JEFFERSON PARK STORMWATER CAPTURE PROJECT STORMWATER TREATMENT ALTERNATIVES REPORT

September 18, 2026

PRESENTED TO:

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1 INTRODUCTION

The City of Pasadena (City) is committed to its role and responsibilities in addressing the requirements of the National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) Permit Order No. R4-2012-0175 (Permit), which has been superseded by the Regional MS4 Permit Order No. R4-2021-0105 (Regional MS4 Permit). As a participating member of the Upper Los Angeles River (ULAR) and Rio Hondo watersheds, the City of Pasadena collaborated with neighboring municipalities—Alhambra, Burbank, Calabasas, Glendale, Hidden Hills, La Cañada Flintridge, Los Angeles, Montebello, Monterey Park, Rosemead, San Fernando, San Gabriel, South El Monte, South Pasadena, San Marino, Temple City, Unincorporated County, and the Los Angeles County Flood Control District—to collectively comply with the requirements of the Regional MS4 Permit and to enhance water quality while restoring beneficial uses in local waterways and downstream receiving waters. Identification of regional stormwater capture projects served as a principal method to address water quality concerns from multiple jurisdictions within a watershed. The development of the Jefferson Park Stormwater Capture Project in the City of Pasadena represents another major opportunity to continue regional scale progress to achieve pollutant load reductions for the Rio Hondo and Upper Los Angeles River watersheds.

The 2022 Pasadena Stormwater Master Plan (SWMP) identified Jefferson Park as a high priority location to help the City achieve its pollutant load reduction compliance within the Rio Hondo assessment area while delivering a range of community benefits. Additionally, Jefferson Park is situated downstream of areas impacted by the Eaton Fire, including Altadena. As such, the project presents a critical opportunity to support the long-term watershed health and safeguard downstream water quality throughout the City.

Craftwater was retained by the City to evaluate the feasibility of implementing a regional stormwater capture project at Jefferson Park. The purpose of this report is to assess existing hydrologic and water quality conditions at both regional and local stormwater capture locations, including consideration of potential wildfire-related impacts.

1.1 Project Objectives

The goals for the Jefferson Park Stormwater Capture Project consist of compliance-based goals (i.e. the Watershed Management Program) as well as funding-based goals (i.e. Safe, Clean Water Transfer Agreement Metrics and Scoring Criteria). The overarching goals of the project are as follows:

- Improve Water quality outcomes in the Rio Hondo Watershed.
- Promote groundwater recharge benefits through the addition of infiltration facilities.
- Utilize community input to provide new park amenities.
- Educate the public on stormwater quality and local groundwater recharge and water resiliency.

1.1.1 Watershed Management Program

The City of Pasadena is a member agency of the ULAR Watershed Management Group (WMG). Under Section IX of Order R4-2021-0105 (Permit), the ULAR WMG developed a Watershed Management Program (WMP) to fulfill the requirements of the NPDES MS4 Permit. The WMP identified zinc and *E. coli* as the limiting pollutants during wet weather and *E. Coli* as the limiting pollutant during dry weather. It was stated that for over 100 outfalls along the LA River, over 85% of the samples exceeded Water Quality-based Effluent Limitations (WQBELs) for *E. coli* during the dry weather. Because structural control measures primarily target the treatment of sediment, heavy metals, and nutrients, bacteria compliance will adjacently be managed via retention of stormwater runoff volume through this project given its complex fate and transport. Therefore, zinc load reduction was the primary water quality driver for sizing this project. The limiting pollutant analysis in the ULAR WMP indicated that controlling zinc is likely to effectively address other water quality priorities, such as organics and legacy pollutants, that share similar fate and transport characteristics with zinc.

To maintain deemed compliance coverage for water quality priorities until their final deadlines (2028 for metals and 2037 for bacteria), the agencies with ULAR WMG must stay on track with the implementation requirements outlined in their WMP. An analysis was conducted to determine the baseline volume to retain under the bacteria storm, and the volume of best management practices (BMPs) to achieve the retention requirement under the bacteria storm critical condition. For the Rio Hondo assessment area, which includes the City of Pasadena, the volume of runoff to be retained under the bacteria storm critical condition for all 11 municipalities is 100.61 ac-ft. **Table 1-1** displays a summary of volumes to be managed by municipalities in the Rio Hondo assessment area to achieve final bacteria milestones.

***Table 1-1**. Summary of Volumes to be Managed by Municipalities in the Rio Hondo Assessment Area to Achieve Final Bacteria Milestones.*

MUNICIPALITY	BASELINE VOLUME DURING FINAL RAA CRITICAL CONDITION (AC-FT)	BMP CAPACITY REQUIRED DURING FINAL RAA CRITICAL CONDITION (AC-FT)
Alhambra	15.65	126.4
Montebello	11.01	98.0
Monterey Park	8.07	49.5
Pasadena	8.74	161.7
Rosemead	9.29	48.5
San Gabriel	7.03	42.5
San Marino	3.90	74.7
South El Monte	0.35	45.8
South Pasadena	23.88	2.1
Temple City	6.89	38.2
Unincorporated LA County	5.80	89.9
TOTAL	**100.61**	**777.3**

1.1.2 Safe, Clean Water Program Scoring Criteria

The project may seek funding through the Safe, Clean Water Program (SCWP). To be eligible for funding consideration, the project must demonstrate consistency with the SCWP scoring framework, which evaluates how well a project aligns with the goals of the regional program. The SCWP grant funding application includes several evaluation categories, including water quality benefits, water supply benefits, community investment, nature-based solutions, community engagement, and leveraged funds. Projects that achieve a minimum score of 60 points are eligible for funding consideration.

The following scoring categories are most directly influenced by the proposed treatment type and are therefore relevant to the evaluation of treatment alternatives:

- **Wet Weather vs. Dry Weather Project**: Projects may be classified as either wet weather or dry weather projects under the SCWP framework. While any project may be designated as a wet weather project, only projects designed to treat rainfall events of 0.25 inches or less may qualify as dry weather projects. Wet weather projects are eligible for up to 50 points within the Water Quality Benefits category, whereas dry weather projects are eligible for up to 40 points.

- **Water Quality**:
  - **Wet Weather Project**: Removal of 80% or more of divertible primary and secondary pollutants achieves the maximum 50 points in this category.
  - **Dry Weather Project**: Removal of 100% of all tributary dry weather flows from at least a 200-acre drainage area will achieve the maximum 40 points in this category.

- **Water Supply**: Points may be awarded for projects that capture stormwater for beneficial reuse or groundwater recharge. Higher scores are achieved by maximizing the annual water supply generated by the project while optimizing the life-cycle cost efficiency of the proposed system. Preliminary soil investigation results indicate that infiltration is feasible.
- **Nature-Based Solutions**: Points may be awarded for project elements that implement or mimic natural processes to manage stormwater, such as infiltration-based systems, the use of natural materials (e.g., soils and vegetation), or removal of impervious surfaces. The Jefferson Park project will further evaluate opportunities to incorporate nature-based solutions with the objective of capturing and infiltrating stormwater where feasible.

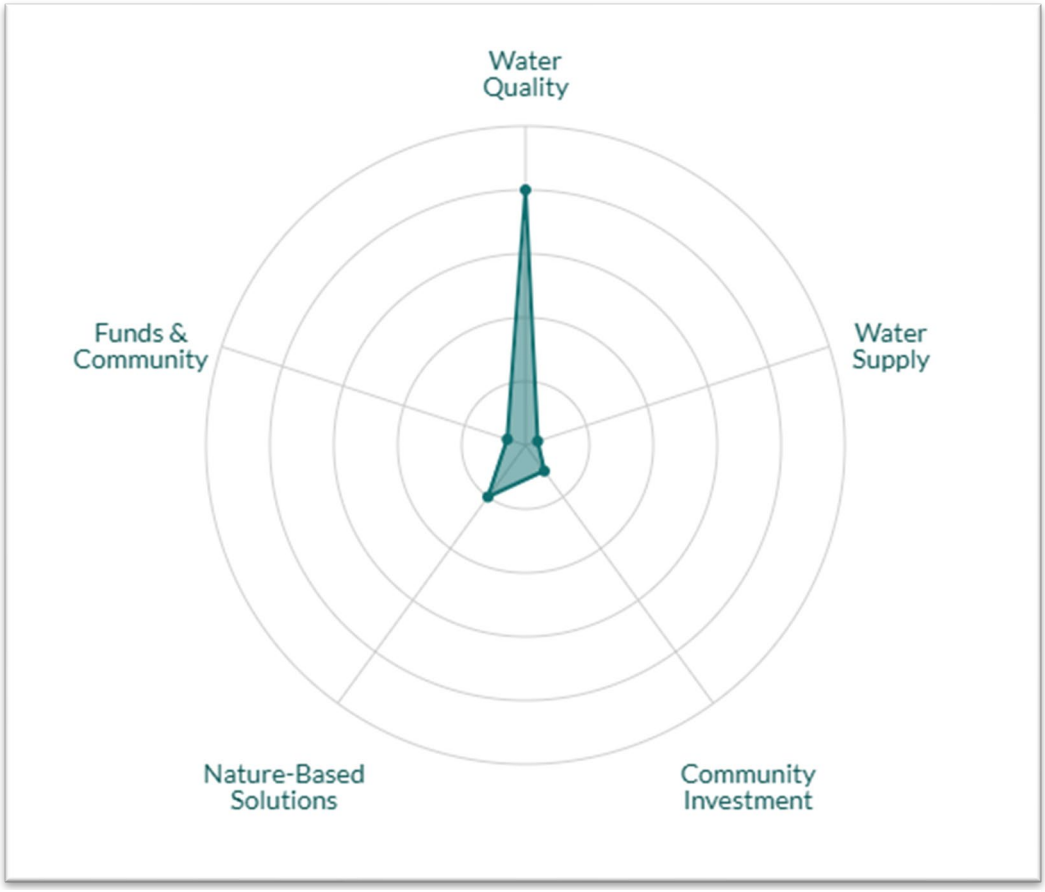


Figure 1-1. *Scoring categories within Safe, Clean Water Program application.*

2 EXISTING CONDITIONS SUMMARY

2.1 Baseline Hydrology

A previous **Existing Hydrology Report** was prepared and submitted in April 2026. This report developed baseline runoff and pollutant loading conditions using multiple modeling approaches to support different aspects of project evaluation and design.

2.1.1 10-Year Modeling Results

Continuous simulation modeling was performed over a 10-year period to estimate average annual runoff volumes and pollutant loads associated with the contributing regional drainage areas for the Rubio Wash (**Figure 2-1**) and BI 0189 (**Figure 2-2**) storm drains. These results provide the basis for evaluating overall project benefits, including annual water quality improvement and groundwater recharge potential, and can be found below in **Table 2-1** and **Table 2-2** for the Rubio Wash and BI 0189 storm drains, respectively.

