAGE PATH
- CENTROID
- NODE

BASE MAP SOURCE: ACS CONSULTING ENGINEERS

ENGEO Expect Excellence

PRE-DEVELOPMENT WATERSHEDS
MONTE VISTA MEMORIAL GARDENS DEVELOPMENT
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO.: 15426.000.000
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FIGURE NO. 4

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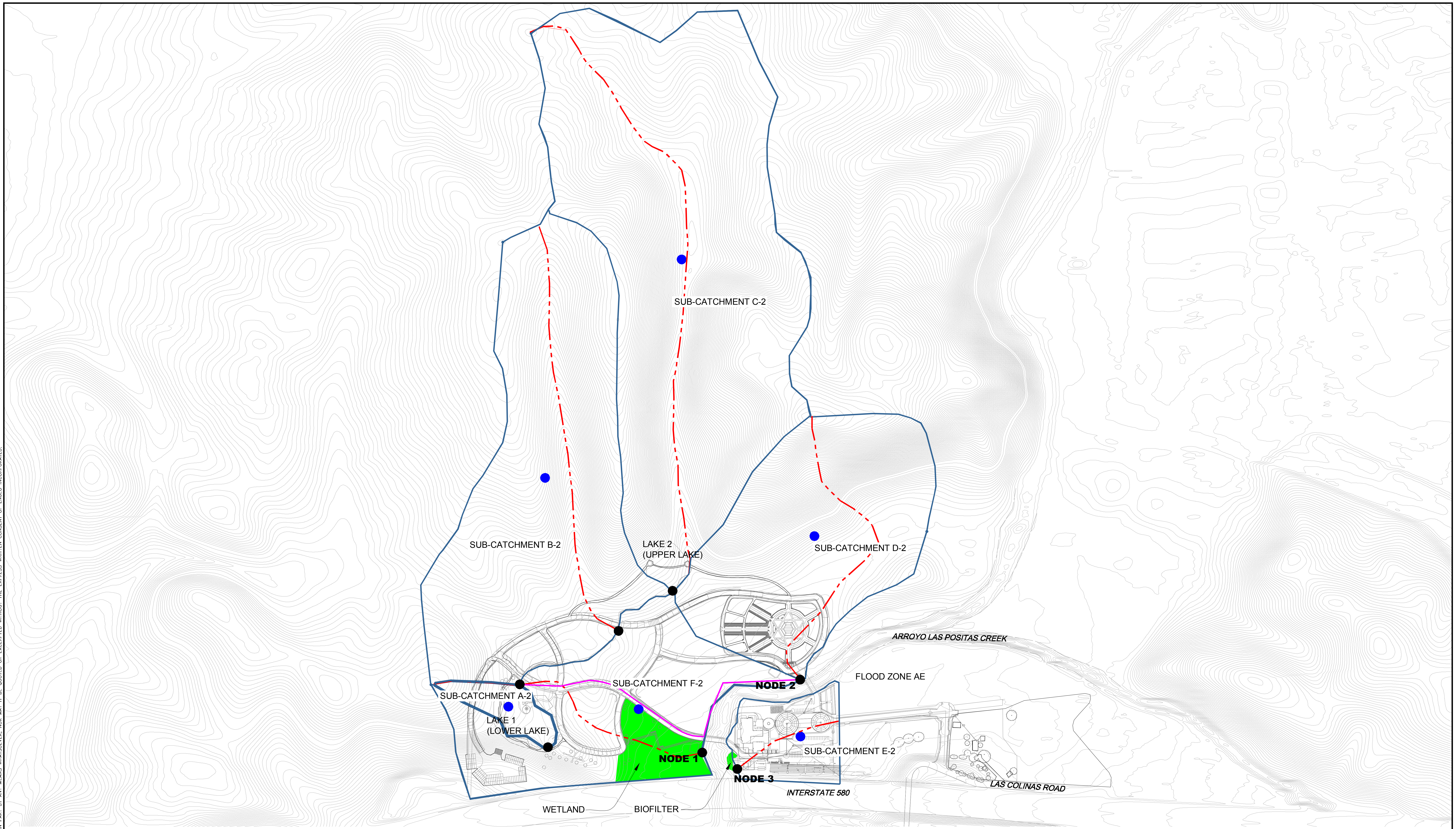

