

# Hydrologic Basis of Design Report for the Ellis Basin 3A Stormwater Facility, City of Tracy

Prepared for:

**The Surland Companies**

June 2022

Prepared by:



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June 17, 2022

**A REPORT PREPARED FOR:**

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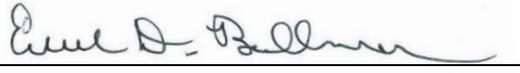
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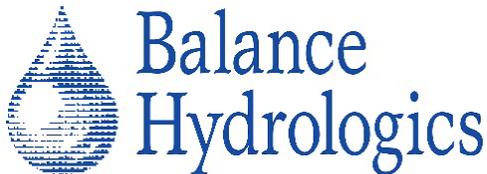
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## TABLE OF CONTENTS

<b>1</b>	<b>INTRODUCTION</b>	<b>1</b>
1.1	<i>Overview and Purpose</i>	1
1.2	<i>Past Studies and Current References</i>	1
<b>2</b>	<b>HYDROLOGIC SETTING</b>	<b>3</b>
2.1	<i>Geographic Location</i>	3
2.2	<i>Land Use</i>	4
2.2.1	Historical and Existing Land Use	4
2.2.2	Proposed Land Use	4
2.3	<i>Climate</i>	5
2.4	<i>Topography and Soils</i>	6
2.5	<i>Drainage Features and Patterns</i>	8
2.5.1	Pre-development Drainage and Runoff	8
2.5.2	Proposed Drainage System	9
2.6	<i>FEMA Floodplain Information</i>	11
<b>3</b>	<b>BASIN 3A CONFIGURATION AND STORAGE VOLUME</b>	<b>12</b>
3.1	<i>Location and Footprint</i>	12
3.2	<i>Basin 3A Characteristics</i>	12
3.3	<i>Storage Capacity</i>	13
<b>4</b>	<b>PERCOLATION TESTING AND DESIGN INFILTRATION CAPACITY</b>	<b>15</b>
4.1	<i>Field Test Borings</i>	15
4.2	<i>Percolation Test Results</i>	16
4.3	<i>Calculated Infiltration Rates per Field Data</i>	18
4.4	<i>Design Safety Factor</i>	19
<b>5</b>	<b>HYDROLOGIC MODELING METHODOLOGY AND INPUT</b>	<b>21</b>
5.1	<i>Hydrologic Modeling Methodology</i>	21
5.2	<i>HEC-HMS Input Parameters</i>	22
5.2.1	Sub-watershed Areas	22
5.2.2	Soil Moisture Accounting Parameters	24
5.2.3	Design Storm Characteristics	25
5.2.4	Rainfall-Runoff Hydrograph and Lag Times	26
5.2.5	Control Specifications	26
5.2.6	Inflow from South of Linne	26

HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A  
STORMWATER FACILITY, CITY OF TRACY

<b>6</b>	<b>MODEL OUTPUT AND PREDICTED BASIN PERFORMANCE</b>	<b>27</b>
6.1	<i>Runoff Rates and Volume</i>	27
6.2	<i>Maximum Water Surface Elevation and Storage</i>	28
6.3	<i>Facility Freeboard</i>	28
6.4	<i>Drawdown and Storage Recovery</i>	28
6.5	<i>Basin Performance per Water Quality Management Criteria</i>	29
<b>7</b>	<b>LIMITATIONS</b>	<b>30</b>

## LIST OF TABLES

Table 2-1	Average representative precipitation and evapotranspiration.	6
Table 2-2	Study area soil types and key characteristics.	8
Table 3-1	Elevation-storage relationships for the Ellis Basin 3A Main Basin.	14
Table 4-1	US EPA falling head test results and Porchet conversions.	17
Table 5-1	Summary of land use type hydrologic parameters	24
Table 5-2	Composite surface soil hydrologic parameters by sub-watershed	24

## LIST OF FIGURES

Figure 2-1	Study area location map, Ellis Basin 3A watershed.	3
Figure 2-2	Planned near- and mid-term development in the study watershed.	5
Figure 2-3	Study watershed soils map.	7
Figure 2-4	Regional drainage infrastructure as proposed in the Citywide Storm Drainage Master Plan Update.	11
Figure 3-1	Basin 3A Plan.	13
Figure 4-1	General locations of boring pairs from the January 2022 field tests.	16
Figure 4-2	Porchet Method	18
Figure 4-3	Factor of Safety and Design Infiltration Rate Worksheet.	20
Figure 5-1	Basin 3A sub-watersheds used in the hydrologic modeling.	23
Figure 5-2	Rainfall distribution for the 100-year, 10-day design storm.	25
Figure 6-1	Modeled cumulative runoff to Basin 3A.	27
Figure 6-2	Modeled water surface elevation in Basin 3A.	28

## APPENDICES

Appendix A	NOAA Precipitation Data
Appendix B	HEC-HMS Model Input and Output

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## 1 INTRODUCTION

### 1.1 Overview and Purpose

This report presents pertinent background information and hydrologic modeling that forms the design basis for the Ellis Basin 3A stormwater facility in the City of Tracy, San Joaquin County, California. Basin 3A will be a major infrastructure element sized to handle the runoff from a roughly 700-acre urbanizing watershed in the southwest portions of the City. Portions of the watershed have already been developed (i.e., Ellis Project and several church properties) and are currently relying on interim infiltration basins to handle stormwater runoff. Construction of Basin 3A will provide a new terminal facility to serve the area envisioned in City storm drainage master planning documents, thus allowing the interim basins to be converted to other uses. Sized to infiltrate all runoff up to and including that from a 100-year, 10-day storm event the basin will provide stormwater quantity and quality management for the watershed south of Valpico Road and west of Corral Hollow Road, serve to recharge the local aquifer, and obviate the need to provide a connection to the capacity-limited downstream Westside Channel watershed

The basis of design for Basin 3A has progressed from preliminary hydrologic modeling used to estimate storage needs to a more robust analysis that incorporates information from a detailed field investigation on infiltration capacity, and the latest available data on existing and future urbanization. Importantly, the final modeling work explicitly incorporates the updated design guidelines set forth in the Citywide Storm Drainage Master Plan Update (CSDMP) prepared by Wood Rodgers in September of 2021. The modeling described herein was used to directly inform the final design of the facility as prepared by Carlson, Barbee & Gibson (CBG), lead engineers for the project.

### 1.2 Past Studies and Current References

The City of Tracy has a long history of comprehensive stormwater management planning that has been foundational to the work presented in this report. Examples of pertinent past studies include:

- City of Tracy Design Standards, Section 5, Storm Drainage Design Standards, 2008;
- Ellis Program Sub-basin Final Storm Drainage Technical Report, prepared by Storm Water Consulting and Stantec, January 2011;

## HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A STORMWATER FACILITY, CITY OF TRACY

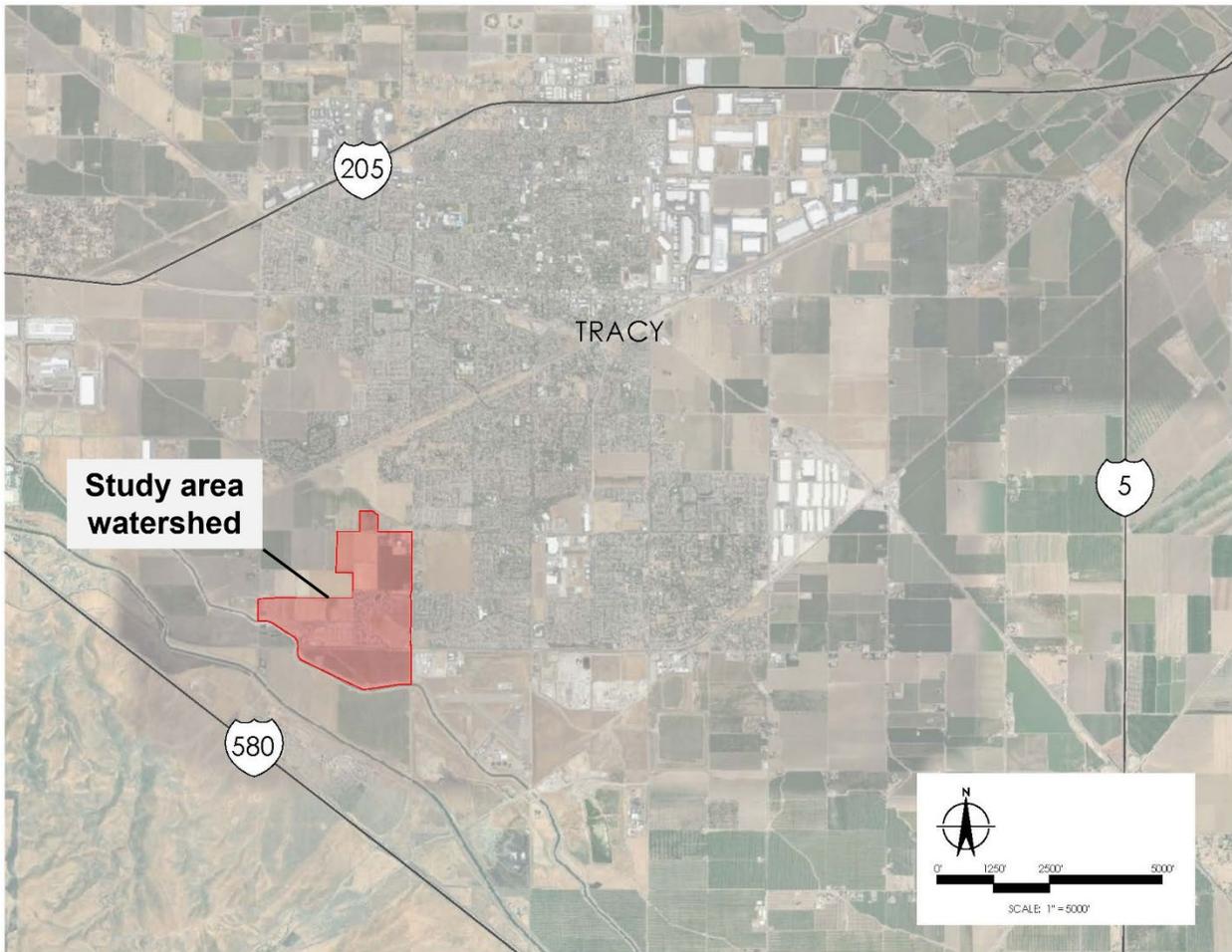
- City of Tracy Citywide Storm Drainage Master Plan, prepared by Stantec, November 2012;
- Ellis Program Sub-basin Reevaluation of Program Infrastructure, technical memorandum prepared by Storm Water Consulting, August 2015
- Project Stormwater Plan for Ellis and Avenues, Tracy, California, prepared by Carlson, Barbee & Gibson, August 2016;
- Citywide Storm Drainage Master Plan Update, prepared by Wood Rodgers, March 2020; and
- Geotechnical Engineering Report Ellis Avenues Detention Basins, prepared by Wallace Kuhl and Associates, March 2022.

The authors would particularly like to recognize the important technical input and feedback provided by staff at CBG and at Wallace Kuhl and Associates throughout the planning and design process. Additional important supplemental information and feedback was provided by staff at Wood Rodgers as the basin design progressed.

## 2 HYDROLOGIC SETTING

### 2.1 Geographic Location

This study encompasses an approximately 704-acre (1.1 square mile) watershed of urbanizing lands in the southwestern portion of the City of Tracy in San Joaquin County, California (**Figure 2-1**). The area included in the Ellis Basin 3A model is an approximately 542-acre (0.85 square mile) portion of land north of the ACE railroad track. The additional 162-acre (0.25 square mile) area south of the railroad tracks will be managed by a separate detention basin that will discharge via pipe to Basin 3A<sup>1</sup>.