Table 2-1. *10-Year Modeling Results for the Rubio Wash Diversion.*

PARAMETER	RUBIO WASH
Area	312.6
Impervious Area[1]	9.9%
Average Annual Runoff (ac-ft/yr)	79.7
Average Annual Zinc Load (lb/yr)	26.1
Average Annual Copper Load (lb/yr)	5.2

Table 2-2. *10-Year Modeling Results for BI0189 Diversion.*

PARAMETER	BI0189
Area	491.9
Impervious Area[2]	16.5%
Average Annual Runoff (ac-ft/yr)	142.8
Average Annual Zinc Load (lb/yr)	59.5
Average Annual Copper Load (lb/yr)	12.6

[1] Calculated from the WMMS 2.0 model's Complete HRUs file.
[2] Calculated from the WMMS 2.0 model's Complete HRUs file.

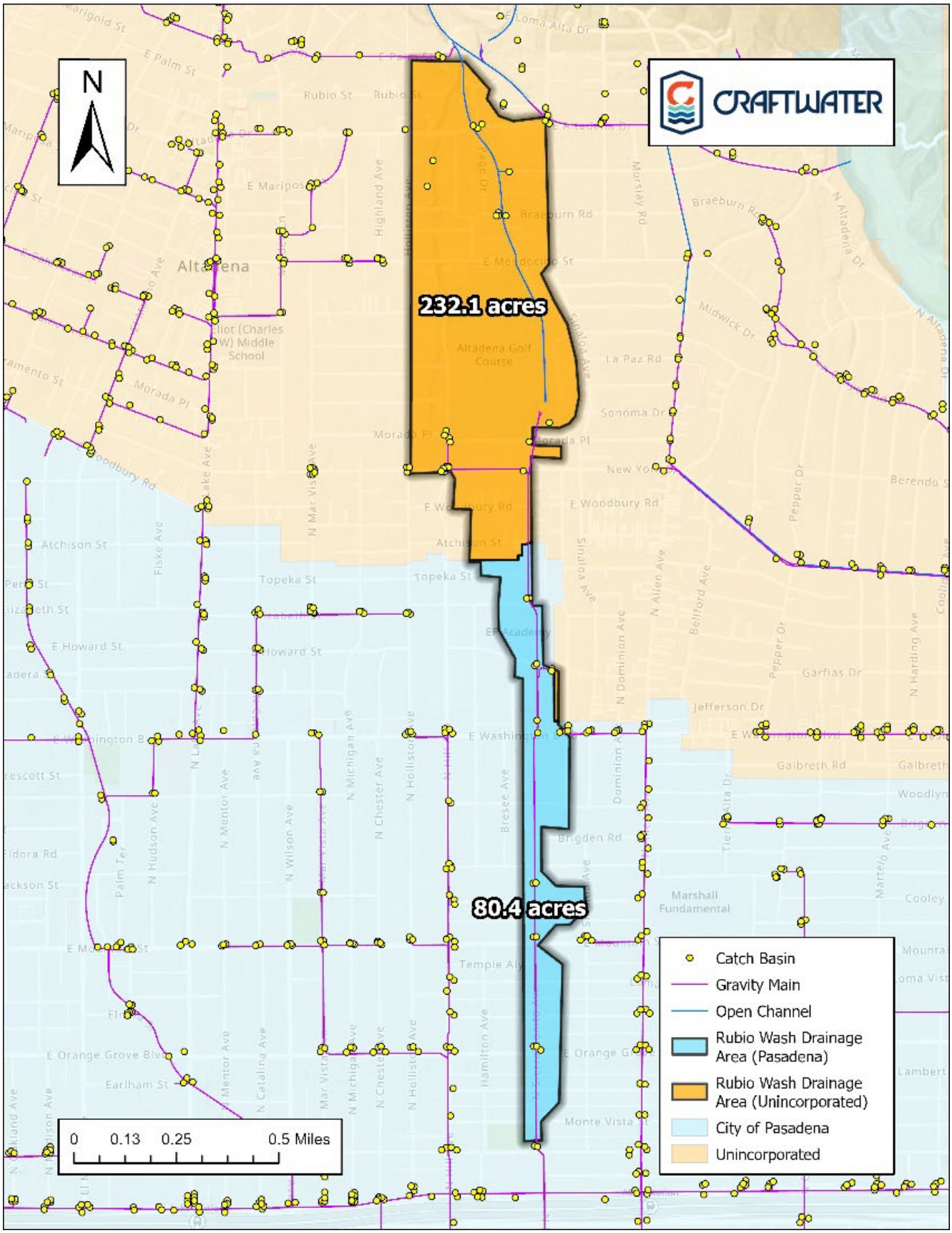


***Figure 2-1**. Drainage area jurisdiction map for the proposed Rubio Wash storm drain.*

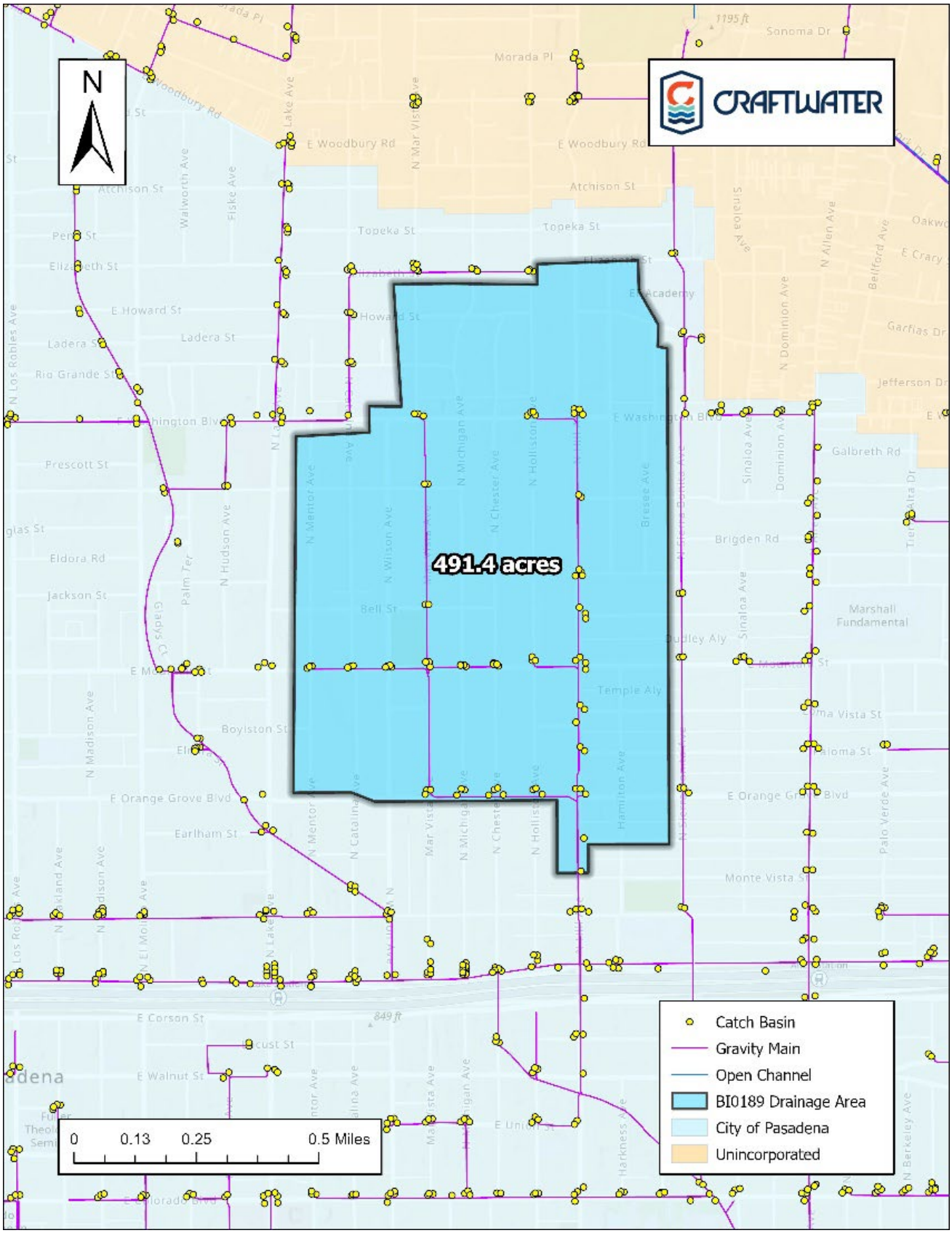


***Figure 2-2**. Drainage area jurisdiction map for the proposed BI 0189 storm drain showing the drainage area is located entirely within Pasadena City limits.*

2.1.2 85th Percentile, 24-Hour Design Storm Results

In addition to long-term simulations, event-based modeling was conducted to evaluate system performance under representative design storm conditions. The 85th percentile, 24-hour storm event was used to characterize peak flow rates and runoff volumes associated with a typical large storm and to support preliminary sizing of diversion and BMP infrastructure. The results of this analysis for regional and local drainage areas can be found in **Table 2-3**.

Table 2-3. *Modeling Results Depicting the 85th, 24-hour Storm.*

SITE	AREA	DESIGN STORM (IN)	PEAK FLOW (CFS)	TOTAL RUNOFF (AC-FT)
Rubio Wash	312.6	1.1	22.36	10.01
BI0189	491.9	1.1	38.59	17.91
Hamilton Ave	10.4	1	0.945	0.375
Sierra Bonita Ave East Basin	6.1	1	0.511	0.280
Sierra Bonita Ave West Basin	3.3	1	0.220	0.119

2.1.3 90th Percentile Bacteria Design Storm

An additional analysis was conducted for the 90th percentile bacteria storm, which represents the regulatory critical condition defined in the ULAR WMP. This storm is used to establish retention targets necessary to achieve compliance related to bacteria Total Maximum Daily Load (TMDL) requirements. The results of this analysis on the regional sites are presented in **Table 2-4** below.

Table 2-4. *Results of the Bacteria Critical Storm Simulation.*

PARAMETER	RUBIO WASH	BI0189
Area	312.6	491.9
Precipitation (in/yr)	0.19	0.19
Average Annual Runoff (ac-ft)	0.91	1.64

2.2 Geotechnical Investigation

A desktop-level geotechnical assessment was completed to inform preliminary design assumptions. This effort relied on publicly available soil data from the United States Department of Agriculture (USDA) Web Soil Survey, groundwater depth information from Los Angeles County Department of Public Works monitoring wells, and infiltration testing data provided by Torrent Resources. During preparation of this memorandum, preliminary results from a site-specific geotechnical investigation were prepared and were incorporated into the modeling analyses presented herein.