FOR POST-DEVELOPMENT CONDITIONS

Subcatchment	Area (Square Miles)	Initial Loss (inches)	Uniform Loss Rate (inches)	Longest Flow Path (ft)	Channel Length from Centroid (ft)	Average Stream Slope (ft/mile)	Weighted Roughness Factor (N)	% Impervious	Distance Factor	Basin Lag Time (hr)	Basin Peaking Factor
A-2	0.005	1	0.052	470	540	655	0.05	6.20	16	0.040	0.696
B-2	0.076	1	0.051	2547	1079	197	0.05	2.34	20	0.155	0.600
C-2	0.121	1	0.050	3730	1978	112	0.05	0.10	23	0.282	0.600
D-2	0.053	1	0.055	1911	1087	548	0.05	11.95	19	0.108	0.680
E-2	0.012	1	0.070	1355	937	13	0.04	85.00	18	0.154	0.600
F-2	0.045	1	0.056	1245	788	119	0.05	15.97	18	0.101	0.600
G-2	0.008	1	0.053	620	362	84	0.05	8.12	16	0.056	0.600

N

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FEET

10-YEAR PEAK FLOW COMPARISON OF PRE- AND POST- DEVELOPMENT

Node	1	2	3
Pre-Development Discharge (cfs)	56.2	62.5	2.9
Post-Development Discharge (cfs)	48.8	N/A	2.9
Pre-Development Volume Runoff (ac-ft)	7.2	7.7	0.5
Post-Development Volume Runoff (ac-	18.4	N/A (0)	0.6

100-YEAR PEAK FLOW COMPARISON OF PRE- AND POST- DEVELOPMENT

Node	1	2	3
Pre-Development Discharge (cfs)	98.5	114.6	5
Post-Development Discharge (cfs)	222.9	N/A	5
Pre-Development Volume Runoff (ac-ft)	18.8	20.9	1.1
Post-Development Volume Runoff (ac-	45	N/A (0)	1.2

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

WATERSHED BOUNDARY
LONGEST DRAINAGE PATH
CENTROID
NODE
WETLANDS/WATERQUALITY BMP
SUB-DRAIN DISCHARGE PUMP TO LOWER LAKE

BASE MAP SOURCE: ACS CONSULTING ENGINEERS

ENGEO Expect Excellence	POST-DEVELOPMENT WATERSHEDS MONTE VISTA MEMORIAL GARDENS DEVELOPMENT 3656 LAS COLINAS ROAD LIVERMORE, CALIFORNIA	PROJECT NO.: 15426.000.000 SCALE: AS SHOWN DRAWN BY: JV CHECKED BY: SPC	FIGURE NO. 5

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APPENDIX A

HYDROLOGIC CALCULATIONS

TABLE A.1 – Temporal Rainfall Distribution (24-hour, 10-year)