**Figure 2-1 Study area location map, Ellis Basin 3A watershed.**

<sup>1</sup> The area is referred to as the South of Linne watershed (see Section 2.5.2 for further explanation). The runoff and basin routing from this area is accounted for by adding a 1 cfs of constant inflow to Ellis Basin 3A during the model runs; this equates to the maximum allowable discharge from the basin as prescribed in the CSDMP.

The watershed is roughly bounded by Corral Hollow Road on the east, the Delta Mendota Canal on the south, South Lammers Road and agricultural lands on the west, and Valpico Road on the north. The site location of Basin 3A is immediately north of Valpico Road and south of the West Side Irrigation District Upper Main Canal.

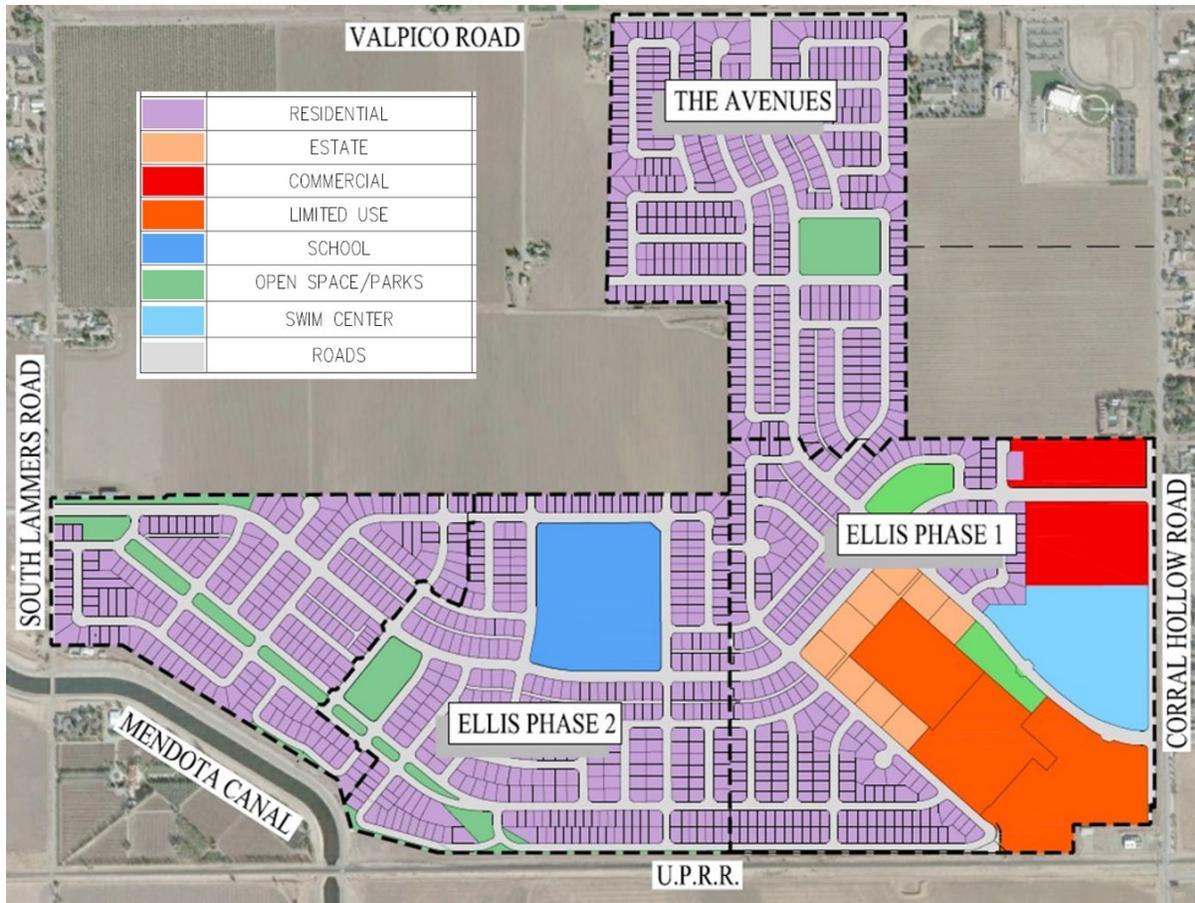
## 2.2 Land Use

### 2.2.1 HISTORICAL AND EXISTING LAND USE

The entire watershed area was formerly used for agricultural purposes prior to 2015 apart from several church properties located to the south and west of the intersection of Valpico Road and Corral Hollow Road. Beginning in 2015, residential development began as part of the Ellis Specific Plan Area to the west of Corral Hollow Road in the southern half of the watershed and now extends west to South Lammers Road.

### 2.2.2 PROPOSED LAND USE

Specific and general plan documents as well as the CSDMP anticipate the eventual full urbanization of the watershed. Over the near to mid-term this will include completion of the Ellis and Avenues Projects consisting largely of residential neighborhoods but also including commercial areas, a school, aquatic center, and parks as shown in **Figure 2-2**.



**Figure 2-2** Planned near- and mid-term development in the study watershed. Data sourced from Carlson, Barbee & Gibson.

### 2.3 Climate

Climate at the site can be characterized as Mediterranean, with cool, somewhat wet winters and hot, dry summers typical of the transition zone from the coastal mountains to the San Joaquin Valley. That said, located on the west side of the San Joaquin Valley, Tracy is characterized by a low mean annual rainfall on the order of 13 inches as summarized in **Table 2-1** for the nearby Tracy Carbona site. Annual rainfall also reflects both wet and dry periods associated with the El Niño Southern Oscillation; as an example, the maximum annual rainfall measured at the same site was 21.1 inches in 1983 and the minimum was only 4.8 inches in 1987.

Average monthly reference evapotranspiration (ET) is also shown in **Table 2-1**. Reference ET is typically high in the warm summer months; precipitation typically only exceeds ET in the winter months of December, January, and February.

**Table 2-1 Average representative precipitation and evapotranspiration.** Rainfall data for the period 1981 to 2010 at National Weather Service Tracy Carbona Station (COOP 048999). ET per Zone 15 as published by the Irrigation Research and Training Center, CalPoly.

<u>Month</u>	<u>Rainfall (inches)</u>	<u>ET (inches)</u>
October	0.76	4.04
November	1.33	1.58
December	2.12	1.09
January	2.55	0.95
February	2.53	2.30
March	1.81	4.38
April	0.86	6.30
May	0.67	8.18
June	0.12	8.18
July	0.00	8.35
August	0.04	7.33
September	0.26	5.72
<b>Total</b>	<b>13.05</b>	<b>58.40</b>
	<b>Average D-J-F</b>	<b>1.45</b>

## 2.4 Topography and Soils

The topography within the study watershed gently slopes to the northeast without any prominent features. The highest elevations of approximately 197 feet are found to the south, adjacent to the Delta Mendota Canal Project. From the southwest the land descends uniformly to a low point of 108.3 feet in the north, adjacent to the West Side Irrigation District Upper Main Canal. On average, the ground slope is just over one percent.

Surface soils within the watershed play an important role in framing the potential for stormwater runoff and are distributed as shown in **Figure 2-3**.

HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A  
STORMWATER FACILITY, CITY OF TRACY

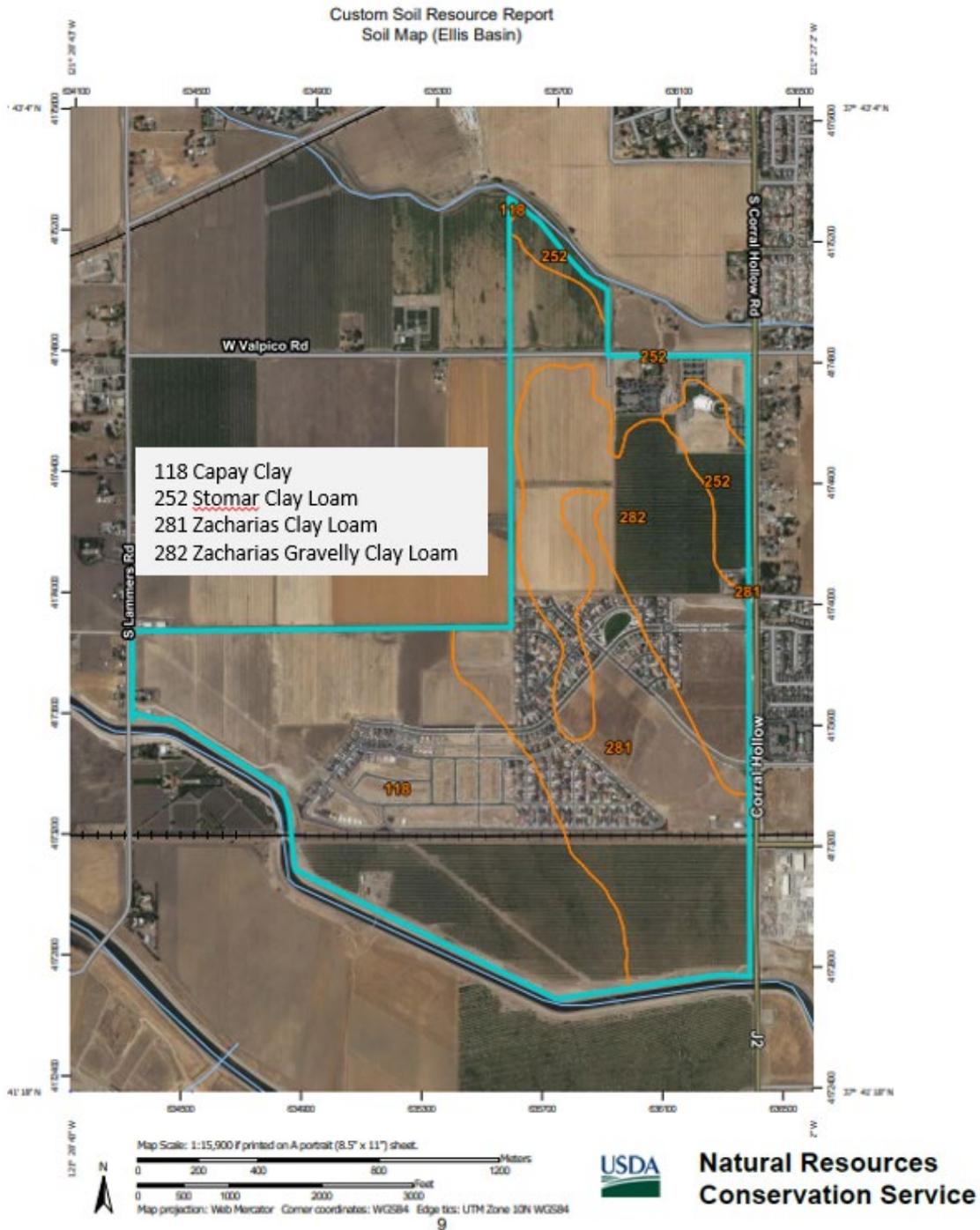


Figure 2-3 Study watershed soils map. Data sourced from NRCS Web Soil Survey.

Soils within the overall watershed are predominately characterized as clays, clay loams, or gravelly clay loams, and all are officially classified in Hydrologic Soil Groups (HSG) C.<sup>2</sup> Soils mapped as Capay clays are found in roughly the western third of the watershed, a transition to Zacharias clay loams occurs to the north and east, which also underlie approximately one third of the area. Zacharias gravelly loams are mapped in a band through the eastern portions of the watershed and cover roughly one quarter of the area. The remaining area in the northeast is underlain with Stomar clay loam.