2.2.1 Soils Desktop Analysis

The Soil Survey Geographic Database (SSURGO), maintained by the Natural Resources Conservation Service (NRCS), was used to characterize subsurface conditions and assign a preliminary infiltration rate. According to the Web Soil Survey, soils underlying the site generally consist of sandy loam to gravelly sand, corresponding to Hydrologic Soil Group A. Reported saturated hydraulic conductivity (Ksat) values for the most limiting soil layer range from approximately 2.00 to 6.00 in/hr. The relationship between hydraulic conductivity and infiltration rate can be described using Darcy's Law:

$$k = v/i$$

Where:

- v is equal to the velocity or "infiltration rate" and,
- i is equal to the hydraulic gradient (pressure), where i is assumed to equal 1 in the absence of more data. However, it is acknowledged that the hydraulic gradient during infiltration can fluctuate within a structural control measure and may not always be exactly equal to 1.

To supplement the desktop soil characterization, Torrent Resources provided infiltration rates measured during dry well installations at sites in the broader vicinity of Jefferson Park. These locations are outside the 500-foot proximity typically considered representative under Safe Clean Water Program guidance; however, they represent the most relevant field data available and provide a useful basis for comparison. Details of these locations and corresponding infiltration rates are summarized in **Table 2-5**. These results suggest that higher infiltration rates may be present at greater depths, indicating that incorporating dry wells could be advantageous.

Table 2-5. *Locations Where Torrent Resources Installed Dry Wells and Recorded Infiltration Rates.*

ADDRESS	DISTANCE TO JEFFERSON PARK (FT)	DESIGN INFILTRATION RATE (IN/HR)
1880 E Walnut St	3,000	4
2116 E Villa St	3,500	2.86

2.2.2 Groundwater Depths

Well data from the Los Angeles County Public Works (LACPW) database were reviewed and summarized in **Table 2-6**. The shallowest groundwater depth recorded in the vicinity of the project was 208.3 feet in 2014 at LACDPW Well No. 4054. The closest well to Jefferson Park, LACDPW Well No. 4084, recorded a shallowest depth of 381.3 feet in April 2024. Over the past two decades, groundwater depths at Well No. 4084 have ranged from 381.3 feet (April 2024) to 455.5 feet (October 2012).

Based on these historical groundwater elevations, infiltration of stormwater captured by the project appears feasible. The Los Angeles County Low Impact Development Manual requires a minimum 10-foot separation between the bottom of a structural control and the groundwater

table. No wells in the vicinity of the park have recorded groundwater depths shallower than 100 feet within the past 20 years. Therefore, it is unlikely that groundwater would be encountered within 10 feet of the proposed subsurface basin invert.

Table 2-6.*Groundwater Wells in Vicinity of Jefferson Park.*

WELL ID	MONITORING PERIOD	APPROXIMATE LOCATION RELATIVE TO SITE	SHALLOWEST GROUNDWATER DEPTH (WITHIN THE LAST 20 YEARS)	LAST DEPTH MEASURED
#4084	January 1928-October 2024	778 ft north	384.3 (April 10, 2024)	399.3 (October 9, 2024)
#4085A	January 1931-October 2024	2,069 ft southeast	255.5 (April 12, 2023)	324.5 (October 9, 2024)
#4094	January 1928-October 2024	3,526 ft northeast	329.6 (May 19, 2011)	438.6 (October 9, 2024)
#4054	January 1928-October 2020	7,725 ft east	208.3 (August 26, 2014)	217.3 (October 13, 2020)

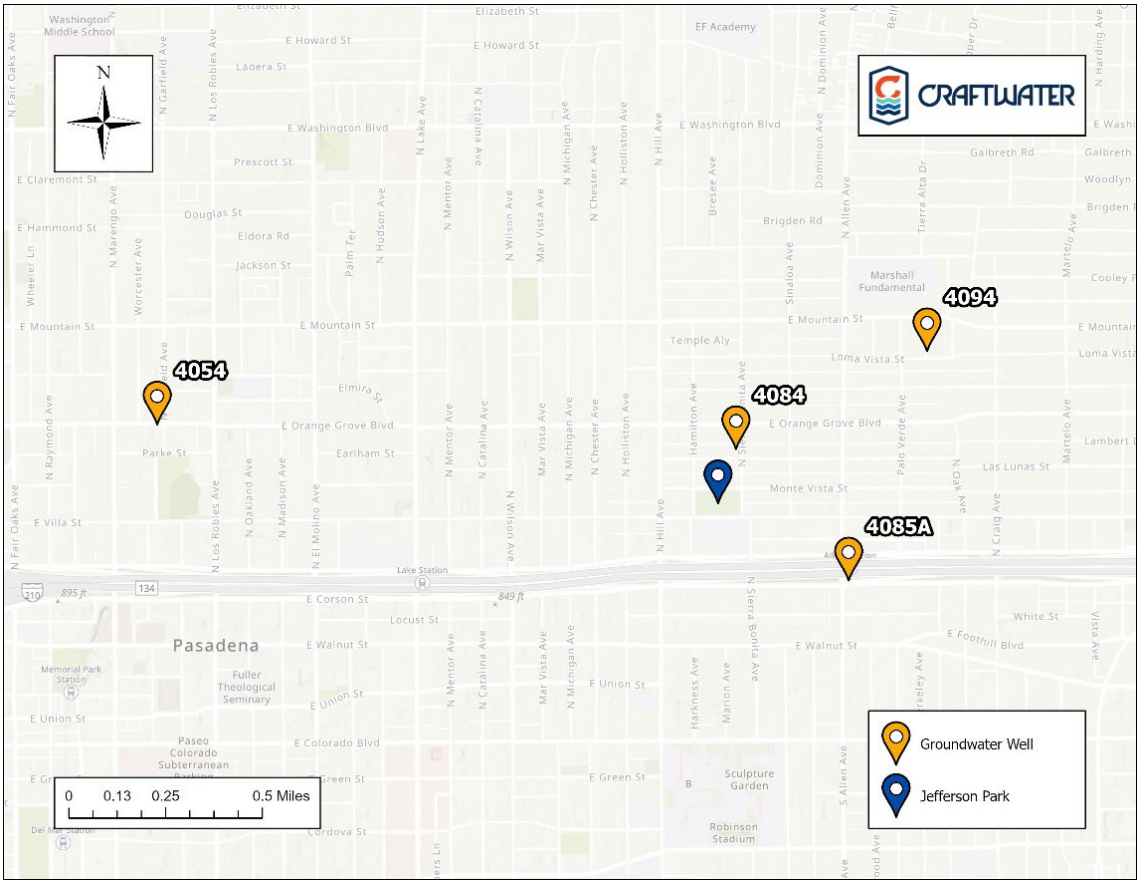


Figure 2-3. *Map Showing Groundwater Wells in Vicinity of Jefferson Park.*

2.2.3 Geotechnical Investigation

Geocon conducted a geotechnical investigation in May 2026. The boring locations are shown in **Figure 2-4** and preliminary infiltration test results are summarized in **Table 2-7**. Borings B1 and B2, tested at infiltration depths of approximately 15 to 20.5 feet below ground surface, exhibited infiltration rates indicating favorable conditions for the proposed subsurface infiltration gallery.

In contrast, Borings B3 and B4, evaluated at shallower depths, produced substantially lower infiltration rates. While B3 slightly exceeded the minimum infiltration rate of 0.3 in/hr required under the Los Angeles County LID Manual, B4 did not pass the minimum threshold. These findings suggest that shallow infiltration practices, such as permeable gutters or bioretention facilities, may not be feasible at the site, whereas deeper infiltration systems, such as subsurface infiltration galleries and dry wells, remain feasible.

***Figure 2-4**. Location of geotechnical borings at Jefferson Park.*

***Table 2-7**. Preliminary infiltration rates at Jefferson Park.*

BORING	INFILTRATION DEPTH (FT)	FIELD MEASURED INFILTRATION RATE (IN/HR)	REDUCTION FACTOR (RF)	DESIGN INFILTRATION RATE (IN/HR)
B1	15–20 ½	2.12	3.0	0.71
B2	15–20 ½	2.51	3.0	0.84
B4	3–5	0.67	3.0	0.22
B5	3–5	1.29	3.0	0.43

2.3 Existing Site Conditions

2.3.1 Regional Sites

The Jefferson Park project has the potential to capture regional flows from storm drains in the vicinity of the site. Diverted flows would be conveyed to subsurface storage galleries providing temporary detention during storm events. These facilities could reduce peak flows to downstream systems and may also support groundwater recharge through infiltration.

Three areas were identified as candidates for subsurface storage: the baseball field, the multi-use field, and the field adjacent to the restroom. Preliminary footprints meant for housing subsurface storage galleries and diversion facilities were developed using aerial imagery and are shown in **Figure 2-5**. These footprints are conceptual in nature, as detailed topographic and utility survey data are not yet available. A uniform setback of approximately 15 feet was applied from property boundaries, buildings, mature trees, and visible infrastructure to reduce the likelihood of conflicts. The maximum available areas associated with each field are summarized in **Table 2-8**.

***Table 2-8.** Estimated Available Footprint Areas for Potential Subsurface Storage Galleries and Diversion Facilities.*

FIELD	AVAILABLE FOOTPRINT (AC)
Baseball	1.01
Multi-use field	0.43
Restroom Adjacent	0.15

2.3.1.1 Baseball Field

One area identified for potential to house an alternative is the baseball field. This field makes up the eastern portion of the park and provides an available footprint of roughly 1.01 acres as shown in **Figure 2-5**. The field is currently utilized by Pasadena American Little League and consists of a dirt infield with a turf outfield. Perimeter fencing along N Sierra Bonita Avenue and Monte Vista Street encloses the field, providing a defined and controlled area for play.