TIME (HR)	RAINFALL FRACTION	CUMULATIVE RAINFALL (INCHES)	TIME (HR)	RAINFALL FRACTION	CUMULATIVE RAINFALL (INCHES)
0.25	0.00	0.01	12.25	0.49	1.42
0.50	0.01	0.03	12.50	0.56	1.63
0.75	0.01	0.04	12.75	0.62	1.80
1.00	0.02	0.05	13.00	0.65	1.90
1.25	0.02	0.07	13.25	0.68	1.97
1.50	0.03	0.09	13.50	0.69	2.01
1.75	0.04	0.11	13.75	0.71	2.06
2.00	0.04	0.11	14.00	0.72	2.10
2.25	0.04	0.12	14.25	0.73	2.13
2.50	0.05	0.13	14.50	0.74	2.17
2.75	0.05	0.16	14.75	0.75	2.20
3.00	0.06	0.17	15.00	0.77	2.23
3.25	0.07	0.19	15.25	0.78	2.26
3.50	0.07	0.19	15.50	0.79	2.29
3.75	0.07	0.21	15.75	0.80	2.32
4.00	0.07	0.21	16.00	0.80	2.34
4.25	0.08	0.24	16.25	0.81	2.37
4.50	0.09	0.26	16.50	0.82	2.39
4.75	0.10	0.28	16.75	0.83	2.42
5.00	0.10	0.30	17.00	0.84	2.44
5.25	0.11	0.31	17.25	0.85	2.46
5.50	0.11	0.33	17.50	0.85	2.48
5.75	0.12	0.35	17.75	0.86	2.50
6.00	0.13	0.37	18.00	0.87	2.53
6.25	0.13	0.38	18.25	0.88	2.55
6.50	0.14	0.40	18.50	0.88	2.57
6.75	0.15	0.42	18.75	0.89	2.58
7.00	0.15	0.44	19.00	0.89	2.60
7.25	0.16	0.47	19.25	0.90	2.62
7.50	0.17	0.49	19.50	0.91	2.64
7.75	0.18	0.51	19.75	0.91	2.65
8.00	0.18	0.53	20.00	0.92	2.67
8.25	0.19	0.55	20.25	0.92	2.69
8.50	0.20	0.58	20.50	0.93	2.71
8.75	0.21	0.61	20.75	0.94	2.73
9.00	0.22	0.63	21.00	0.94	2.74
9.25	0.23	0.66	21.25	0.95	2.76
9.50	0.24	0.69	21.50	0.95	2.78
9.75	0.25	0.72	21.75	0.96	2.79
10.00	0.26	0.75	22.00	0.96	2.80
10.25	0.27	0.79	22.25	0.97	2.82
10.50	0.28	0.82	22.50	0.98	2.85
10.75	0.30	0.86	22.75	0.98	2.84
11.00	0.31	0.90	23.00	0.98	2.86
11.25	0.33	0.95	23.25	0.99	2.87
11.50	0.35	1.01	23.50	0.99	2.88
11.75	0.37	1.09	23.75	1.00	2.90
12.00	0.42	1.23	24.00	1.00	2.91

TABLE A.2 – Temporal Rainfall Distribution (24-hour, 100-year)

TIME (HR)	RAINFALL FRACTION	CUMULATIVE RAINFALL (INCHES)	TIME (HR)	RAINFALL FRACTION	CUMULATIVE RAINFALL (INCHES)
0.25	0.00	0.02	12.25	0.49	2.39
0.50	0.01	0.04	12.50	0.56	2.74
0.75	0.01	0.07	12.75	0.62	3.04
1.00	0.02	0.09	13.00	0.65	3.20
1.25	0.02	0.11	13.25	0.68	3.31
1.50	0.03	0.16	13.50	0.69	3.39
1.75	0.04	0.18	13.75	0.71	3.47
2.00	0.04	0.18	14.00	0.72	3.53
2.25	0.04	0.20	14.25	0.73	3.59
2.50	0.05	0.22	14.50	0.74	3.65
2.75	0.05	0.27	14.75	0.75	3.70
3.00	0.06	0.29	15.00	0.77	3.75
3.25	0.07	0.32	15.25	0.78	3.80
3.50	0.07	0.32	15.50	0.79	3.85
3.75	0.07	0.35	15.75	0.80	3.90
4.00	0.07	0.35	16.00	0.80	3.94
4.25	0.08	0.41	16.25	0.81	3.99
4.50	0.09	0.44	16.50	0.82	4.03
4.75	0.10	0.47	16.75	0.83	4.07
5.00	0.10	0.50	17.00	0.84	4.11
5.25	0.11	0.53	17.25	0.85	4.14
5.50	0.11	0.56	17.50	0.85	4.18
5.75	0.12	0.59	17.75	0.86	4.22
6.00	0.13	0.62	18.00	0.87	4.26
6.25	0.13	0.64	18.25	0.88	4.29
6.50	0.14	0.67	18.50	0.88	4.32
6.75	0.15	0.71	18.75	0.89	4.35
7.00	0.15	0.75	19.00	0.89	4.38
7.25	0.16	0.79	19.25	0.90	4.41
7.50	0.17	0.82	19.50	0.91	4.44
7.75	0.18	0.86	19.75	0.91	4.47
8.00	0.18	0.90	20.00	0.92	4.50
8.25	0.19	0.93	20.25	0.92	4.53
8.50	0.20	0.98	20.50	0.93	4.56
8.75	0.21	1.02	20.75	0.94	4.59
9.00	0.22	1.07	21.00	0.94	4.62
9.25	0.23	1.11	21.25	0.95	4.65
9.50	0.24	1.16	21.50	0.95	4.68
9.75	0.25	1.22	21.75	0.96	4.70