The underlying soil types are hugely important to the design considering that a soils natural capacity to infiltrate and store water is a primary driver in determining runoff potential on both a storm event and annual basis. The characteristics of the upper A Horizon of the soil column are the most important in this regard and include the saturate hydraulic conductivity ( $K_{sat}$  measured in units of inches/hour) and measures of the moisture retention potential such as available water capacity, tension storage, and soil storage. The tabulated properties for the A Horizon of the soils in the watershed are summarized in **Table 2-2**.

**Table 2-2 Study area soil types and key characteristics.** Data sourced from NRCS Web Soil Survey.

Map Unit Symbol	Map Unit Name	$K_{sat}$ (in/hr)	Thickness (inches)	Available Water Capacity (in/in)	Tension Storage (inches)	Soil Storage (inches)
118	Capay Clay	0.11	20	0.15	3.0	5.0
252	Stomar Clay Loam	0.38	17	0.17	2.9	3.9
281	Zacharias Clay Loam	0.38	19	0.17	3.2	4.4
282	Zacharias Gravelly Loam	0.38	14	0.13	1.8	3.8

*All soil properties are for the A horizon only.  $K_{sat}$  is the saturated hydraulic conductivity.*

*Available water capacity is the water retained in the horizon after draining and is modeled as tension storage.*

*Soil storage is the pore volume of the soil (assumed porosity = 0.4) less the tension storage.*

## 2.5 Drainage Features and Patterns

### 2.5.1 PRE-DEVELOPMENT DRAINAGE AND RUNOFF

The gently sloping agricultural lands that characterized the watershed prior to urbanization had no notable drainage features. This reflects the fact that the surficial soils generally have sufficient capacity to infiltrate and store the relatively low annual rainfall.

<sup>2</sup> The Natural Resources Conservation Service classifies soils into hydrologic soil groups based on overall runoff potential. The lowest runoff and highest infiltration rate soils are in HSG A, while those with highest runoff potential and lowest infiltration rates are in HSG D.

Any runoff from large storm events would typically be stored temporarily in depressions but would not be sufficient to form an organized channel network.

### 2.5.2 PROPOSED DRAINAGE SYSTEM

The increased runoff potential accompanying urbanization requires a coordinated approach to stormwater management and drainage infrastructure. This is particularly true in the southwest portions of the City where the natural tendency will be for runoff to flow to the north and east with the potential to impact the existing capacity-limited City storm drain system.

The Citywide Storm Drainage Master Plan in 2012 recognized the need to effectively manage runoff from the watershed to avoid adverse impacts to the large, downslope Westside Channel watershed that comprises much of the previously urbanized parts of the City. Hydrologic and hydraulic modeling in that document identified capacity constraints in the downstream storm drain system that required major reductions in the maximum outflow rate from the study watershed. On that basis, the 2012 plan called for two large stormwater detention basins (South of Linne Basin and Basin 3A) to meter the peak flow from the watershed to no more than 3 cfs.

The modeling associated with the 2012 report was further refined in the recent update to the CSDMP prepared by Wood Rodgers. The drainage framework identified in that document is illustrated in **Figure 2-4**, which is an excerpt from the original Figure 26 in the CSDMP<sup>3</sup>. The drainage configuration outline in the updated report called for a stormwater detention basin for the South of Linne area ("DET SL" south of the ACE railroad tracks) with a total storage capacity of 42.5 acre-feet and a maximum 100-year outflow rate of 1.1 cfs. That basin would be connected to a planned 66-inch diameter storm drain line that would serve as the main collector, i.e., trunk line, to convey runoff to Basin 3A (DET 3A). Basin 3A was identified as requiring a total storage capacity of 122 acre-feet and a maximum 100-year outflow rate of 3.5 cfs. At the time of the CSDMP update, it was assumed that Basin 3A would drain to the Westside Channel storm drain network. That connection was shown as an approximately 4,300-foot run of 18-inch diameter pipe

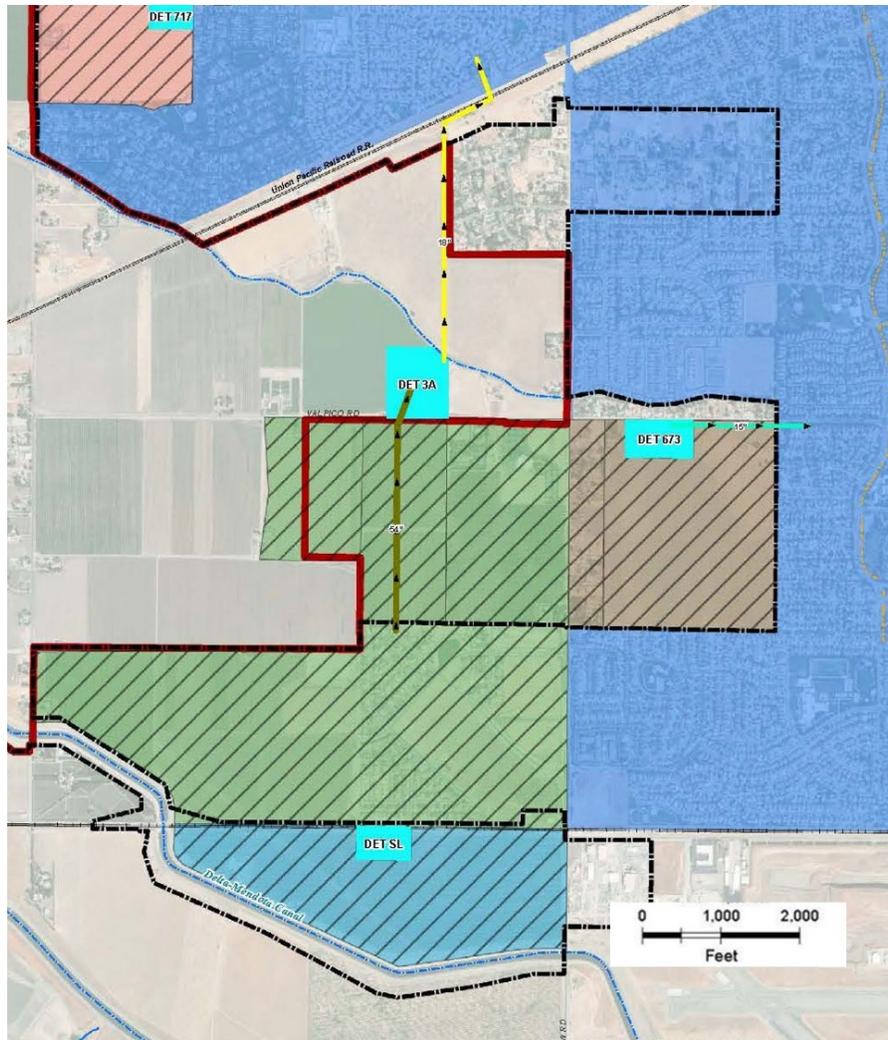
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<sup>3</sup> It should be noted that there is no current plan to develop the hatched area in Figure 2-4 located to the west of the red boundary, adjacent to "The Avenues" watershed. If that area is developed in the future, modifications can be made to Basin 3A to accommodate the additional runoff if more storage is required.

## HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A STORMWATER FACILITY, CITY OF TRACY

running north to and then under the Union Pacific railroad tracks and connecting to the existing storm drain system at Carol Ann Drive.

The assumptions underlying the sizing of Basin 3A presented in the CSDMP were: one, that the basin would function as a traditional gravity outflow detention basin, and two, that the peak flow rate to the Westside Channel watershed would need to be limited to 3.5 cfs to preserve the channel capacity of the receiving waters. The modeling described in the CSDMP included infiltration based on saturated hydraulic conductivity values from USDA NRCS soil data. The success of other infiltration basins in the water shed - specifically, at the church properties along Valpico Road and, on a much larger scale, for interim stormwater management for the first phases of the Ellis Project - prompted the updated hydrologic modeling presented in this report to assess the opportunity to construct and operate Basin 3A as an infiltration basin with sufficient capacity to infiltrate all runoff from the contributing watershed. Configuring the basin as an infiltration facility maximizes recharge of the underlying groundwater basin and eliminates the need for a connection to the downstream storm drain system, thus offloading planned runoff to the Westside Channel watershed.



**Figure 2-4 Regional drainage infrastructure as proposed in the Citywide Storm Drainage Master Plan Update.** Graphic excerpted from Figure 26 of the report prepared by Wood Rodgers.

## 2.6 FEMA Floodplain Information

Neither Basin 3A nor its watershed are mapped as special flood hazard areas (aka, “100-year floodplains”) by the Federal Emergency Management Agency. The pertinent Flood Insurance Rate Map would be Panel 06077C0730F but has not been published by FEMA due to the lack of mapped flood hazards.

### 3 BASIN 3A CONFIGURATION AND STORAGE VOLUME

This report section discusses pertinent characteristics of Basin 3A that informed both the basis of design and hydrologic modeling.

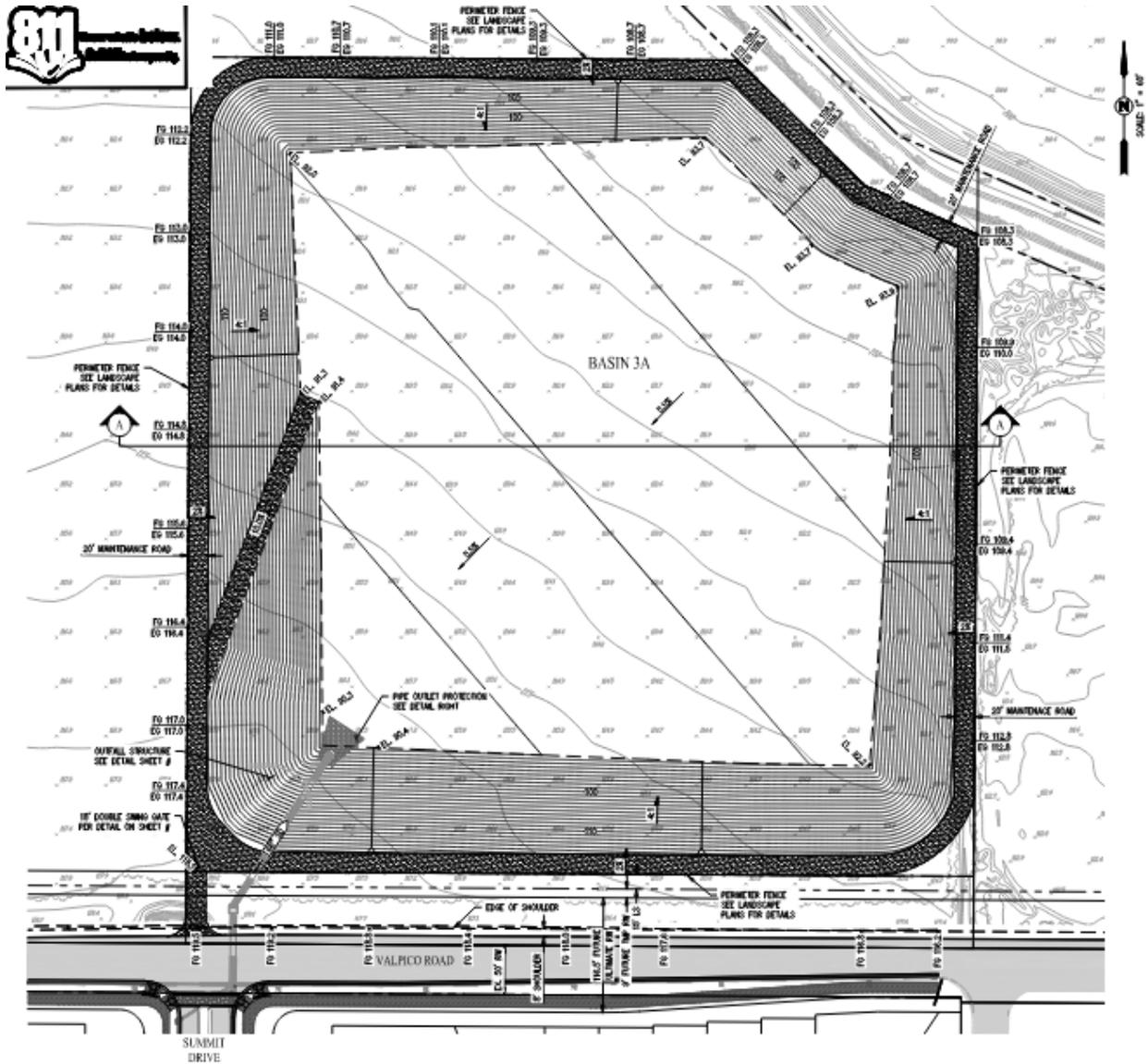
#### 3.1 Location and Footprint

The basin will be constructed by excavating an area located immediately north of Valpico Road. The full storage volume will be excavated below existing grade. The lowest top of bank grades will be 108.3 feet at the northeast corner of the basin, and the total area at the 108-foot contour is 11.8 acres. The basin will be surrounded by an access/maintenance road and other setbacks giving it a somewhat larger footprint; conservatively, a total area of just over 16 acres was used in the hydrologic modeling to represent the area that will drain directly to the basin. Top of bank elevations are greatest at the southwest corner of the basin at 118 feet, but these higher top of bank elevations do not increase the total storage volume, which is set by the low point in the northeast corner.