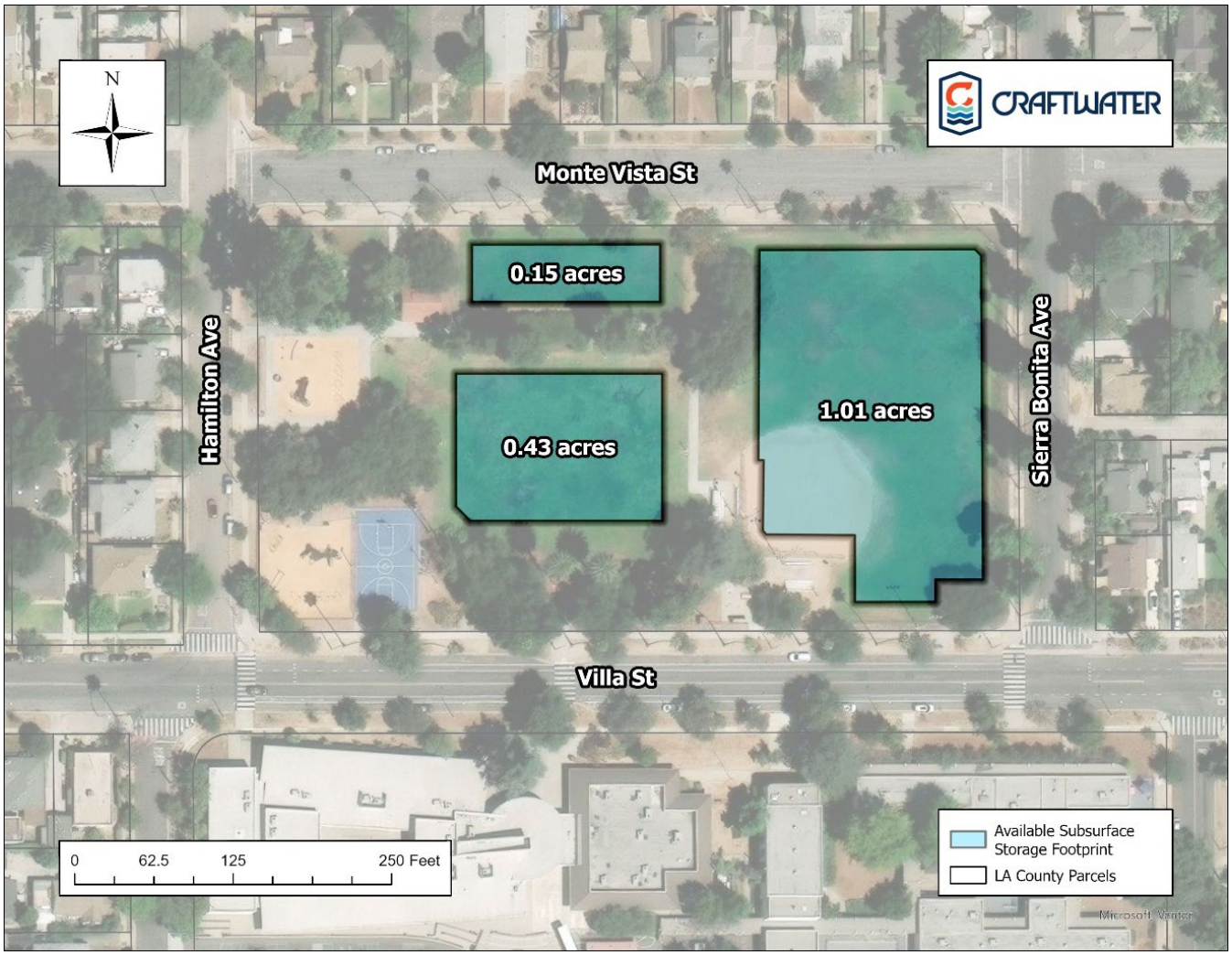


Figure 2-5. *Delineation of Available Footprints for Potential Subsurface Stormwater Storage Galleries and Diversion Facilities.*

The LACFCD-owned Rubio Wash Storm Drain runs along Sierra Bonita Avenue, bordering the site to the east on Sierra Bonita Ave. Due to the close proximity of this facility and the drain invert elevations being higher than the proposed BMP invert, a gravity-operated diversion line can sufficiently convey flows to the BMP without the need for pumping. The diversion would originate south of the intersection of Monte Vista Street and N Sierra Bonita Avenue, travel west into the park, before bending south and discharging into the proposed BMP within the field.

This configuration has potential to introduce conflicts with mature tree root systems located within the parkway. To avoid impacts to these trees, which are designated to be protected in place, the selected alignment avoids routing through this area. This configuration also limits construction impacts primarily to the eastern portion of the park, allowing the remainder of the park to remain operational during construction.

2.3.1.2 Restroom Adjacent

The lawn area immediately east of the restroom was also evaluated as a potential BMP location. This site provides a limited footprint of approximately 0.15 acres for subsurface infrastructure.

The primary constraint at this location is the small available footprint, as infiltration-based systems rely on the bottom surface area of the facility to discharge flows into the underlying soils. A smaller footprint therefore results in reduced infiltration capacity and lower outflow rates, requiring the system to rely more heavily on higher infiltration rates to maintain adequate drawdown and storage availability. Based on these constraints, this location is not considered viable at this time.

2.3.1.3 Multi-use Field

The multi-use field was another area identified during early investigations. Located in the center of the park, this area provides approximately 0.43 acres of available footprint and functions as an open, flexible green space without a defined or dedicated use. While the available footprint is moderate, the central location of this site introduces significant constraints related to park access and circulation during construction.

The proposed diversion serving this alternative would extend west along Monte Vista Street before turning south into the park and continuing through the multi-use field. Installation of both the diversion pipeline and the subsurface BMP would require trenching through the park and the establishment of temporary fencing around active work areas. Together, these activities would create a north-south construction corridor extending through much of the park, in addition to the closure of the multi-use field itself. This corridor would intersect key pedestrian routes and result in the temporary closure of east-west walkways across the central portion of the park as well as along the northern half of the site.

Although east-west access would remain available along the southern edge of the park near Villa Street, users traveling between the east and west sides of the park would no longer be able to take direct routes through the interior. Instead, pedestrians would be required to detour to the southern perimeter walkway before crossing and returning north to their destination. This rerouting disrupts the existing circulation pattern, which currently allows for multiple direct crossings and a continuous internal walking loop, and would reduce connectivity between park amenities during construction.

Concerns regarding these impacts were raised by community members during outreach with particular emphasis on the loss of connectivity and increased disruption compared to other alternatives. Based on these considerations, the Multi-use Field alternative was not advanced for further evaluation.

2.3.2 Local Sites

While Jefferson Park's existing parkways present an opportunity to capture and manage local surface runoff, these were deemed infeasible per the preliminary geotechnical investigation results. However, the parkways still present an opportunity for native landscaping. The parkways line the perimeter of the park and serve as the interface between the park and the surrounding community. Currently, these areas consist primarily of mature trees with limited understory vegetation and minimal groundcover.

A map illustrating the delineated parkway along the park perimeter is provided in **Figure 2-6**.

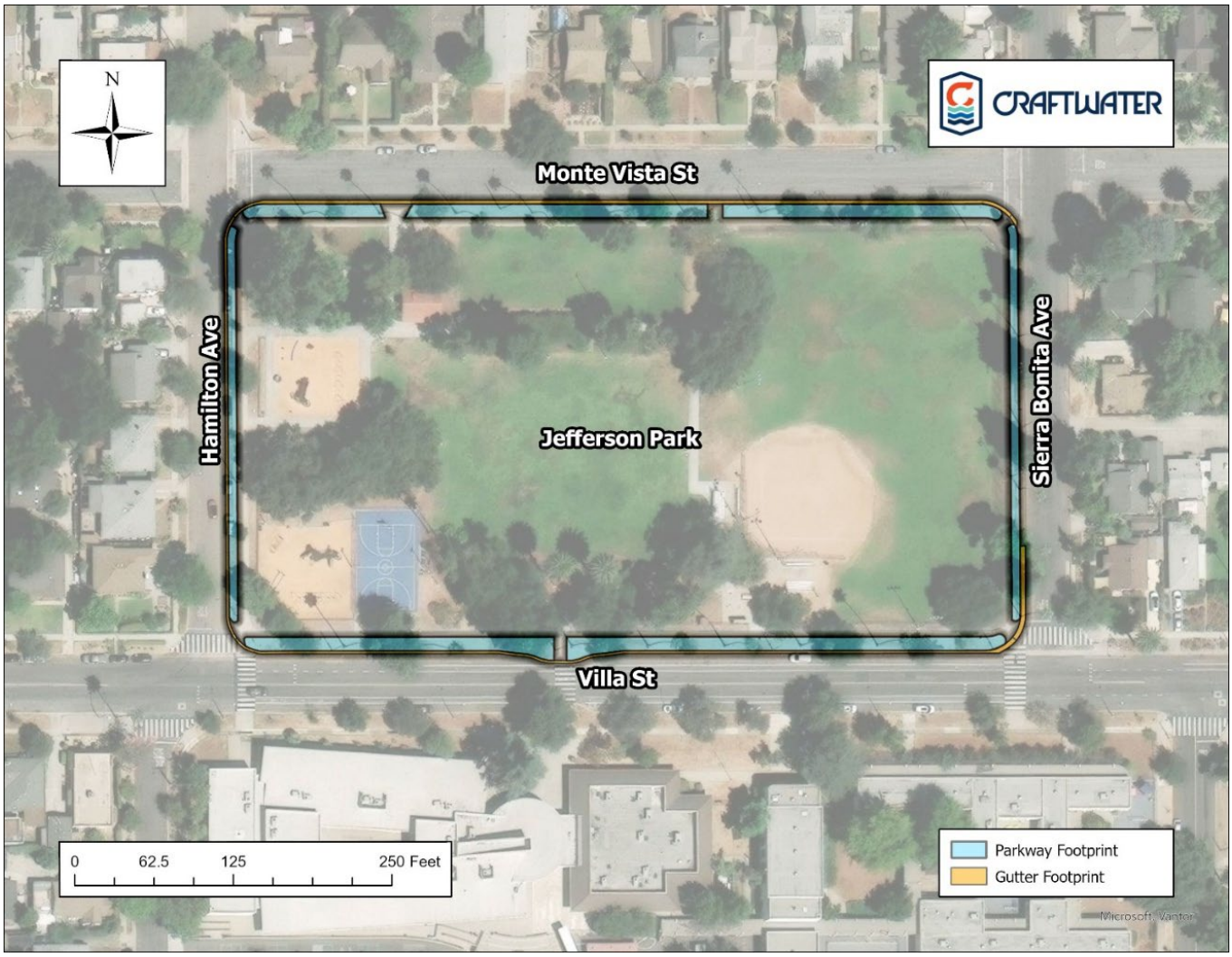


Figure 2-6. *Delineated Parkway and Gutter Footprints.*

The dimensions of the parkways along the four perimeter streets were measured to estimate the available area for potential native landscaping improvements. Length, width, and resulting area are summarized in **Table 2-9**.

Table 2-9. *Parkway Dimensions and Available Area.*

STREET	LENGTH (FT)	WIDTH (FT)	AREA (SQFT)
Monte Vista St	600	11	6,180
Sierra Bonita Ave	310	7	2,201
Villa St	600	11	6,765
Hamilton Ave	310	7	2,169

2.3.3 Regional Diversion Feasibility Assessment

Two regional storm drain systems were initially evaluated for potential diversion to the proposed BMP: Rubio Wash and the BI0189 storm drains. While both systems provide access to substantial upstream drainage areas, feasibility of diversion is governed by a combination of hydraulic constraints, site geometry, constructability considerations, and long-term operations and maintenance requirements. As such, preliminary engineering analysis was conducted to assess whether each system could support a practical and sustainable diversion configuration within the constraints of the Jefferson Park site.

Evaluation of the BI0189 storm drain in Hill Avenue indicates that a gravity diversion is not feasible due to the relatively low invert elevation of the facility and the distance between the proposed diversion point and the BMP location. Conceptual layout of a diversion pipeline approximately 1,000 feet in length (
), designed at a standard slope of 0.5 percent, results in an elevation drop of approximately 5 feet along the alignment. When combined with the existing invert elevation of the storm drain, this condition produces a BMP invert elevation roughly 24 to 28 feet below finished grade as detailed in **Table 2-10**. The deep systems resulting from the BI0189 diversion significantly increase excavation costs and approach the practical limits for routine maintenance access using vacuum truck equipment, which effectively reaches depths of about 30 feet.