TIME (HR)	RAINFALL FRACTION	CUMULATIVE RAINFALL (INCHES)	TIME (HR)	RAINFALL FRACTION	CUMULATIVE RAINFALL (INCHES)
10.00	0.26	1.27	22.00	0.96	4.72
10.25	0.27	1.32	22.25	0.97	4.74
10.50	0.28	1.38	22.50	0.98	4.79
10.75	0.30	1.45	22.75	0.98	4.79
11.00	0.31	1.52	23.00	0.98	4.81
11.25	0.33	1.60	23.25	0.99	4.83
11.50	0.35	1.70	23.50	0.99	4.86
11.75	0.37	1.83	23.75	1.00	4.88
12.00	0.42	2.06	24.00	1.00	4.90

TABLE A.3 –Initial Losses (Alameda County Hydrology and Hydraulic Manual)

TABLE 5 INITIAL LOSS	
Design Storm *(hr)*	Initial Loss *(inches)*
6	0.8
24	1.0

(District 1994)

TABLE A.4 –Uniform Loss Rates by Soil Group Type (Alameda County Hydrology and Hydraulic Manual)

TABLE 6 UNIFORM LOSS RATES			
Hydrologic Soil Group	Rural Coverage	New Urban Coverage	Existing Urban Coverage
A	0.45	0.45	0.45
B	0.35	0.37	0.40
C	0.14	0.19	0.25
D	0.05	0.07	0.09

All loss rates in inches/hour. (District 1994)

TABLE A.5 –Basin Roughness Factors (Alameda County Hydrology and Hydraulic Manual)

TABLE 8 BASIN ROUGHNESS FACTORS FOR RURAL WATERSHEDS	
Basin Type	Basin Roughness Factor (*N*)
1. Rural watersheds with generally clear stream bed and minimal vegetation growth in the drainage reaches.	0.05
2. Rural watersheds with moderate to high levels of vegetation growth, or rock and boulder deposits within the main drainage reaches.	0.07
3. Rural watersheds with dense vegetation or high levels of boulder deposits within the main drainage reaches.	0.08

(District 2015)

TABLE A.6 – Pre-Development Soil Type Uniform Loss Rate Assumptions – Groups C and D

	Uniform Loss Rate for Soil Group D	Uniform Loss Rate for Soil Group C	% of Sub-Basin A-1 Group C	% of Sub-Basin A-1 Group D	% of Sub-Basin B-1 Group C	% of Sub-Basin B-1 Group D	% of Sub-Basin C-1 Group C	% of Sub-Basin C-1 Group D
Rural	0.05	0.14	18.88481249	81.11518751	67.91367389	32.08632611	0	79.073
New Urban	0.07	0.19	0	0	0	0	0	0
Existing Urban Coverage	0.09	0.25	0	0	0	0	0	20.927
Soil Group Reference	(Attachment 9, 2018 ACFCD HH Manual)							

TABLE A.7 – Pre-Development Infiltration Conditions

	Initial Loss (inches)	Uniform Loss Rate (inches) (Ref. Table 6 AC Manual)	Weighted Roughness Factor (N)	N Assumption
Subcatchment				
A-1	1	0.066996331	0.07	>80% is Rural; Use 0.07 Per manual
B-1	1	0.111122307	0.07	>80% is Rural; Use 0.07 Per manual
C-1	1	0.0583708	0.07	>80% is Rural; Use 0.07 Per manual
Reference	Table 5, ACWD Manual (24-hr storm)		Assumed Rural Watershed	

TABLE A.8 – Post-Development Uniform Soil Loss Rate Assumptions – Group D Soils

	Uniform Loss Rate for Soil Group D	% of Sub-Basin A-2	% of Sub-Basin B-2	% of Sub-Basin C-2	% of Sub-Basin D-2	% of Sub-Basin E-2	% of Sub-Basin F-2
Rural	0.05	0	67.83839434	26.19769675	48.79010035	0	0
New Urban	0.07	75.03470886	9.150625749	1.753437604	16.31011207	60	90.046747
Existing Urban Coverage	0.09	0	0	0		0	0
Soil Group Reference	(Attachment 9, 2018 ACFCD HH Manual)						