Inflow will come to a small degree from direct rainfall on the basin and to a much larger degree from a single 66-inch trunk storm drain line constructed as part of the Avenues Project along the alignment of Summit Drive. The single main inflow point is advantageous in that it focuses routine maintenance in a specific area.

#### 3.2 Basin 3A Characteristics

Basin 3A, shown in **Figure 3-1**, will consist of a single, large, excavated area occupying the entirety of the roughly square overall basin footprint. The basin floor will slope to the southwest from roughly 94 feet at the northeast corner to a low point, near the trunk line inlet, at 90.3 feet. An access ramp will be provided on the eastern side of the basin to facilitate routine inspections and maintenance.



**Figure 3-1 Basin 3A Plan.** Excerpted from basin plans prepared by Carlson, Barbee & Gibson.

As discussed previously, there will be no regular outlet from Basin 3A since it is designed for full infiltration of all incoming runoff up to and including that associated with the 100-year, 10-day design storm.

### 3.3 Storage Capacity

The respective elevation-storage relationship for the Basin 3A was taken from basin geometry information provided by CBG and is summarized in **Table 3-1**, below. The total storage volume at the controlling top of bank elevation (108 feet) is 156 acre-feet.

HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A  
 STORMWATER FACILITY, CITY OF TRACY

**Table 3-1 Elevation-storage relationships for the Ellis Basin 3A Main Basin.**

<b>Elevation</b> <i>(feet)</i>	<b>Area</b> <i>(acres)</i>	<b>Storage</b> <i>(ac-ft)</i>	<b>Elevation</b> <i>(feet)</i>	<b>Area</b> <i>(acres)</i>	<b>Storage</b> <i>(ac-ft)</i>
90.4	0.01	0.00	100	9.80	69.59
91	0.66	0.20	101	10.03	79.50
92	3.40	2.23	102	10.28	89.66
93	6.83	7.35	103	10.52	100.05
94	8.42	14.98	104	10.76	110.70
95	8.64	23.51	105	11.01	121.58
96	8.87	32.26	106	11.26	132.72
97	9.10	41.25	107	11.52	144.11
98	9.33	50.46	108	11.77	155.76
99	9.56	59.91			

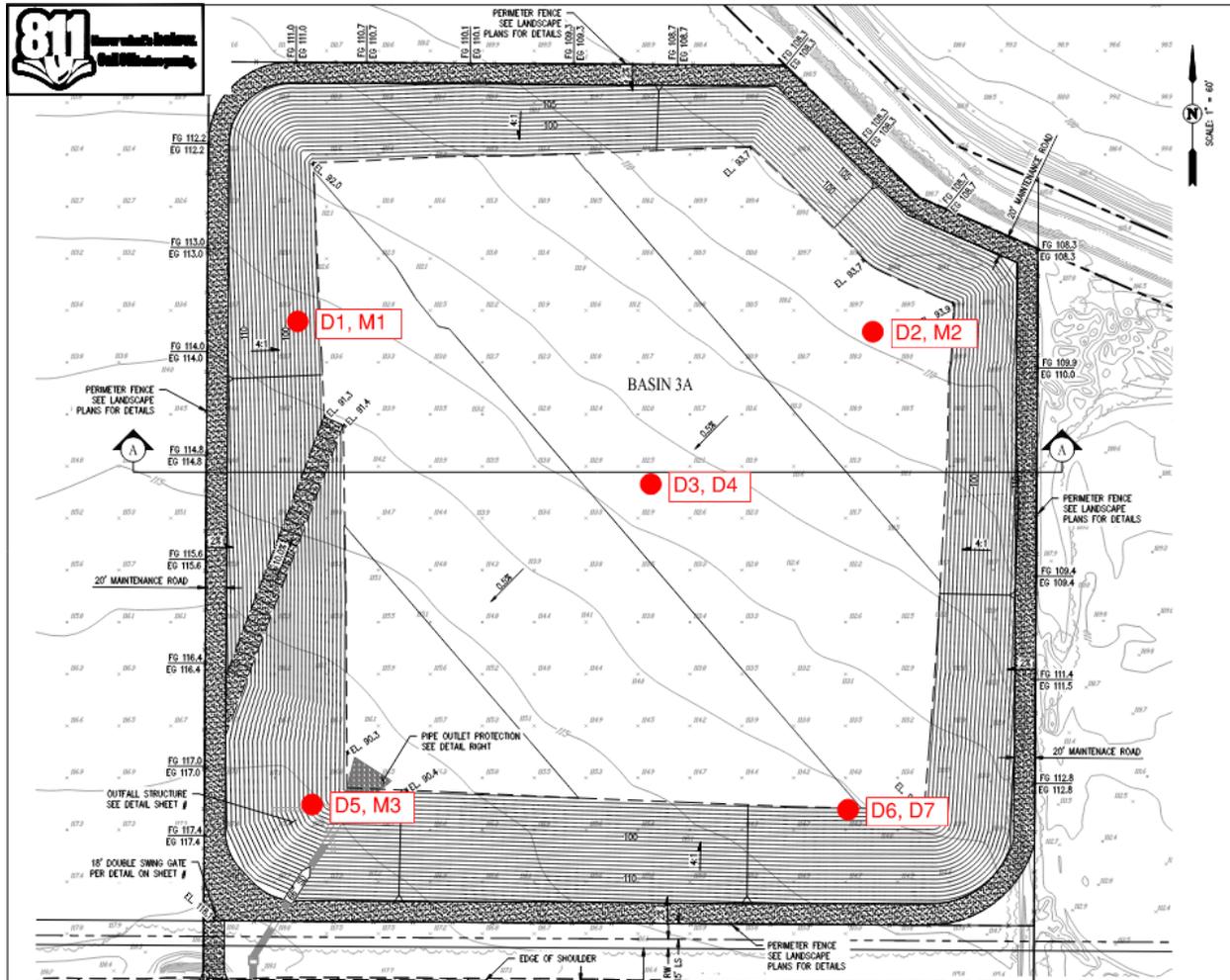
Total Storage = 155.76 acre-feet

## 4 PERCOLATION TESTING AND DESIGN INFILTRATION CAPACITY

Construction and safe operation of Basin 3A as an infiltration basin requires careful consideration of the site-specific geologic characteristics as they relate to managed groundwater recharge. These include the nature and distribution of underlying strata, the degree of separation from the ground water table, and the infiltration capacity of the soils at the anticipated elevation of the basin floor. Through communication with Wood Rodgers, Appendices C and D of the Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans for South Orange County (SOC Guidance) were used as a basis for guiding the infiltration testing and design component of the project. The information presented in the SOC Guidance was used to inform a comprehensive field investigation program carried out by Wallace Kuhl and Associates in early 2022. The pertinent findings were presented in the Geotechnical Engineering Report Ellis Avenues Detention Basins prepared in March 2022 (Ellis Basins Geotech Report). Key aspects of that report that impact the hydrologic modeling are discussed below.

### 4.1 Field Test Borings

A key component of the field testing and evaluation was the completion of a set of borings at five different locations within the footprint of the proposed basin. The general locations of the borings are shown in **Figure 4-1**. The number, depth, and location of the borings were selected to gain a better understanding of the heterogeneity of the subsurface conditions beneath the basin, the depth to groundwater at the site, and the ability for water to infiltrate into the underlying soil column.



**Figure 4-1** General locations of boring pairs from the January 2022 field tests. Each pair consists of a boring to the proposed basin depth and a deeper boring to characterize underlying strata.

## 4.2 Percolation Test Results

Percolation tests were completed in borings D1, D2, D4, D5, and D7 using a slightly modified version of the U.S. Environmental Protection Agency Falling Head Test procedures. The tests were completed at the proposed depth of the infiltration basin in

compliance the standards outlined in the SOC Guidance<sup>4</sup>. The pertinent data for the final stabilized falling head results in each boring are summarized in **Table 4-1**.

While the results for most of the tests are within two inches for measured water level fall, the measured fall at boring D4, located at the center of the basin, is an order of magnitude lower than the results at the four other locations. The D3 and D4 boring logs provide no clear indication that a layer of fine-grained material is present at the depth that the percolation test was performed; however, considering that a roughly 4-foot-thick sandy lean clay layer was observed in the MW1 soil boring, it can be assumed that there is a discontinuous layer of fine sediment located in the vicinity of the D4 boring. To remedy the significantly lower infiltration rates measured at the D4 boring, Wallace Kuhl and Associates has developed a remediation plan to outline methods for finding and removing the clay layer such that the basin floor can be adequately connected with deeper, permeable layers such that overall infiltration is improved. For a full description of the percolation test methods and the proposed clay remediation plan please refer to the Ellis Basin Geotech Report.

**Table 4-1 US EPA falling head test results and Porchet conversions.** Data collected by Wallace Kuhl & Associates in January 2022.

<b>Boring</b>	<b>Time (min)</b>	<b>Measured Depths</b>			<b>Fall (inches)</b>	<b>Porchet Parameters</b>		
		<b>Start (inches)</b>	<b>Finish (inches)</b>	<b>Average (inches)</b>		<b>Volume (in<sup>3</sup>)</b>	<b>Area (in<sup>2</sup>)</b>	<b>Rate (in/hr)</b>
D1	10	6	1	3.50	5.0	65.9	131.9	3.00
D2	10	6	2.2	4.10	3.8	50.1	143.2	2.10
D4	10	6	5.7	5.85	0.3	4.0	138.5	0.17
D5	10	6	1.1	3.55	4.9	64.6	132.8	2.92
D7	10	6	2.4	4.20	3.6	47.5	145.1	1.96

<sup>4</sup> After completion of the geotechnical investigation the basin grading was revised to omit a levee that would've been constructed at the northeastern corner of the basin adjacent to the West Side Irrigation District Upper Main Canal. To maintain the necessary design storage and freeboard, the depth of Basin 3A was lowered between 1 and 7 feet, with the largest decreased occurring in the southwest corner of the basin.