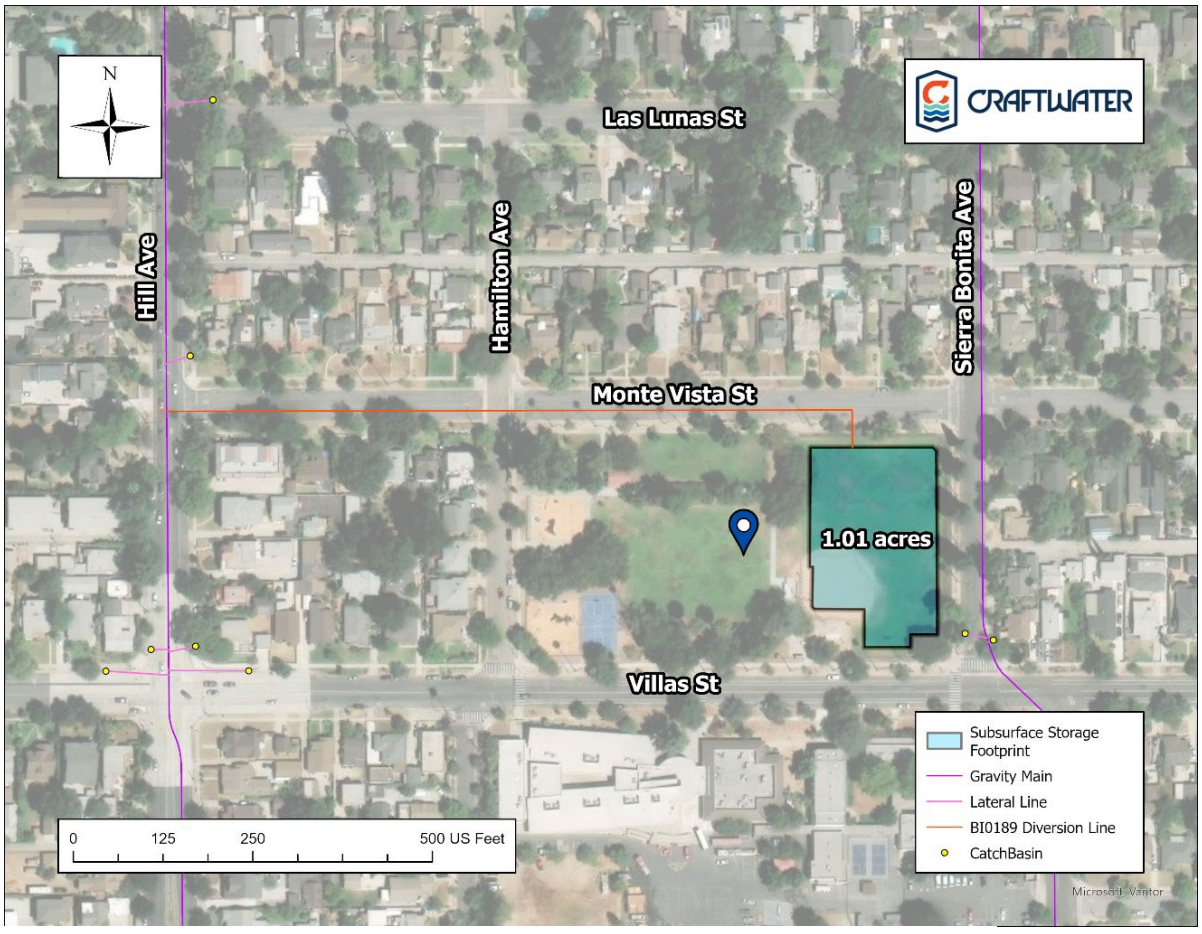


***Figure 2-7**. Diversion Line Capturing Flows from the BI0189 Storm Drain.*

Table 2-10. *Diversion Line Characteristics and Resulting BMP Inver Depth. These Values Represent an Estimate and Will Later be Refined Upon Detailed Site Surveying.*

DRAIN	SURFACE ELEVATION (FT)	DRAIN INVERT DEPTH (FT)	DIVERSION LENGTH (FT)	BMP INVERT DEPTH (FT)
Rubio Wash	846	7.5	158	15-19
BI0189	843	12.6	1,007	24-28

Although diversion from BI0189 could be achieved through incorporation of a pump station to lift flows to a suitable elevation, reliance on pumping introduces additional considerations and expenses for both capital and operations/maintenance. Pump systems require higher capital investment, energy consumption, and mechanical maintenance, inherently reducing system reliability due to dependence on electrical and mechanical components. In contrast, passive systems that operate under gravity are generally preferred for stormwater capture applications because they provide greater long-term reliability, lower operations and maintenance burden, and improved resilience during storm events. Given these factors, and the City of Pasadena's preference to minimize operational complexity and long-term maintenance burden, a gravity-driven diversion approach is favored over a pumped system for this project.

In comparison, Rubio Wash Storm Drain is located immediately adjacent to the project site and presents favorable hydraulic conditions for a gravity diversion. The relative invert elevation of the facility allows stormwater to be conveyed to the proposed BMP with minimal elevation loss over a short alignment, resulting in a substantially shallower system configuration. This condition supports a more constructible design and ensures that the facility remains within practical limits for long-term maintenance access.

Based on these considerations, the BI0189 diversion alternative, including both gravity and pumped configurations, was not advanced for further evaluation. Subsequent alternatives analysis and performance modeling were therefore focused on diversion of flows from Rubio Wash Storm Drain, which provides a more practical, reliable, and maintainable solution within the constraints of the project site.

3 REGIONAL BMP ANALYSIS AND PERFORMANCE MODELING

3.1 Water Quality Optimization Strategy

The primary water quality goal of the Jefferson Park Stormwater Capture project is to reduce the long-term annual loading of pollutants to Rubio Wash. Zinc was used as the primary limiting pollutant of interest following the Reasonable Assurance Analysis (RAA) conducted in the 2021 update of the ULAR WMP. Lead was considered a secondary pollutant for optimization. To ensure that the diversion, storage, and discharge system will be sized to maximize load reductions in a cost-effective manner, optimization modeling was performed.

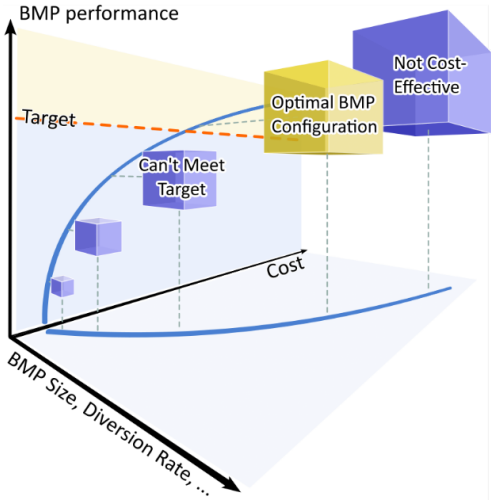


Figure 3-1. *Conceptual illustration of optimization modeling balancing various design components to maximize performance.*

The purpose of optimization modeling is to balance design components (including storage footprint, storage depth, inflow diversion rates) such that no one component limits the performance of the system subject to potential discharge options (**Figure 3-1**). Optimization supports decision making throughout the design process by guiding selection of the most cost-effective system design.

The model setup for water quality simulation and optimization is complex, involving several modeling systems and iterative feedback from design engineers. The general methodology is discussed below, and the results are presented thereafter.

3.2 Performance Modeling and Optimization Methods

The first step of the modeling is to predict water quality performance for a range of 1) diversion rates, 2) discharge alternatives and rates, and 3) storage sizes. Different combinations of these parameters will lead to different performance statistics, including runoff capture and pollutant capture. These model inputs and outputs are discussed in this section.

A custom Best Management Practice (BMP) model was used to improve upon certain modeling limitations in EPA's System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN). This custom model is grounded in the physical BMP representations (stage-storage, stage-discharge) used in SUSTAIN, and it provides built-in optimization algorithms to more systematically automate the process of evaluating many different BMP configurations to select an effective solution related to project goals. The model processes 10 years of runoff and pollutant loading time-series data generated by the Loading Simulation Program in C++ (LSPC) at an hourly time step for Water Years 2009 to 2018. For each potential BMP configuration, the

hourly inflow, storage, and outflow of stormwater and pollutants were simulated. The simulated performance statistics of all the BMP configurations are then evaluated against the objectives in **Section 1.1** and considerations in **Section 2.2.1** to develop the optimal project configuration alternatives.

Figure 3-2 presents a concept profile schematic for the general offsite alternative design, depicting the key components modeled from a runoff and pollutant mass balance perspective, except for the pretreatment unit which will be examined in future studies.

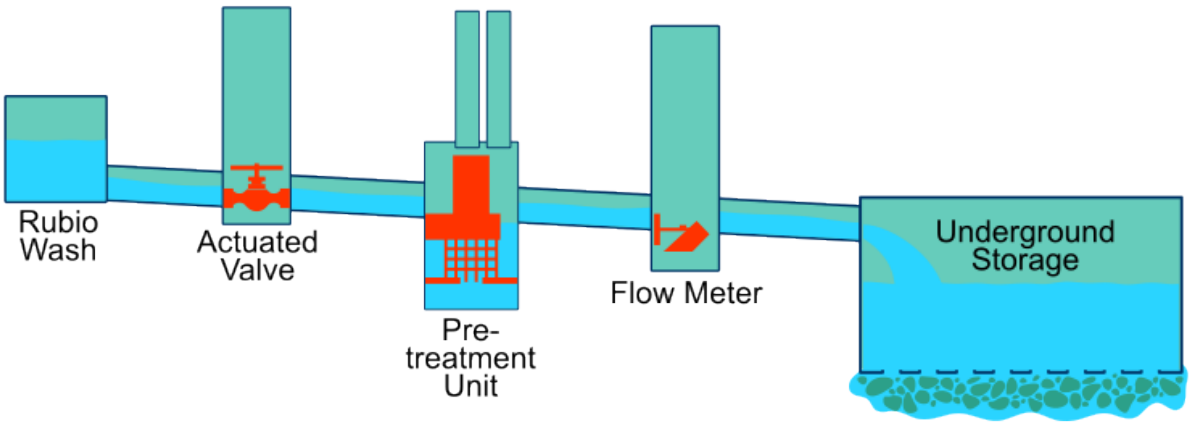


Figure 3-2. *Concept Profile of the Gravity Fed Infiltration Facility.*

3.2.1 Model Input–Diversion Rates

Model runs were limited to feasible diversion ranges for the proposed diversion point based on prior project knowledge related to the drainage area and potential project sizes. For this project, a gravity diversion approach is proposed in accordance with the City of Pasadena's preference to minimize operational complexity and long-term maintenance requirements associated with pumped systems.