TABLE A.9 – Post-Development Uniform Soil Loss Rate Assumptions – Group C Soils

	Uniform Loss Rate for Soil Group C	% of Sub-Basin A-2	% of Sub-Basin B-2	% of Sub-Basin C-2	% of Sub-Basin D-2	% of Sub-Basin E-2	% of Sub-Basin F-2
Rural	0.14	0	21.20405984	72.04886565	21.83889359	0	0
New Urban	0.19	25.41692454	1.806920065	0	13.06089399	0	9.9532527
Existing Urban Coverage	0.25	0	0	0		0	0
Soil Group Reference	(Attachment 9, 2018 ACFCD HH Manual)						

TABLE A.10 – Post-Development Infiltration Conditions

	Initial Loss (inches)	Uniform Loss Rate (inches/hr)	Weighted Roughness Factor (N)	N Assumption
Subcatchment				
A-2	1	0.100816453	0.07	>80% is Rural; Use 0.07 Per manual
B-2	1	0.115194667	0.07	>80% is Rural; Use 0.07 Per manual
C-2	1	0.115194667	0.07	>80% is Rural; Use 0.07 Per manual
D-2	1	0.091202278	0.07	>80% is Rural; Use 0.07 Per manual
E-2	1	0.07	0.427136816	Mixed Rural/Urban, weight N obtained from Table 8 and Eqn. 12 of ACWD Manual , Assumed
F-2	1	0.081943903	0.07	>80% is Rural; Use 0.07 Per manual
Reference	Table 5, ACWD Manual (24-hr storm)			

TABLE A.11 – Basin Roughness Factor for Urban Watersheds (Alameda County Hydrology and Hydraulic Manual)

EQUATION 12 BASIN ROUGHNESS FACTOR FOR URBAN WATERSHEDS

$$N = 0.52\,n^{0.79} \tag{12}$$

where:

N = *basin roughness factor*

n = *Manning's roughness coefficient (from **Table 9**)*

(District 2015)

Sub-Basin E-2 is classified as urban given more than 80% of the sub-basin is not classified as rural. The equation above was used to calculate the basin roughness factor, N, using Manning's roughness coefficient. We assumed a Manning's 'n' of 0.014 for a reinforced concrete pipe less than 36 inches in diameter.

TABLE A.12 – Manning's Roughness Coefficient (Alameda County Hydrology and Hydraulic Manual)

TABLE 9 MANNING'S ROUGHNESS COEFFICIENT

Type of Facility	n
Reinforced Concrete Pipe	
Conduit > 36″ diameter	0.012
Conduit ≤ 36″ diameter	0.014
Corrugated Metal Pipe	
Annular	0.021
Helical	0.018
Concrete-Lined Channels	
Smooth-troweled	0.015
District Simulated Stone	0.017
Reinforced Concrete Box	
Cast-in-Place	0.015
Pre-Cast	0.014
Earth Channels	
Smooth Geometric	0.030 – 0.035
Irregular or Natural	0.045 – 0.050

(Chow 1959, District 1989)

This was averaged with the basin roughness factor for the pervious portions of the sub-basin to determine a weighted average N. In accordance with the Alameda County Hydrology and Hydraulic Manual, a weighted N is used for a mixed rural and urban sub-basin when less than 80% rural.

TABLE A.13– Basin Roughness Factor for Urban Watersheds (Alameda County Hydrology and Hydraulic Manual)

EQUATION 13 BASIN LAG TIME

$$t_L = K \cdot N \left(\frac{L \cdot L_c}{\sqrt{S}} \right)^{0.38} \tag{13}$$

where:

t_L = *lag time (hr)*

K = *distance factor*
for $L > 1.7$ *mi,* $K = 24$
for $L \leq 1.7$ *mi,* $K = 15.22 + 2.15L + 8.7/L$
if calculated value of K *is greater than* 40*, use* $K{=}40$

N = *basin roughness factor (from **Table 8**, rural watershed or **Equation 12**, urban watershed)*

L = *length of longest watercourse (mi) (see **Figure 4**)*

L_c = *length along longest watercourse measured from the point of concentration to a point opposite the watershed centroid (see **Figure 4**)*

S = *average stream slope (ft / mi from **Equation 11**)*

(District 1994)

As shown above, the basin lag time for each sub-basin in the pre- and post- development model was calculated using the parameters shown in Sections 3.4 and 3.5 of the report.