### 4.3 Calculated Infiltration Rates per Field Data

Upon receipt of the percolation results from Wallace Kuhl and Associates, the Porchet Method was used to convert the percolation rate to a measured infiltration rate that could be used to inform the sizing of Basin 3A. As seen in **Figure 4-2**, the Porchet Method corrects the measured percolation to discount additional losses through the sidewalls of the borehole<sup>5</sup>.

$$I_t = \frac{\Delta H \pi r^2 60}{\Delta t (\pi r^2 + 2\pi r H_{avg})} = \frac{\Delta H 60 r}{\Delta t (r + 2H_{avg})}$$

Where:

- $I_t$  = tested infiltration rate, inches/hour
- $\Delta H$  = change in head over the time interval, inches
- $\Delta t$  = time interval, minutes
- $r$  = effective radius of test hole
- $H_{avg}$  = average head over the time interval, inches

#### Figure 4-2 Porchet Method

Considering that a remediation plan will be developed to remove any near-surface fines at the bottom of the basin, we feel that it is appropriate to omit the results of the D4 infiltration test when determining the average measured infiltration rate. As evidenced by **Table 4-1**, this results in an increase from 0.54 to 0.67 inches/hour in the measured infiltration rate. The effect of the soil remediation will be quantified by further infiltration testing performed after the basin has been excavated and the underlying soil remediated.

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<sup>5</sup> The infiltration test method used by Wallace Kuhl and Associates differs slightly from what is assumed in Figure 4-2; in their field testing, Wallace and Kuhl extended a 2-inch perforated PVC pipe to the base of the boring and back-filled with gravel (imagine a donut of gravel at the bottom of the boring) to perform the infiltration testing. Conversely, the Porchet Method assumes that gravel is applied uniformly across the bottom of the boring before infiltration testing is conducted. The calculations included in Table 4-1 are adjusted to account for this discrepancy.

**Table 4-2 Design Infiltration Rate.**

	Design Infiltration Rate (in/hr)	
	Without D4 Test	With D4 Test
Average Measured Infiltration Rate	2.50	2.03
Safety Factor	3.75	3.75
Design Infiltration Rate	0.67	0.54

#### 4.4 Design Safety Factor

As shown in **Table 4-2**, a final step in converting the measured percolation rate to an acceptable design infiltration rate for use in sizing the basin was to apply a factor of safety to the measured values. The SOC Guidance provides a worksheet shown below in **Figure 4-3** for use in determining an accurate factor of safety. The worksheet considers a variety of site characteristics and design parameters that are weighted 1, 2, or 3 based on the effect that the individual component will have on the ability for water to infiltrate into the soil; the higher the value, the higher likelihood that infiltration will be impeded.

HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A  
STORMWATER FACILITY, CITY OF TRACY

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25	1	0.25
		Predominant soil texture	0.25	2	0.5
		Site soil variability	0.25	2	0.5
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Tributary area size	0.25	3	0.75
		Level of pretreatment/ expected sediment loads	0.25	3	0.75
		Redundancy	0.25	3	0.75
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor, $S_{TOT} = S_A \times S_B$				<b>3.75</b>	
Measured Infiltration Rate, inch/hr, $K_M$ (corrected for test-specific bias)				<b>2.50</b>	
Design Infiltration Rate, in/hr, $K_{DESIGN} = S_{TOT} \times K_M$				<b>0.67</b>	

**Figure 4-3 Factor of Safety and Design Infiltration Rate Worksheet.** Outlined in Section VII.4.3 of the SOC Technical Guidance.

Based on the information from the geotechnical investigation and the design features, the safety factor for the Basin 3A site was determined to be 3.75. Notable positive features that influenced the calculation are the soil assessment methods, and the depth to groundwater – groundwater was not encountered during the site investigation. Conversely, detrimental variables are the large size of the tributary area and the lack of redundancy of upstream BMPs in the site design. It is assumed that minimizing compaction during construction can be easily managed and tested with additional infiltration tests upon completion of the basin to ensure that the construction process has not negatively impacted the infiltration rate of the soil. Based on the average measured infiltration rates – calculated from the Porchet Method - and the combined safety factor, the design infiltration rate used to size the basin is 0.67 inches per hour.

## 5 HYDROLOGIC MODELING METHODOLOGY AND INPUT

As noted previously, the City has recently updated its standards for stormwater infrastructure design, including the appropriate parameters and standards to be used for infiltration/retention basins. This report section provides an overview of the methodology used to develop the Basin 3A design to that it complies with the new standards. That is followed by a summary of the specific parameters used to set up and run the design hydrologic model.

### 5.1 Hydrologic Modeling Methodology

The primary design tool for assessing the basin size, configuration, and performance per the updated criteria is the U.S. Army Corps of Engineers HEC-HMS modeling platform (Version 4.9). This modeling software provides a number of advantages for basin design including the ability to model long-duration storm events, variations in infiltration rate, and inclusion of the SCS unit hydrograph methodology that provides consistency with the CSDMP. The modeling utilized the soil moisture accounting method to account for rainfall losses, including those to infiltration and evapotranspiration, in the contributing sub-watersheds over the multi-day design storm.

As discussed previously, the watershed area for Basin 3A currently has no direct connection to the City storm drain system. Prior to the Ellis Specific Plan, the area was used for agriculture, and local runoff (to the extent it occurred) infiltrated locally. Therefore, no pre-project model runs were completed for this study. Rather, the modeling focused on the post-project conditions under full projected build out, which is the same approach used in the CSDMP. The basin design will be conservative over the near-term, since the predicted total design runoff will be markedly less than what is predicted in the full build-out modeling. Of course, this also means that the basis of design presented herein will be valid for the long-term.

Previous City design standards for infiltration basins were based on a number of criteria, including the following:

- Design storm. The design storm rainfall was stipulated as a total of 3.12 inches, equivalent to the 0.26 feet, representing an estimate of the 10-year, 48-hour rainfall depth. No specific guidance was provided as to rainfall distribution.
- Runoff volume. Runoff volume was calculated using generalize coefficients based on ground cover type (i.e., 0.95 for paving, 0.80 for roofs, 0.20 for landscaped areas, etc.).

- Basin volume. Actual utilized infiltration basin volume was set to twice the runoff volume for the design storm.
- Basin freeboard. The required basin freeboard was 1 foot, with the shape and depth of the basin dependent on the design infiltration rate; basin geometry had to be such that retention basins could drain completely within ten calendar days.

The former standards are updated and refined by the new criteria that are clearly presented in the Citywide Storm Drainage Master Plan updated prepared by Wood Rodgers. For infiltration/retention basins these include:

- Design storm. The updated design storm is now based on a predicted 100-year, 10-day event with multiple storms of varying magnitude occurring in succession. The total rainfall depth is obtained from data published by the National Oceanic and Atmospheric Administration. The total 10-day rainfall depth in this case is 4.69 inches.
- Runoff volume. Runoff volume is now calculated using an appropriately parameterized rainfall-runoff model so that infiltration basin inflow and outflow can be explicitly modeled on the 10-day period.
- Basin volume. Basin volume is now predicated by the maximum required active storage for the 10-day duration design storm plus freeboard.
- Basin freeboard. The design freeboard requirement in the updated standards is reframed to be at least 25 percent of the total basin volume. The 2012 CSDMP assumed that most detention basins were 5 feet in depth. At this depth 1 foot of freeboard roughly equates to 25 percent of the storage volume. The requirement was updated such that the functional freeboard requirement could be applied to a wider variety of basin geometries.

## 5.2 HEC-HMS Input Parameters

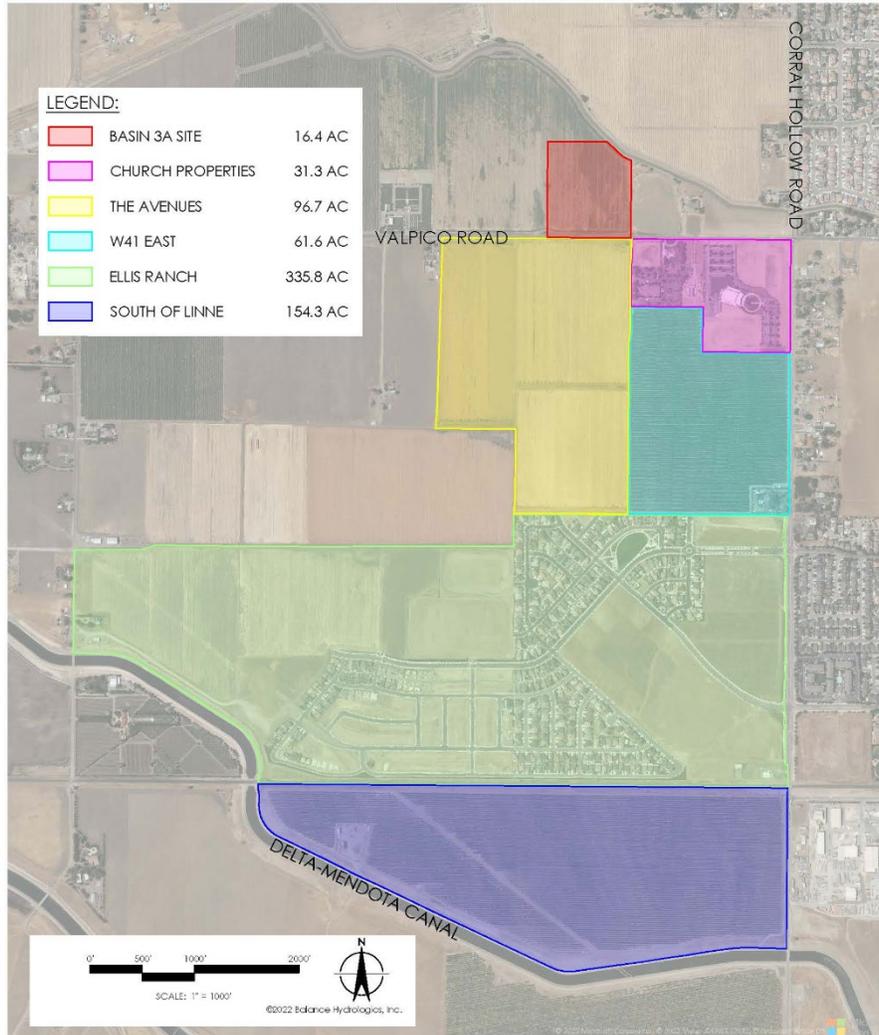
As with all hydrologic analyses, the selection of appropriate modeling parameters is an important consideration. Key hydrologic components and parameters are summarized and discussed below.

### 5.2.1 SUB-WATERSHED AREAS

The overall watershed that is programmed to drain to Basin 3A is illustrated in **Figure 5-1**. This area was divided into a total of six sub-watersheds to account for the differences in

HYDROLOGIC BASIS OF DESIGN REPORT FOR THE ELLIS BASIN 3A  
STORMWATER FACILITY, CITY OF TRACY

future land use and five of these were explicitly modeled using the rainfall-runoff methodology. Rainfall-runoff characteristics for the southern-most sub-watershed (“South of Linne”) were not explicitly considered since it is programmed to have its own future stormwater detention basin and the CSDMP has already defined the maximum outflow (see the further discussion in **Section 5.2.6**).



**Figure 5-1 Basin 3A sub-watersheds used in the hydrologic modeling.**

Land use parameters for the five sub-watersheds modeled in detail are summarized in **Table 5-1**. It is important to note that the impervious cover for each of the sub-watersheds is not reduced for on-site Low Impact Development measures. Therefore, the modeling represents the full anticipated runoff based on the actual planned impervious cover (assumed to all be directly connected).