A gravity diversion requires the structural control measure to be located below the existing storm drain conveyance invert to allow stormwater to flow into the facility by gravity. In this case, a gravity diversion from Rubio Wash Storm Drain is feasible due to the favorable invert elevation at the diversion point and its relatively close proximity to the proposed BMP location. Based on typical design considerations, practical gravity diversion rates are generally limited to approximately 40 cfs.

3.2.2 Model Input–Discharge Alternatives

According to the preliminary geotechnical results (detailed in **Section 2.2**), infiltration at a depth of 15–20 ½ feet is viable given that the infiltration rate exceeds the minimum required rate of 0.3 inches per hour, as specified by the Los Angeles County Administration Guidelines for Geotechnical Investigation of Low Impact Development Stormwater Infiltration manual. The desktop infiltration rate used for modeling of the subsurface infiltration gallery was the average of Borings B1 and B2, which is 0.775 in/hr.

3.2.3 Model Input–Storage Volume

Commonly available storage depths of 6 and 10 ft were modeled. Modeling was carried out for a BMP footprint ranging from 0.01 acres to the maximum BMP footprint of 0.81 acres at 0.01-acre increments. The footprint range was selected based on the available space within the proposed site, as detailed in **Section 2.3.1**.

3.2.4 Model Output–Long-Term BMP Performance Statistics

Hourly inflow and outflow of runoff and pollutant loads can be generated by the custom model for each BMP configuration. This output timeseries data can be summarized into the following performance statistics that are relevant to the project objectives for BMP optimization:

- **Average Annual Runoff Reduction**: The average amount of stormwater treated from the storm drain system per year.
- **Average Annual Pollutant Load Reduction**: The average mass of pollutants the BMP can remove through treatment per year.

3.2.5 Model Output–85th Percentile 24-Hour Design Storm Performance Statistics

A separate spreadsheet model was developed to verify whether a combination of diversion rate, treatment rate, and storage volume can fully capture the peak flow and total runoff volume from an 85th percentile 24-hour design storm.

3.2.6 Model Output–90th Percentile Bacteria Storm Performance Statistics

A separate spreadsheet model was developed to verify the portion of the bacteria storm that could be retained or infiltrated via the BMP.

3.2.7 Model Output–Cost and Cost-Effectiveness

The model generates a planning-level cost, which is comprised of an estimated construction cost for each configuration. It's important to note that the costs included in the modeling do not include costs associated with surface improvements. These planning level costs are provided for each of the alternatives in the subsequent sections in **Section 3.3**. These costs were then weighed against various configuration parameters such as but not limited to footprint size, percent pollutant removal, pollutant removal magnitude, and percent capture of the 85th percentile design storm to select the most cost-effective configuration.

3.3 Optimization Modeling Results

3.3.1 85th Percentile Design Storm Capture

Modeling results indicate that full capture of the 85th percentile, 24-hour design storm is achievable within the available footprint constraints of the baseball field. A configuration consisting of a 10 ft storage depth and diversion at the peak flow rate of 22.36 cfs requires approximately 0.873 ac of footprint and results in an estimated planning-level cost of $21.0 million (Alternative 1). This footprint is below the maximum feasible BMP footprint of

approximately 1.01 ac identified for the site, confirming that full capture of the design storm is feasible without exceeding spatial limitations.

The minimum footprint needed to satisfy full capture of the 85th percentile, 24-hour design storm for a configuration consisting of a 6 ft storage depth would need to be 1.34 acres, which exceeds the available footprint of the baseball field.

3.3.2 BMP Optimization for Long-Term Performance

The following analysis takes into account the project objectives described in **Section 1.1**. While an alternative capable of fully capturing the 85th percentile design storm has been identified at an estimated cost of $20.9 million, this section will evaluate additional configurations that optimize long-term performance by balancing cost-effectiveness, achieving at least 80% removal of divertible zinc loading on an annual average basis based on the WMMS2 model, and maximizing the volume captured from the 85th percentile design storm.

Because a portion of the drainage area contributing flows to Rubio Wash Storm Drain was impacted by the Eaton Fire, both the baseline WMMS 2.0 model and the Eaton Fire Effects Model, detailed in the previously submitted **Existing Hydrology Report**, were used to evaluate system performance under both pre- and post-fire conditions. This approach ensures that the selected alternative reflects current watershed conditions and provides a conservative basis for design.

3.3.2.1 Baseline WMMS 2.0 Model Results

The baseline WMMS 2.0 model was used to generate performance relationships across a range of BMP configurations, with each data point representing a unique combination of diversion rate, storage depth, and footprint. The resulting plots illustrate long-term average annual zinc reduction as a function of storage footprint under varying ponding depths (**Figure 3-3**), as well as the marginal cost associated with incremental increases in zinc removal performance (**Figure 3-4**).

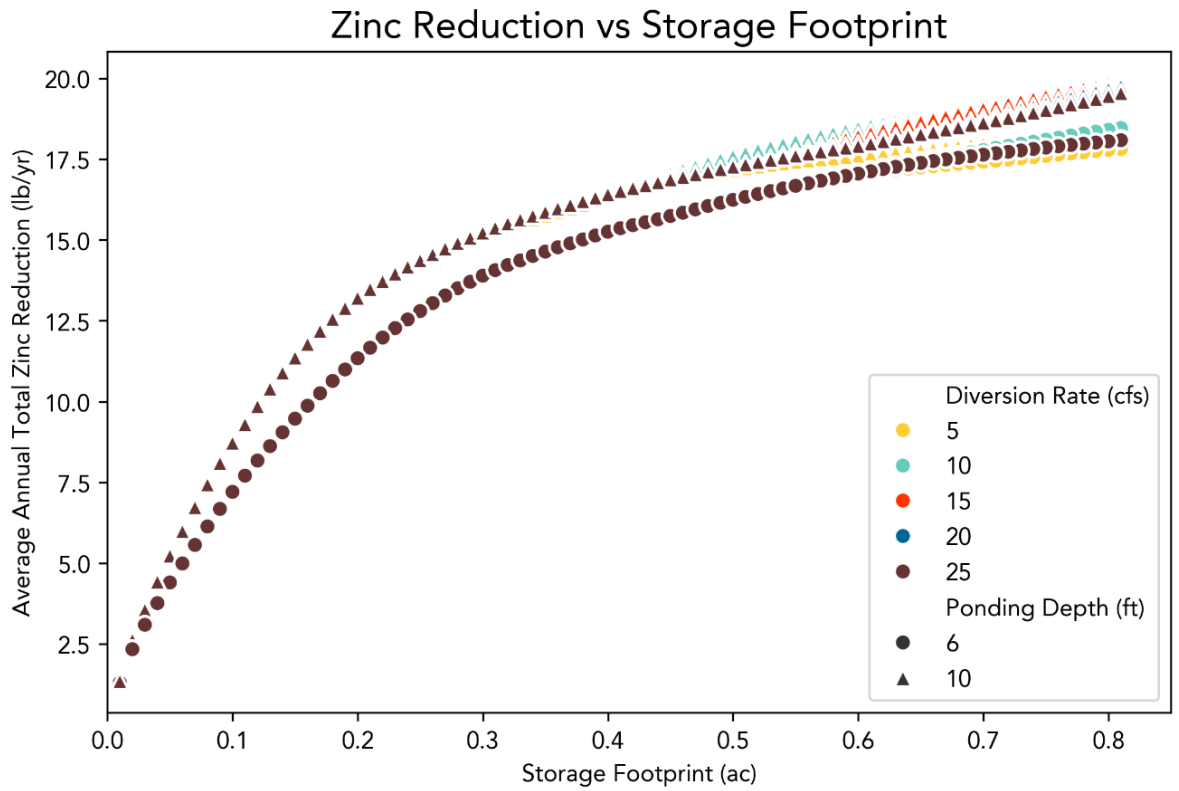


***Figure 3-3**. Long-Term Average Annual Zinc Reduction vs. BMP Footprint (Baseline WMMS 2.0 Model)*

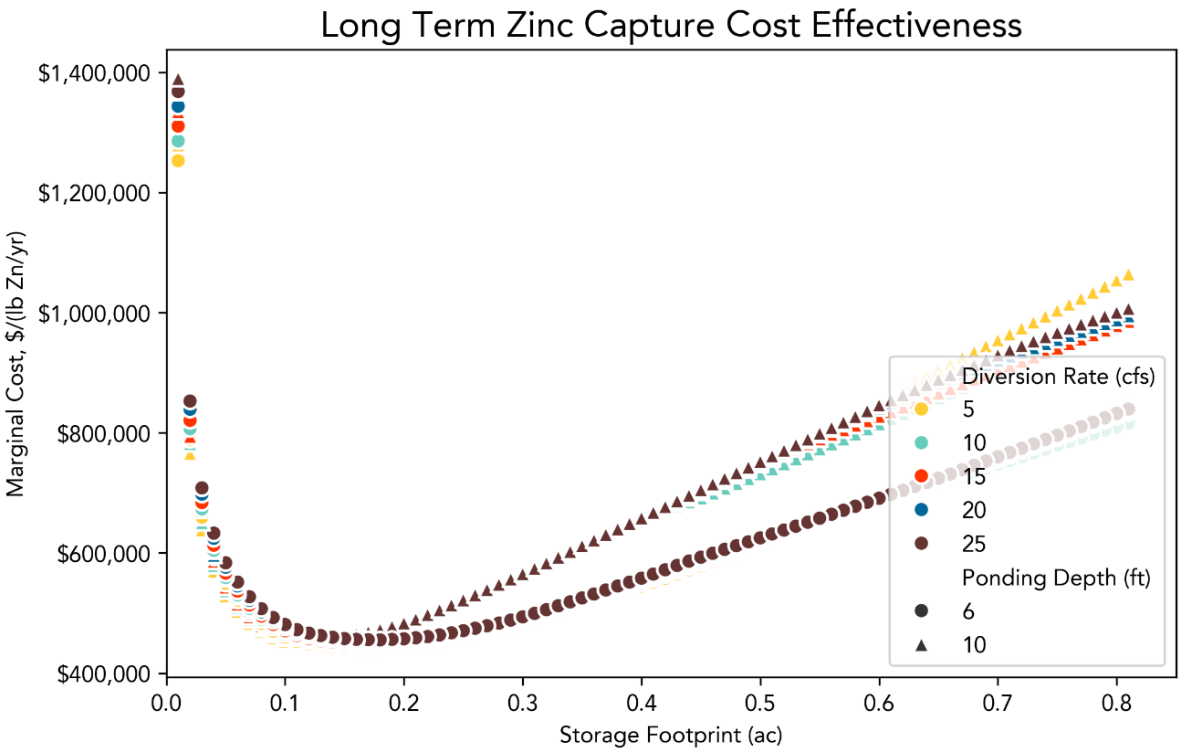


***Figure 3-4**. Marginal Cost of Zinc Reduction vs. BMP Footprint (Baseline WMMS 2.0 Model)*

3.3.2.2 Wildfire Impacts on BMP Performance

A parallel evaluation was conducted using the Eaton Fire Effects Model to assess how post-fire conditions influence BMP performance. Similar to the baseline model, the results are presented as relationships between zinc reduction and footprint under varying ponding depths (**Figure 3-5**), along with corresponding cost-performance trends (**Figure 3-6**).