TABLE A.14– Basin Peaking Factor (Alameda County Hydrology and Hydraulic Manual)

EQUATION 14 BASIN PEAKING FACTOR

$$C_p = 0.6e^{0.06\left(S_o/A\right)} \tag{14}$$

where:

C_p = *basin peaking factor* $(C_p \leq 0.85)$
S_o = *average watershed slope (percent from* ***Attachment 10****)*
For $S_o \leq 5\%,\ C_p = 0.6$
A = *drainage area* $(mi^2$*; if* $A < 5mi^2$*, use* $A = 5mi^2)$

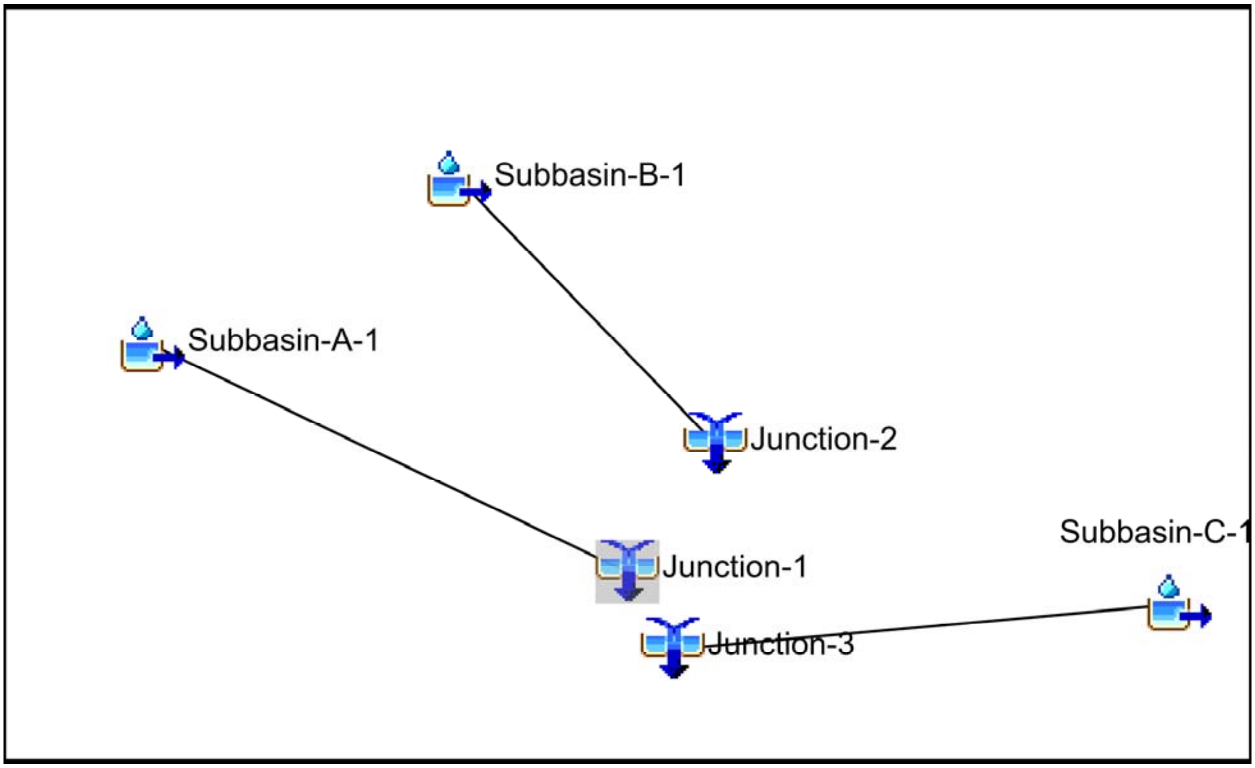
(District 1994)

The above equation was used to determine the basin peaking factor for each sub-basin of the pre- and post- development model.