**Table 5-1 Summary of land use type hydrologic parameters**

<b>Sub-watershed</b>	<b>Area</b>		<b>Impervious</b>			<b>Pervious</b>	
	<i>(acres)</i>	<i>(sq miles)</i>	<i>(acres)</i>	<i>(sq miles)</i>	<i>(%)</i>	<i>(acres)</i>	<i>(sq miles)</i>
Ellis Ranch	335.8	0.5247	251.9	0.3935	75	84.0	0.1312
Avenues	96.7	0.1511	58.0	0.0907	60	38.7	0.0604
W41 East	61.6	0.0963	37.0	0.0578	60	24.6	0.0385
W41B Churches	31.3	0.0489	24.1	0.0377	77	7.2	0.0112
Basin 3A	16.4	0.0256	14.8	0.0231	90	1.6	0.0026
<b>Total</b>	<b>541.8</b>	<b>0.8466</b>	<b>385.7</b>		<b>71.2</b>	<b>156.1</b>	

### 5.2.2 SOIL MOISTURE ACCOUNTING PARAMETERS

The soil moisture accounting method was used to account for rainfall losses over the multi-day design storm. The method has the capacity to model losses due to canopy interception, evaporation (and evapotranspiration), and infiltration to the shallow soil layer, and deeper groundwater layers. The only two components used in the current modeling are evaporation and infiltration to the topsoil layer.

The evaporation component uses a constant monthly rate of 1.45 inches, representative of the average rate for December through February per the data previously summarized in **Table 2-1**. Composite topsoil layer values were used for the infiltration parameters, weighted by the percent areal coverage of the various soil types. The resulting composite values are summarized in **Table 5-2** below.

**Table 5-2 Composite surface soil hydrologic parameters by sub-watershed**

<b>Sub-watershed</b>	<b>K<sub>sat</sub></b> <i>(in/hr)</i>	<b>Tension Storage</b> <i>(inches)</i>	<b>Soil Storage</b> <i>(inches)</i>
Ellis Ranch	0.24	2.9	4.6
Avenues	0.38	2.6	4.1
W41 East	0.38	3.8	3.8
W41B Churches	0.38	4.2	4.2
Basin 3A	0.38	4.3	4.3

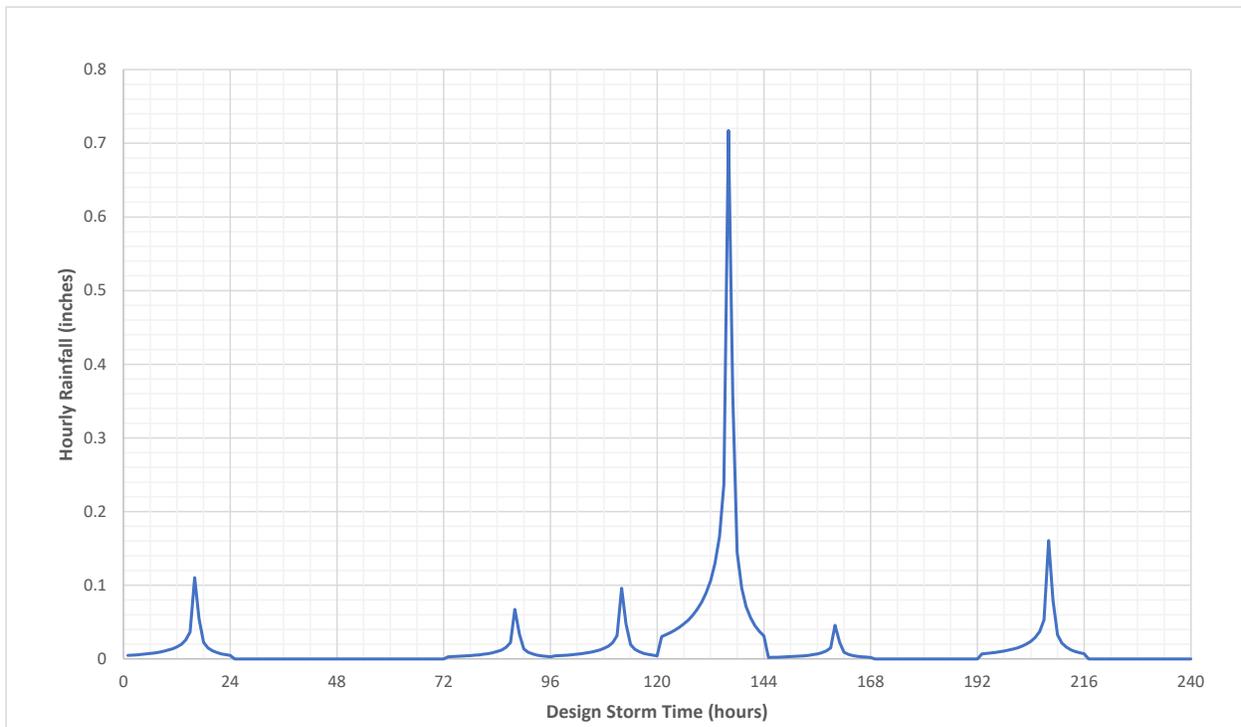
Except for the sub-watershed representing the basin, the sub-watersheds were divided into a pervious and impervious component and parameterized separately. The main

difference between the two components is that infiltration into the soil layer is not allowed for the impervious area, though evaporation is allowed from both. Basin 3A was parameterized with only one sub-basin component as the effective impervious there represents rainfall directly into basin.

### 5.2.3 DESIGN STORM CHARACTERISTICS

The 100-year, 10-day design storm distribution is illustrated in **Figure 5-2** and is based directly on guidance from the CSDMP. Rainfall across the 10-day period totals 4.69 inches and is based on precipitation frequency data from the NOAA Atlas 14 database. The pertinent data from Atlas 14 is included in **Appendix A**.

The rainfall distribution includes precipitation on six days of the 10-day period and preserves the pertinent 100-year totals for shorter periods as well. For example, the 24-hour rainfall total on day six is equivalent to the Atlas 14 100-year, 24-hour total.



**Figure 5-2** Rainfall distribution for the 100-year, 10-day design storm. Total precipitation is 4.69 inches as per data from the NOAA precipitation frequency data server (Atlas 14 data).

#### 5.2.4 RAINFALL-RUNOFF HYDROGRAPH AND LAG TIMES

The modeling used the SCS unit hydrograph with the HEC-HMS software. Basin 3A is designed as a terminal full infiltration basin, and peak inflow rates are much less important than total runoff within the modeling time frame for infiltration facilities. Therefore, lag times for each sub-watershed were simply set to a uniform value of 15 minutes. The choice lag time does not impact the calculated overall runoff volume, which only depend on the rainfall and loss factors. Lag time does impact the shape and peak flow rate of the output hydrograph, and the peak flow rates from the modeling should only be view as generally indicative of actual full 100-year inflow rates.

#### 5.2.5 CONTROL SPECIFICATIONS

The control specifications for the model runs were set for an arbitrary 14-day time period in January 2040. A 14-day period was selected so all runoff from the 10-day storm would be accounted for and so that the basin drawdown rate after the end of the design storm could be explicitly simulated.

#### 5.2.6 INFLOW FROM SOUTH OF LINNE

The earlier discussion noted that the South of Linne sub-watershed was not used in the rainfall-runoff modeling. This reflects the fact that the area south of the Ellis sub-watershed and the railroad tracks is programmed to have a separate stormwater basin. Design of that basin will occur at some time in the future and specific runoff modeling is not needed if the anticipated outflow from the basin is accounted for in the Basin 3A design. Per the CSDMP, the South of Linne basin is also programmed to have an outflow connection to the Ellis storm drain system, which will convey the outflow to Basin 3A. The maximum allowed discharge from the South of Linne basin has been identified as 1 cfs in the CSDMP. Although a value of 1 cfs is quite small, it does represent an additional inflow of roughly 24 acre-feet to Basin 3A over the 14-day simulation period, and that is explicitly included in the modeling as a constant 1 cfs source flowing into the main Ellis trunk storm drain line. This means that the Basin 3A design basis will be valid as long as the 1 cfs maximum flow rate criteria is applied to future development in the South of Linne sub-watershed.<sup>6</sup>

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<sup>6</sup> The future South of Linne basin may well be designed as a full infiltration facility with no surface outflow like Basin 3A. In that case, there will be a significant reduction in total runoff to Basin 3A, and it will have excess capacity.

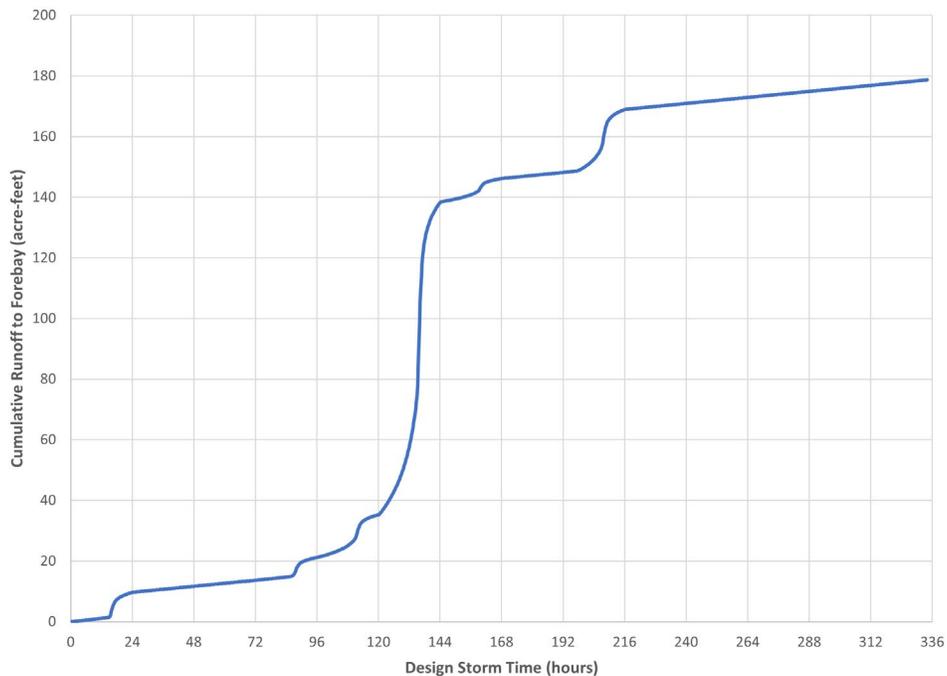
## 6 MODEL OUTPUT AND PREDICTED BASIN PERFORMANCE

This section summarizes the key output from the HEC-HMS modeling. Additional details related to the model input and output are included in **Appendix B**.

### 6.1 Runoff Rates and Volume

The peak flow rate into the basin for the 100-year, 10-day event is predicted to be on the order of 370 cfs occurring at 16:00 hours on the sixth day of the rainfall period. As noted previously, this peak flow rate is only approximate as it is derived using a set lag time for the sub-watershed and does not include the details of future in-tract storm drain networks.

More important from an infiltration performance perspective is the total design storm runoff volume. **Figure 6-1** shows the cumulative runoff in acre-feet to Basin 3A through the design storm.