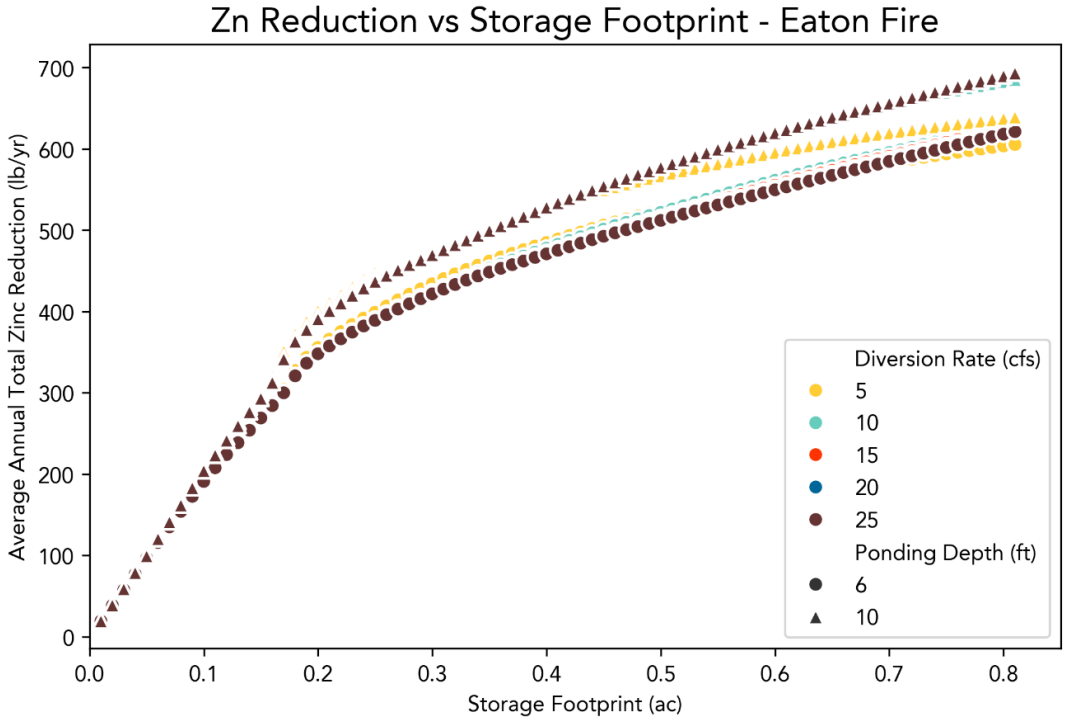


***Figure 3-5**. Long-Term Average Annual Zinc Reduction vs. BMP Footprint (Eaton Fire Effects Model)*

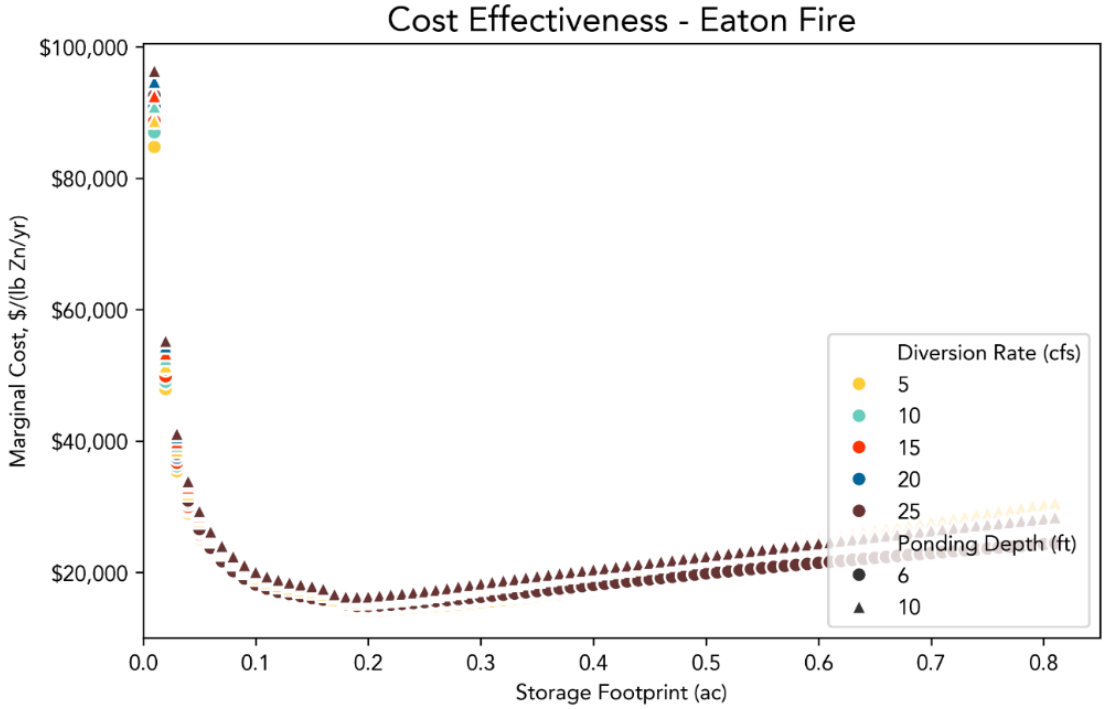


***Figure 3-6**. Marginal Cost of Zinc Reduction vs. BMP Footprint (Eaton Fire Effects Model)*

Comparison of the baseline and fire-affected model outputs indicates that while overall trends remain consistent, the Eaton Fire Effects Model predicts significantly higher zinc loading.

3.3.2.3 Screening of Design Parameters

Several observations can be made from the plots presented in the previous section:

- **Diversion Rate**: While there is a noticeable increase in the magnitude of zinc load that can be treated when comparing 5 cfs to 10 cfs, the benefits diminish when choosing a diversion rate beyond 10 cfs. Given this diminishing return, the additional investment required to accommodate a higher diversion rate is not justified. Therefore, a **diversion rate of 10 cfs is recommended**.
- **Ponding Depth**: The cost-effectiveness and zinc reduction results indicate competing trends between ponding depths. Configurations with a 10 ft ponding depth achieve slightly greater zinc reduction for a given footprint, while 6 ft systems demonstrate improved cost-effectiveness based on marginal cost per unit of zinc removed. Due to this tradeoff, neither depth provides a clear standalone advantage, necessitating a more comprehensive evaluation approach that balances both performance and cost.
- **Footprint**: The zinc reduction results from both the WMMS2 and Eaton Fire watershed models show that increasing storage footprint generally improves annual zinc reduction performance, with larger footprints and deeper ponding depths providing greater treatment benefits. In both models, the 10-ft ponding depth configurations consistently outperform the 6-ft configurations due to the increased available storage volume. While the curves begin to show some diminishing returns at larger footprints, there is not a clear point where the curves fully flatten out or where additional footprint stops providing meaningful benefit. This suggests that, from a treatment standpoint, maximizing the feasible storage footprint would likely still be beneficial since the evaluated range does not identify a distinct footprint threshold where performance gains become negligible. Compared to the WMMS2 model, the Eaton Fire watershed model produces substantially higher zinc reduction values and maintains stronger performance gains across the full footprint range, likely due to the increased runoff volumes and pollutant loading associated with post-fire watershed conditions.

3.3.2.4 Weighted Multi-Criteria Evaluation

To support selection of a preferred alternative, a spreadsheet-based evaluation tool was developed to compare configurations using weighted project objectives. This approach allows decision-makers to assign relative importance to key factors, including cost-effectiveness, zinc reduction, and capture of the 85th percentile design storm. Initial weighting prioritized cost-effectiveness (45%), followed by design storm capture volume (30%), and then zinc reduction (25%).

Based on this evaluation, the highest-performing configuration for long-term performance consists of a 10 cfs diversion rate, 10 ft ponding depth, and a footprint of approximately 0.77 ac (Alternative 2). While this configuration reduces capture of the 85th percentile storm to approximately 90%, it still achieves a similar magnitude of zinc reduction (19.45 pounds under the WMMS2 model and 675.0 pounds under the Eaton Fire model on an annual average basis)

and results in a lower estimated cost of $18.7 million, representing a more balanced and cost-effective solution. By comparison, the minimum alternative that can capture the 85th percentile design storm (25 cfs diversion rate, 10 ft ponding depth, and a footprint of approximately 0.873 ac) reduces 20.2 pounds of zinc under the WMMS2 model and 713.2 pounds of zinc under the Eaton Fire model on an annual average basis.

3.3.3 Dry Wells Incorporation

Given the potential for higher infiltration rates associated with sandy soils at depths below the invert elevation of the proposed subsurface infiltration gallery, the benefits and tradeoffs of incorporating dry wells were evaluated for both: (1) the minimum configuration designed to capture the 85th percentile design storm discussed in **Section 3.3.1**, and (2) the balanced configuration described in **Section 3.3.2**.

The conceptual analysis assumes the dry wells would be located within the public right-of-way north of the baseball field and hydraulically connected to the subsurface infiltration gallery. An example dry well detail is provided in **Figure 3-7**. Based on a required spacing of approximately 30 feet on center, it is estimated that up to six dry wells could be accommodated within the available area (depicted in **Section 4**). This layout was used to evaluate performance tradeoffs under assumed infiltration rates ranging from 0.84 in/hr, representing the maximum rate observed in Boring B-2 as discussed in **Section 2.2.3**, to 5 in/hr, reflecting the potential for more permeable sandy soils at greater depths.