APPENDIX B

HEC-HMS Output Tables

Pre-Development Basin Model


Subbasin-B-1
Subbasin-A-1
Junction-2
Subbasin-C-1
Junction-1
Junction-3

Pre-Development Model 10-Year Results

Project: Monte Vista Simulation Run: Run 1

Start of Run: 01Jan2000, 00:00 Basin Model

End of Run: 02Jan2000, 00:00 Meteorologi

Compute Time: 22Oct2019, 09:31:01 Control Spe

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
Subbasin-A-1	0.12	56.2	01Jan2000, 12:45	7.2
Subbasin-B-1	0.17	62.5	01Jan2000, 13:00	7.7
Junction-2	0.17	62.5	01Jan2000, 13:00	7.7
Junction-1	0.12	56.2	01Jan2000, 12:45	7.2
Subbasin-C-1	0.01	2.9	01Jan2000, 12:30	0.5
Junction-3	0.01	2.9	01Jan2000, 12:30	0.5

Pre-Development Model 100-Year Results

Project: Monte Vista Simulation Run: Run 1

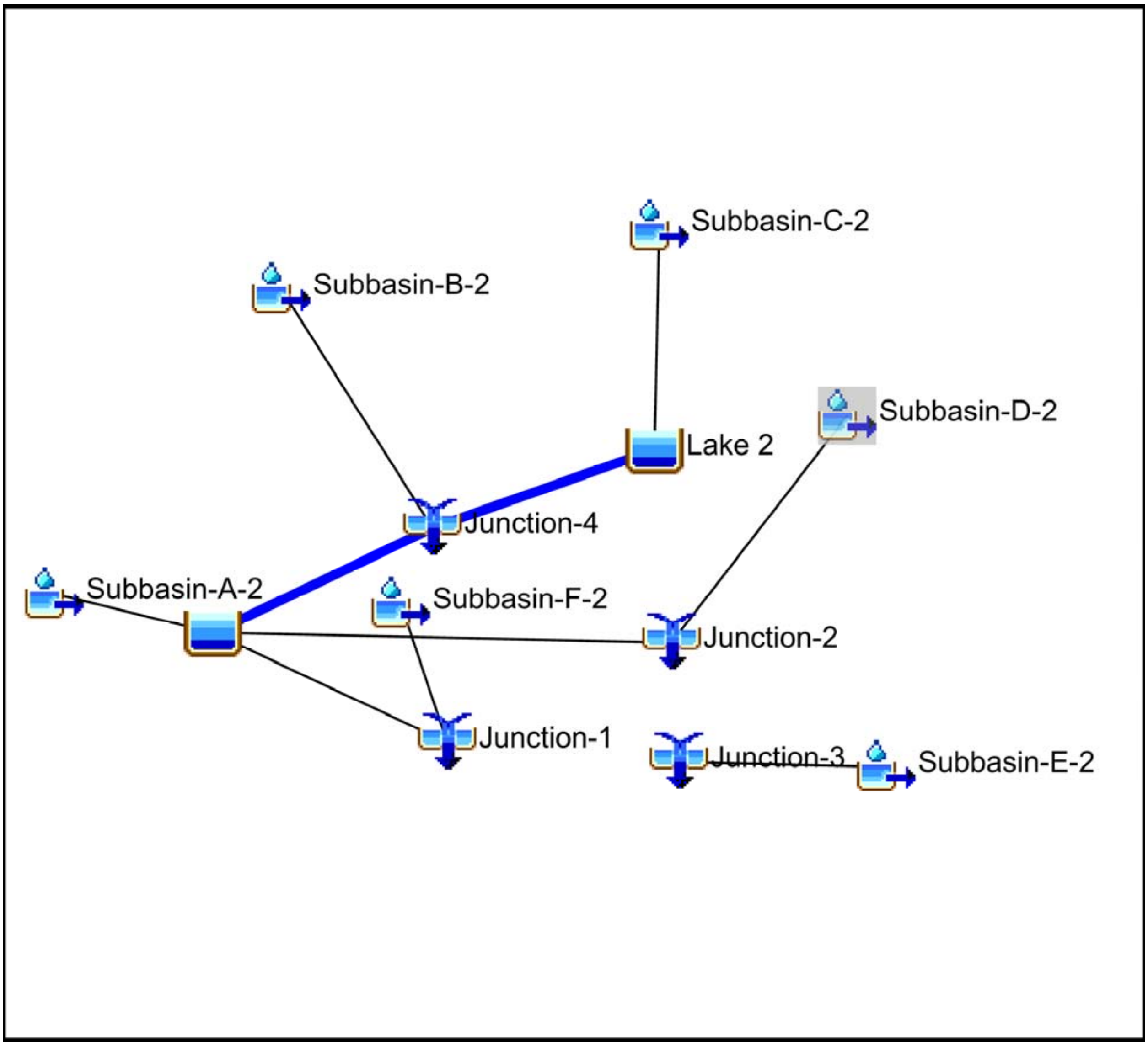
Start of Run: 01Jan2000, 00:00 Basin Model