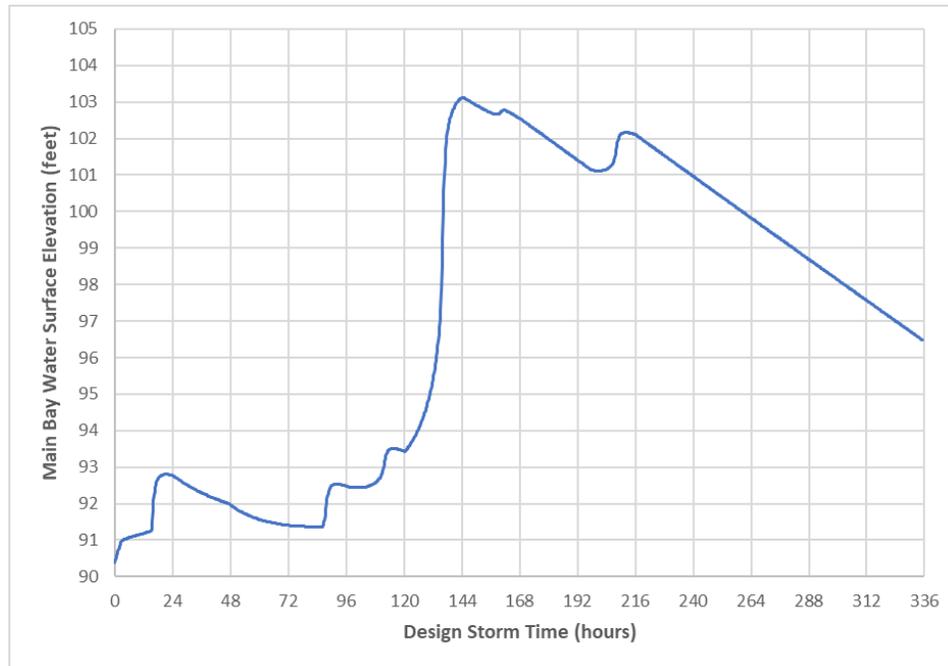


**Figure 6-1 Modeled cumulative runoff to Basin 3A.**

Over the 14-day simulation period this comes to 179 acre-feet, of which just under 28 acre-feet come from the South of Linne basin. Therefore, roughly 151 acre-feet of runoff originates from the watershed north of Linne.

## 6.2 Maximum Water Surface Elevation and Storage

The modeled Basin 3A water surface elevation over the course of the design storm is illustrated in **Figure 6-2**.



**Figure 6-2** Modeled water surface elevation in Basin 3A.

The maximum predicted water surface elevation is 103.1 feet, which is well below the minimum top of bank elevation of 108 feet. The equivalent maximum storage in the basin is 101.4 acre-feet occurring just after hour 144 of the model run.

## 6.3 Facility Freeboard

Considering the maximum design storage of 101.4 acre-feet mentioned above, the minimum basin capacity needed to meet freeboard requirements is 126.8 acre-feet. This is roughly 30 acre-feet below the overall basin storage volume of 155.8 acre-feet. Given the additional storage in the basin, there is potential to direct more runoff to the basin if additional land is developed in the region.

## 6.4 Drawdown and Storage Recovery

The predicted basin water surface elevations shown in **Figure 6-2** also illustrate the drawdown in the basin after the end of the 10-day rainfall period (i.e., after a storm time

of 240 hours). The maximum water surface attained in the last storm peak is 102.2 feet and this draws down to 97.7 feet in the ensuing four days. In terms of storage, this represents a recovery of approximately 44 acre-feet of storage, equivalent to an average recovery rate of about 11 acre-feet per day.

### 6.5 Basin Performance per Water Quality Management Criteria

The modeling demonstrates that Basin 3A will infiltrate all the runoff from an atmospheric river-style, long duration series of storms such as the 100-year, 10-day design event. Therefore, it is a given that the basin will be more than capable of infiltrating the runoff from the routine storm events that generate 80 percent of the mean annual runoff, which serves as the treatment target for water-quality measures under the Statewide MS4 Stormwater Permit. The latter fact, coupled with the large vertical separation from the local groundwater table (in excess of 15 feet) shows that the facility can function as the water-quality management facility for the entire contributing watershed. Inclusion of other upstream runoff water-quality measures will not be necessary for the City to assure compliance with the requirements of the MS4 permit.

## 7 LIMITATIONS

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

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

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

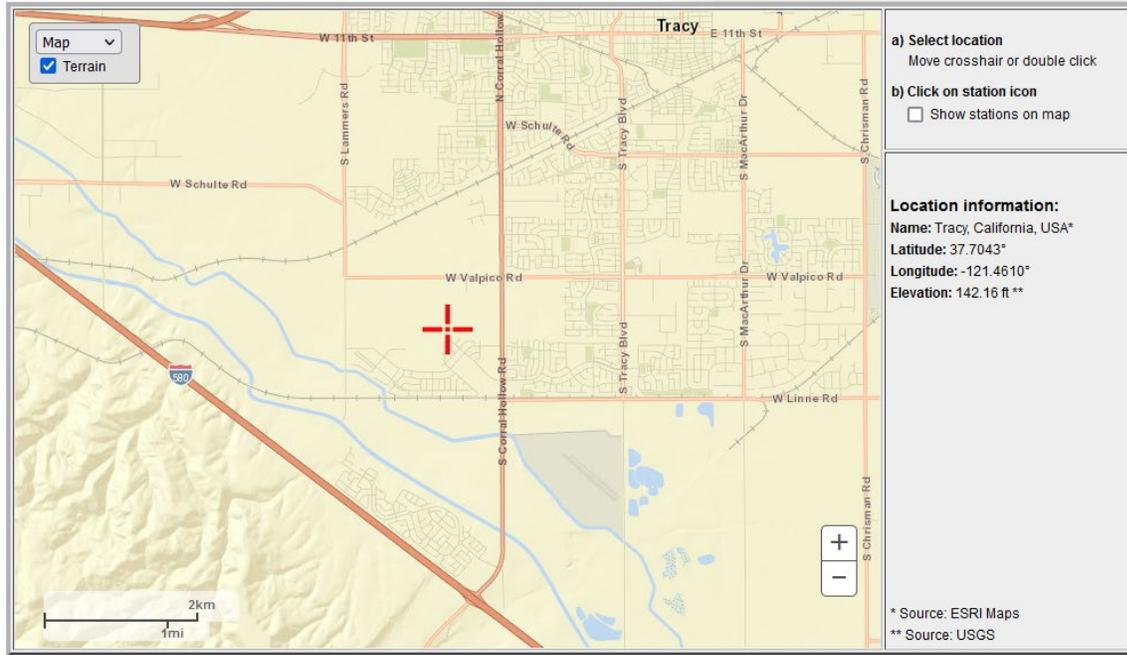
To aid in revisions, we ask that readers or reviewers who have additional pertinent information of new plans, data, or other information, who have observed changed conditions, or who may note material errors should contact us with their findings as soon as possible, so that timely changes may be made.

## **APPENDICES**

## **APPENDIX A**

### **NOAA Precipitation Data**

## Site Rainfall Statistics for the Ellis Specific Plan Area from the NOAA Precipitation Frequency Data Server



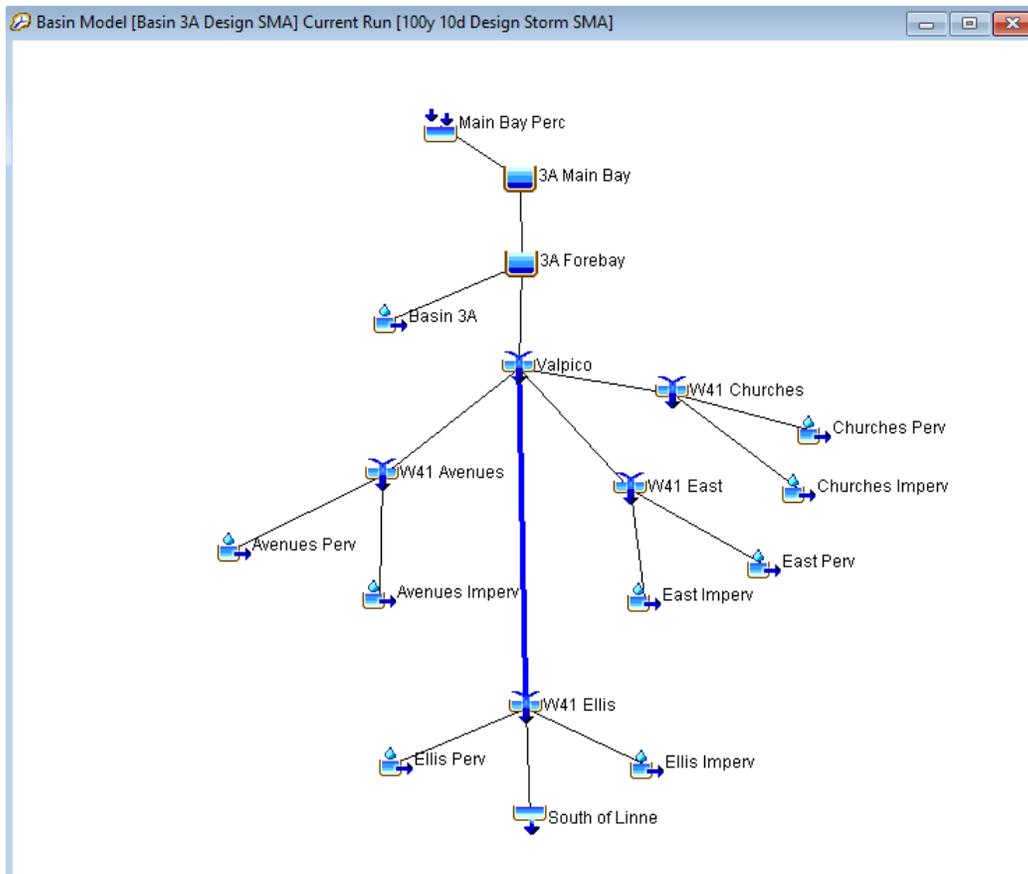
<b>PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)<sup>1</sup></b>										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.080 (0.068-0.096)	0.094 (0.080-0.113)	0.116 (0.098-0.140)	0.137 (0.114-0.166)	0.172 (0.138-0.217)	0.205 (0.160-0.264)	0.245 (0.187-0.325)	0.296 (0.218-0.404)	0.383 (0.270-0.549)	0.470 (0.319-0.700)
10-min	0.115 (0.097-0.138)	0.135 (0.114-0.162)	0.167 (0.140-0.200)	0.197 (0.164-0.239)	0.246 (0.198-0.310)	0.293 (0.230-0.378)	0.351 (0.268-0.466)	0.424 (0.313-0.580)	0.549 (0.387-0.787)	0.673 (0.457-1.00)
15-min	0.139 (0.118-0.167)	0.164 (0.138-0.196)	0.202 (0.170-0.242)	0.238 (0.198-0.288)	0.298 (0.239-0.375)	0.355 (0.278-0.458)	0.425 (0.324-0.563)	0.512 (0.378-0.701)	0.664 (0.468-0.951)	0.814 (0.552-1.21)
30-min	0.188 (0.159-0.225)	0.221 (0.187-0.265)	0.273 (0.229-0.328)	0.322 (0.268-0.390)	0.403 (0.323-0.507)	0.480 (0.376-0.619)	0.574 (0.437-0.761)	0.693 (0.511-0.948)	0.897 (0.632-1.29)	1.10 (0.747-1.64)
60-min	0.264 (0.223-0.315)	0.310 (0.261-0.371)	0.382 (0.321-0.459)	0.450 (0.375-0.546)	0.564 (0.452-0.710)	0.672 (0.526-0.886)	0.804 (0.612-1.07)	0.970 (0.716-1.33)	1.26 (0.885-1.80)	1.54 (1.05-2.30)
2-hr	0.379 (0.320-0.453)	0.445 (0.376-0.533)	0.546 (0.459-0.656)	0.639 (0.533-0.775)	0.788 (0.632-0.993)	0.923 (0.723-1.19)	1.08 (0.823-1.43)	1.27 (0.935-1.73)	1.57 (1.10-2.24)	1.84 (1.25-2.74)
3-hr	0.456 (0.385-0.546)	0.538 (0.454-0.644)	0.660 (0.555-0.793)	0.772 (0.643-0.936)	0.946 (0.759-1.19)	1.10 (0.861-1.42)	1.27 (0.971-1.69)	1.48 (1.09-2.02)	1.79 (1.26-2.57)	2.07 (1.41-3.09)
6-hr	0.593 (0.501-0.710)	0.705 (0.595-0.845)	0.868 (0.730-1.04)	1.01 (0.845-1.23)	1.23 (0.990-1.55)	1.42 (1.11-1.83)	1.63 (1.24-2.16)	1.86 (1.38-2.55)	2.21 (1.56-3.17)	2.51 (1.70-3.74)
12-hr	0.761 (0.642-0.910)	0.922 (0.778-1.10)	1.15 (0.968-1.38)	1.35 (1.13-1.64)	1.65 (1.32-2.08)	1.89 (1.48-2.44)	2.16 (1.65-2.87)	2.46 (1.81-3.36)	2.89 (2.03-4.14)	3.24 (2.20-4.83)
24-hr	0.953 (0.846-1.10)	1.18 (1.05-1.36)	1.49 (1.32-1.72)	1.76 (1.54-2.04)	2.14 (1.82-2.57)	2.45 (2.04-3.00)	2.77 (2.26-3.48)	3.12 (2.48-4.02)	3.62 (2.76-4.85)	4.03 (2.97-5.58)
2-day	1.14 (1.01-1.31)	1.41 (1.25-1.63)	1.78 (1.58-2.06)	2.09 (1.83-2.43)	2.51 (2.13-3.01)	2.83 (2.36-3.47)	3.16 (2.58-3.97)	3.51 (2.78-4.52)	3.98 (3.03-5.33)	4.35 (3.20-6.02)
3-day	1.24 (1.10-1.43)	1.54 (1.36-1.77)	1.94 (1.71-2.23)	2.26 (1.98-2.62)	2.69 (2.28-3.22)	3.01 (2.51-3.69)	3.34 (2.72-4.19)	3.68 (2.92-4.74)	4.13 (3.14-5.53)	4.47 (3.29-6.18)
4-day	1.35 (1.20-1.55)	1.68 (1.49-1.93)	2.11 (1.86-2.43)	2.45 (2.15-2.85)	2.91 (2.47-3.49)	3.25 (2.71-3.98)	3.59 (2.92-4.50)	3.93 (3.12-5.06)	4.38 (3.34-5.87)	4.72 (3.48-6.53)
7-day	1.61 (1.43-1.85)	2.01 (1.78-2.31)	2.52 (2.23-2.91)	2.93 (2.57-3.41)	3.46 (2.94-4.15)	3.85 (3.21-4.72)	4.23 (3.45-5.31)	4.61 (3.66-5.94)	5.10 (3.88-6.83)	5.45 (4.02-7.54)
10-day	1.78 (1.58-2.05)	2.23 (1.98-2.57)	2.80 (2.48-3.24)	3.26 (2.86-3.79)	3.84 (3.27-4.61)	4.27 (3.56-5.23)	4.69 (3.82-5.87)	5.09 (4.04-6.55)	5.61 (4.27-7.51)	5.97 (4.40-8.27)
20-day	2.32 (2.06-2.66)	2.93 (2.60-3.37)	3.70 (3.27-4.27)	4.30 (3.77-5.00)	5.06 (4.30-6.08)	5.61 (4.68-6.87)	6.14 (5.00-7.70)	6.65 (5.27-8.55)	7.27 (5.54-9.73)	7.70 (5.67-10.7)
30-day	2.65 (2.35-3.04)	3.37 (2.99-3.88)	4.27 (3.77-4.93)	4.96 (4.35-5.77)	5.84 (4.97-7.01)	6.47 (5.39-7.93)	7.07 (5.75-8.86)	7.63 (6.05-9.82)	8.31 (6.33-11.1)	8.77 (6.46-12.1)
45-day	3.27 (2.90-3.76)	4.18 (3.70-4.81)	5.29 (4.67-6.10)	6.13 (5.38-7.13)	7.20 (6.12-8.65)	7.97 (6.64-9.76)	8.68 (7.07-10.9)	9.35 (7.41-12.0)	10.2 (7.74-13.6)	10.7 (7.88-14.8)
60-day	3.93 (3.49-4.51)	5.01 (4.44-5.77)	6.33 (5.60-7.30)	7.32 (6.42-8.51)	8.57 (7.28-10.3)	9.45 (7.87-11.6)	10.3 (8.37-12.9)	11.0 (8.75-14.2)	12.0 (9.11-16.0)	12.6 (9.25-17.4)

## **APPENDIX B**

### **HEC-HMS Model Input and Output**

# HEC-HMS Model Input and Output

## Model Schematic



## Basin Area

Subbasin Area [Basin 3A Design SMA]    Show Elements: All Elements    Sorting: Hydrologic

Subbasin	Area (MI <sup>2</sup> )
Ellis Imperv	0.3935
Ellis Perv	0.1312
East Imperv	0.0578
East Perv	0.0385
Avenues Imperv	0.0907
Avenues Perv	0.0604
Churches Imperv	0.0377
Churches Perv	0.0112
Basin 3A	0.0256

## Surface

Simple Surface [Basin 3A Design SMA] — □ ×

Show Elements: All Elements Sorting: Hydrologic

Subbasin	Initial Storage (%)	Max Storage (IN)
Ellis Imperv	0	0.2
Ellis Perv	0	0.2
East Imperv	0	0.2
East Perv	0	0.2
Avenues Imperv	0	0.2
Avenues Perv	0	0.2
Churches Imperv	0	0.2
Churches Perv	0	0.2
Basin 3A	0	0.2

## Soil Moisture Accounting Parameters

Soil Moisture Accounting Loss [Basin 3A Design SMA]

Show Elements: All Elements

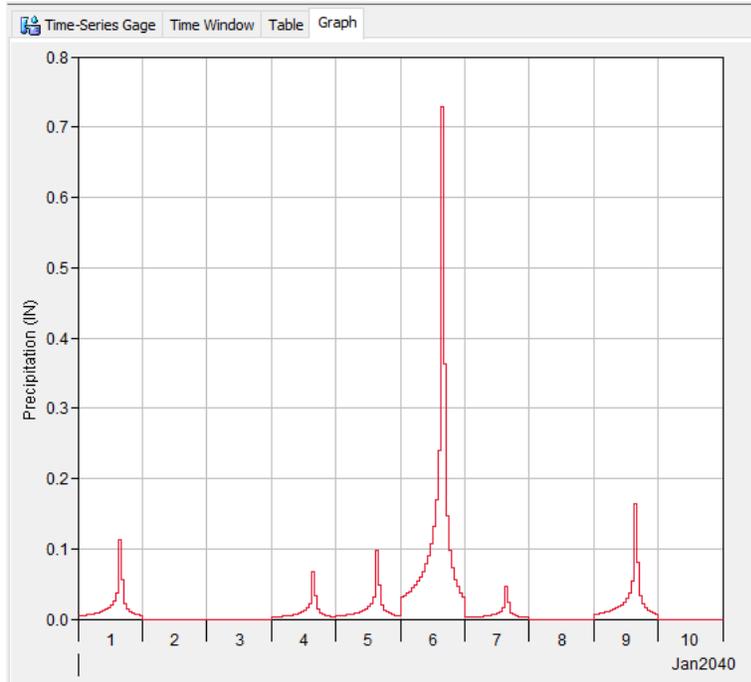
Subbasin	Soil (%)	Groundwater 1 (%)	Groundwater 2 (%)	Maximum Infiltration (IN/HR)	Impervious (%)	Soil Storage (IN)	Tension Storage (IN)
Ellis Imperv	10	10	10	0	0.0	4.6	2.9
Ellis Perv	10	10	10	0.24	0.0	4.6	2.9
East Imperv	10	10	10	0	0.0	3.8	2.2
East Perv	10	10	10	0.38	0.0	3.8	2.2
Avenues Imperv	10	10	10	0	0.0	4.1	2.6
Avenues Perv	10	10	10	0.38	0.0	4.1	2.6
Churches Imperv	10	10	10	0	0.0	4.2	3.1
Churches Perv	10	10	10	0.38	0.0	4.2	3.1
Basin 3A	10	10	10	0.38	90	4.3	3.2

— □ ×

Sorting: Hydrologic

Soil Percolation (IN/HR)	Groundwater 1 Stor... (IN)	Groundwater 1 Perc... (IN/HR)	GW1 Coefficient (HR)	Groundwater 2 Stor... (IN)	Groundwater 2 Perc... (IN/HR)	GW2 Coefficient (HR)
0.24	10	0.24	2	0	0.24	4
0.24	10	0.24	2	0	0.24	4
0.38	10	0.38	2	0	0.38	4
0.38	10	0.38	2	0	0.38	4
0.38	10	0.38	2	0	0.38	4
0.38	10	0.38	2	0	0.38	4
0.38	10	0.38	2	0	0.38	4
0.38	10	0.38	2	0	0.38	4
0.38	10	0.38	2	0	0.38	4

## Design Storm



## Main Basin Stage-Storage-Discharge

Elevation (FT)		Storage (ACRE-FT)	Discharge (CFS)
	90.4	0.00	0.01
	91.0	0.20	0.45
	92.0	2.23	2.30
	93.0	7.35	4.61
	94.0	14.98	5.69
	95.0	23.51	5.84
	96.0	32.26	5.99
	97.0	41.25	6.15
	98.0	50.46	6.30
	99.0	59.91	6.46
	100.0	69.59	6.62
	101.0	79.50	6.78
	102.0	89.66	6.94
	103.0	100.05	7.11
	104.0	110.70	7.27
	105.0	121.58	7.44
	106.0	132.72	7.61
	107.0	144.11	7.78
	108.0	155.76	7.95

## Runoff at Valpico Road

Summary Results for Junction "Valpico"

Project: Basin 3A Mar 2022    Simulation Run: 100y 10d Design Storm SMA  
Junction: Valpico

Start of Run: 01Jan2040, 00:00    Basin Model: Basin 3A Design SMA  
End of Run: 14Jan2040, 23:00    Meteorologic Model: 100y 10d Storm  
Compute Time: 15Apr 2022, 12:07:09    Control Specifications: 14-day Control

Volume Units:  IN     ACRE-FT

Computed Results

Peak Discharge: 358.7 (CFS)	Date/Time of Peak Discharge: 06Jan2040, 16:00
Volume: 172.93 (ACRE-FT)	

## Runoff at Main Basin

Summary Results for Reservoir "Basin 3A no Levee - D4"

Project: Basin\_3A\_Jun\_2022    Simulation Run: 100y 10d SMA no Levee -D4  
Reservoir: Basin 3A no Levee - D4

Start of Run: 01Jan2040, 00:00    Basin Model: SMA Basin 3A No Levee - D4  
End of Run: 14Jan2040, 23:00    Meteorologic Model: 100y 10d Storm  
Compute Time: DATA CHANGED, RECOMPUTE    Control Specifications: 14-day Control

Volume Units:  IN     ACRE-FT

Computed Results

Peak Inflow: 370.2 (CFS)	Date/Time of Peak Inflow: 06Jan2040, 16:00
Peak Discharge: 7.1 (CFS)	Date/Time of Peak Discharge: 07Jan2040, 00:15
Inflow Volume: 178.8 (ACRE-FT)	Peak Storage: 101.4 (ACRE-FT)
Discharge Volume: 142.1 (ACRE-FT)	Peak Elevation: 103.1 (FT)