End of Run: 02Jan2000, 00:00 Meteorologi

Compute Time: 22Oct2019, 09:44:54 Control Spe

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
Subbasin-A-1	0.1247600	98.5	01Jan2000, 12:45	18.9
Subbasin-B-1	0.1737700	114.6	01Jan2000, 13:00	21.0
Junction-2	0.1737700	114.6	01Jan2000, 13:00	21.0
Junction-1	0.1247600	98.5	01Jan2000, 12:45	18.9
Subbasin-C-1	0.0059155	5.0	01Jan2000, 12:30	1.1
Junction-3	0.0059155	5.0	01Jan2000, 12:30	1.1

Post-Development Basin Model


Subbasin-C-2
Subbasin-B-2
Subbasin-D-2
Lake 2
Junction-4
Subbasin-A-2
Subbasin-F-2
Junction-2
Junction-1
Junction-3
Subbasin-E-2

Post-Development Model 10-Year Results

Project: Monte Vista Simulation Run: Run 1

Start of Run: 01Jan2000, 00:00 Basin Model

End of Run: 02Jan2000, 00:00 Meteorologi

Compute Time: 22Oct2019, 09:49:45 Control Spe

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
Junction-2	0.05	24.6	01Jan2000, 12:30	3.3
Junction-4	0.20	82.5	01Jan2000, 12:45	11.9
Reach-2	0.20	81.6	01Jan2000, 12:45	11.9
Lake 2	0.12	49.6	01Jan2000, 12:45	8.3
Subbasin-D-2	0.05	24.6	01Jan2000, 12:30	3.3
Subbasin-C-2	0.12	49.6	01Jan2000, 12:45	8.3
Reach-1	0.12	48.8	01Jan2000, 12:45	8.3
Subbasin-B-2	0.08	34.4	01Jan2000, 12:30	3.7
Subbasin-A-2	0.00	2.1	01Jan2000, 12:30	0.2
Lake 1	0.26	107.0	01Jan2000, 12:45	15.5
Subbasin-F-2	0.04	19.4	01Jan2000, 12:30	2.9
Junction-1	0.30	125.4	01Jan2000, 12:45	18.4
Subbasin-E-2	0.01	2.9	01Jan2000, 12:30	0.6
Junction-3	0.01	2.9	01Jan2000, 12:30	0.6

Post-Development Model 100-Year Results

Project: Monte Vista Simulation Run: Run 1

Start of Run: 01Jan2000, 00:00 Basin Model

End of Run: 02Jan2000, 00:00 Meteorologic

Compute Time: 22Oct2019, 09:24:39 Control Spe

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
Junction-2	0.0528344	43.3	01Jan2000, 12:30	7.9
Junction-4	0.2000452	147.1	01Jan2000, 12:45	29.8
Reach-2	0.2000452	145.9	01Jan2000, 12:45	29.8
Lake 2	0.1205300	87.4	01Jan2000, 12:45	20.1
Subbasin-D-2	0.0528344	43.3	01Jan2000, 12:30	7.9
Subbasin-C-2	0.1205300	87.4	01Jan2000, 12:45	20.1
Reach-1	0.1205300	86.2	01Jan2000, 12:45	20.1
Subbasin-B-2	0.0795152	61.8	01Jan2000, 12:30	9.7
Subbasin-A-2	0.0045724	3.7	01Jan2000, 12:30	0.6
Lake 1	0.2574520	190.7	01Jan2000, 12:45	38.3
Subbasin-F-2	0.0408717	33.8	01Jan2000, 12:30	6.7
Junction-1	0.2983237	222.9	01Jan2000, 12:45	45.0
Subbasin-E-2	0.0059155	5.0	01Jan2000, 12:30	1.2
Junction-3	0.0059155	5.0	01Jan2000, 12:30	1.2

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