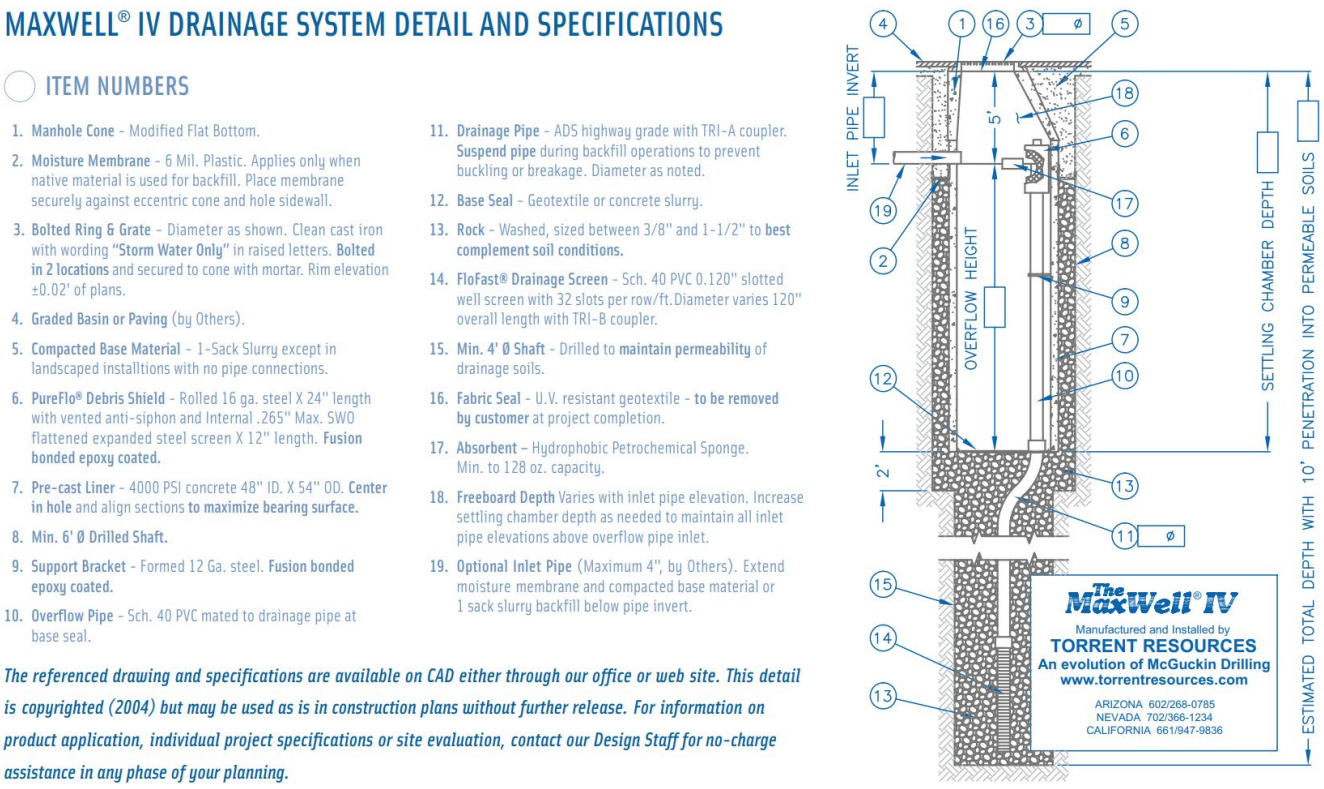


Figure 3-7. *Preliminary schematic for Maxwell IV dry well system.*

The assumed dry well design parameters are listed as follows:

- **Dimensions**: The wells extend 60 feet below the invert of the subsurface infiltration gallery and have a 6-feet diameter (for a total depth at approximately 80 feet). For both systems, the top 10 feet below the invert would provide open space storage while the rest of the column provides gravel-packed storage and infiltration.
- **Material/Voids**: The first 10 feet of the dry well system below the invert of the subsurface infiltration gallery is concrete-lined to prevent infiltration. The bottom 50 feet is filled with gravel with a void ratio of 0.4 and infiltrates through the walls and the bottom.

The storage volume of the dry well in cubic feet is calculated as,

$$\text{Storage Vol.} = \pi R_w^2 H_s + n\pi R_w^2 H_w$$

where R_w is well radius (3') in feet, H_s is sump height (10') in feet, H_w is well height (50') in feet, and n is void ratio (0.4).

The maximum flow rate of the dry well in cubic feet per second is calculated as,

$$\text{Flow Rate} = \frac{I_D(2\pi R_w H_w + \pi R_w^2)}{12 \times 3600}$$

where I_D is design infiltration rate in inch/hour, and H_w is well height (50') in feet.

It should be noted that these calculations represent a simplified screening-level analysis and likely overestimate dry well performance. Typical industry practice assumes that dry well infiltration capacity is directly proportional to the flow-through rate, calculated using the full wetted surface area of the well and the measured vertical infiltration rate, with only minor additional storage benefits attributed to the dry well volume itself. However, this approach may not fully reflect actual field conditions and introduces uncertainty regarding the true infiltration performance of the system. Despite these limitations, the methodology is commonly applied in regional stormwater planning and feasibility evaluations.

The Craftwater team evaluate the methodologies described in the California State Polytechnic University, Pomona scientific study *Evaluation of Infiltration Testing Methods*. The scientific study used the Steady-State Borehole Permeameter (SSBP) method, which is a field-testing approach used to characterize subsurface hydraulic conductivity and more reliably estimate the infiltration capacity of full-scale dry wells. The test maintains steady-state conditions within a borehole for approximately six hours and uses the measured hydraulic head and flow rate to estimate the hydraulic conductivity of the surrounding soils. Unlike conventional borehole methods that primarily report an infiltration rate specific to the geometry and conditions of the test boring, SSBP accounts for the dynamics of unsaturated flow and produces a soil hydraulic conductivity value that can be used to predict dry well performance under different well dimensions, sandpack intervals, and operating heads. **For design purposes, the SSBP methodology and associated equations can be applied to field testing results to estimate the bulk hydraulic conductivity of the subsurface soils, which can then be used to establish or update the design infiltration rate assumed for the proposed dry wells.**

For purposes of stormwater capture modeling, the dry wells were represented using an **equivalent storage-based configuration** rather than explicitly modeling the detailed geometry and hydraulics of each dry well. The equivalent configuration was developed considering both the maximum rate at which runoff could be supplied to and infiltrated by the dry wells and the equivalent depth of water within the upstream storage vault that could be processed by the dry wells. Because the equivalent storage depth was relatively small (generally less than approximately 6 inches to 1 foot), the analysis assumed that sufficient runoff could be supplied to keep the dry wells full during operation and that conveyance to the dry wells would not limit their infiltration capacity. **Accordingly, the dry well design equivalent was represented as storage only, with the modeled storage footprint and infiltration rate modified to reflect the equivalent storage volume and SSBP-derived design infiltration capacity of the proposed dry wells.** This approach allows the modeled storage element to approximate the hydraulic performance of the dry wells without explicitly representing individual well geometry.

3.3.3.1 85th Percentile Design Storm Capture (Just Dry Wells)

To evaluate the extent to which dry wells alone could capture runoff from the Rubio Wash Storm Drain drainage area, analysis estimated the minimum number of dry wells required to fully retain the 85th percentile design storm without the use of a subsurface infiltration gallery. The evaluation also considered infiltration rates ranging from 5 to 10 in/hr to assess conditions in which highly permeable sandy soils may be encountered at depth. **Table 3-1** displays the number of dry wells needed to capture the 85th percentile design storm under various infiltration rates. Based on the available dimensions of the baseball field, it is estimated that only approximately 60 dry wells could be accommodated within the site footprint. Under this configuration, achieving full capture of the 85th percentile design storm would likely require infiltration rates of at least 7 in/hr at depths of approximately 70 to 80 feet below ground surface.

***Table 3-1**. Number of dry wells to capture the 85th percentile design storm for various infiltration rates.*

ASSUMED INFILTRATION RATE (IN/HR)	NUMBER OF DRY WELLS
0.84	188
1.50	138
2.00	118
3.00	95
4.00	81
5.00	72
6.00	65
7.00	59
8.00	54
9.00	51
10.00	47

3.3.3.2 85th Percentile Design Storm Capture (Dry Wells & Subsurface Infiltration Gallery)

For both Alternatives 1 and 2, an evaluation was performed to determine the extent to which the baseline subsurface storage volume for each alternative could be reduced through the addition of six dry wells operating at varying infiltration rates. For Alternative 1, the baseline configuration was sized to fully capture the 85th percentile design storm, while the baseline configuration for Alternative 2 was sized to capture approximately 90% of the 85th percentile design storm volume.

For each assumed dry well infiltration rate, the required subsurface storage volume was iteratively reduced to evaluate the resulting changes in project cost and zinc reduction performance under both the WMMS2 and Eaton Fire modeling scenarios. The analysis considered dry well infiltration rates of 0.84 in/hr, 2 in/hr, and 5 in/hr. The results and changes to costs and pollutant load reduction can be seen for Alternative 1 in **Table 3-2** and Alternative 2 in **Table 3-3**.

***Table 3-2**. Alternative 1 reduction in subsurface storage volume and corresponding results with six dry wells under varying assumed infiltration rates.*

DESIGN INFILTRATION RATE (IN/HR)	REDUCED STORAGE FOOTPRINT (AC)	COST ($M)	AVERAGE ANNUAL ZINC REDUCTION (WMMS2) (LB)	AVERAGE ANNUAL ZINC REDUCTION (EATON FIRE EFFECTS) (LB)
0.84	0.848	$20.4 (**-$0.58**)	20.51 (**+0.28**)	728.83 (**+15.62**)
2.00	0.818	$19.7 (**-$1.35**)	20.73 (**+0.49**)	743.36 (**+30.15**)
5.00	0.755	$18.2 (**-$2.81**)	20.79 (**+0.55**)	764.61 (**+51.40**)

***Table 3-3**. Alternative 2 reduction in subsurface storage volume and corresponding results with six dry wells under varying assumed infiltration rates.*

DESIGN INFILTRATION RATE (IN/HR)	REDUCED STORAGE FOOTPRINT (AC)	COST ($M)	AVERAGE ANNUAL ZINC REDUCTION (WMMS2) (LB)	AVERAGE ANNUAL ZINC REDUCTION (EATON FIRE EFFECTS) (LB)
0.84	0.746	$18.1 (**-$0.58**)	19.83 (**+0.39**)	693.96 (**+18.92**)
2.00	0.715	$17.4 (**-$1.33**)	20.06 (**+0.61**)	709.66 (**+34.62**)
5.00	0.654	$16.0 (**-$2.76**)	20.26 (**+0.82**)	732.00 (**+56.96**)

3.3.4 90th Percentile Bacteria Storm Capture

All configurations evaluated are capable of capturing the 90th percentile bacteria design storm. As this storm represents the regulatory critical condition for bacteria compliance, the results indicate that the proposed BMP alternatives are sufficient to meet storage targets outlined in the WMP.

3.3.5 BMP Sizing Recommendation Summary

Based on the optimization modeling and evaluation of design parameters, two BMP configurations were identified for consideration. The first configuration is sized to fully capture the 85th percentile, 24-hour design storm within the available footprint. The second configuration is a performance-optimized alternative developed through weighted evaluation of design criteria, including cost-effectiveness, long-term zinc reduction, and capture of the 85th percentile design storm. Key design parameters and performance metrics for both alternatives are summarized in **Table 3-4**. The results with the dry wells can be found in **Section 3.3.3.2**.

***Table 3-4**. Summary of BMP Sizing Alternatives*

PARAMETER	ALTERNATIVE 1	ALTERNATIVE 2
Diversion Rate (cfs)	22.36	10
Ponding Depth (ft)	10	10
Footprint (ac)	0.873	0.77
Storage Volume (ac-ft)	8.73	7.7
Infiltration Rate (in/hr)	0.775	0.775
Zinc Reduction (WMMS2) (lb)	20.23	19.45
Zinc Reduction (Eaton Fire) (lb)	713.21	675.04
85th Capture	100%	88%
Cost ($ Million)	21.0	18.7

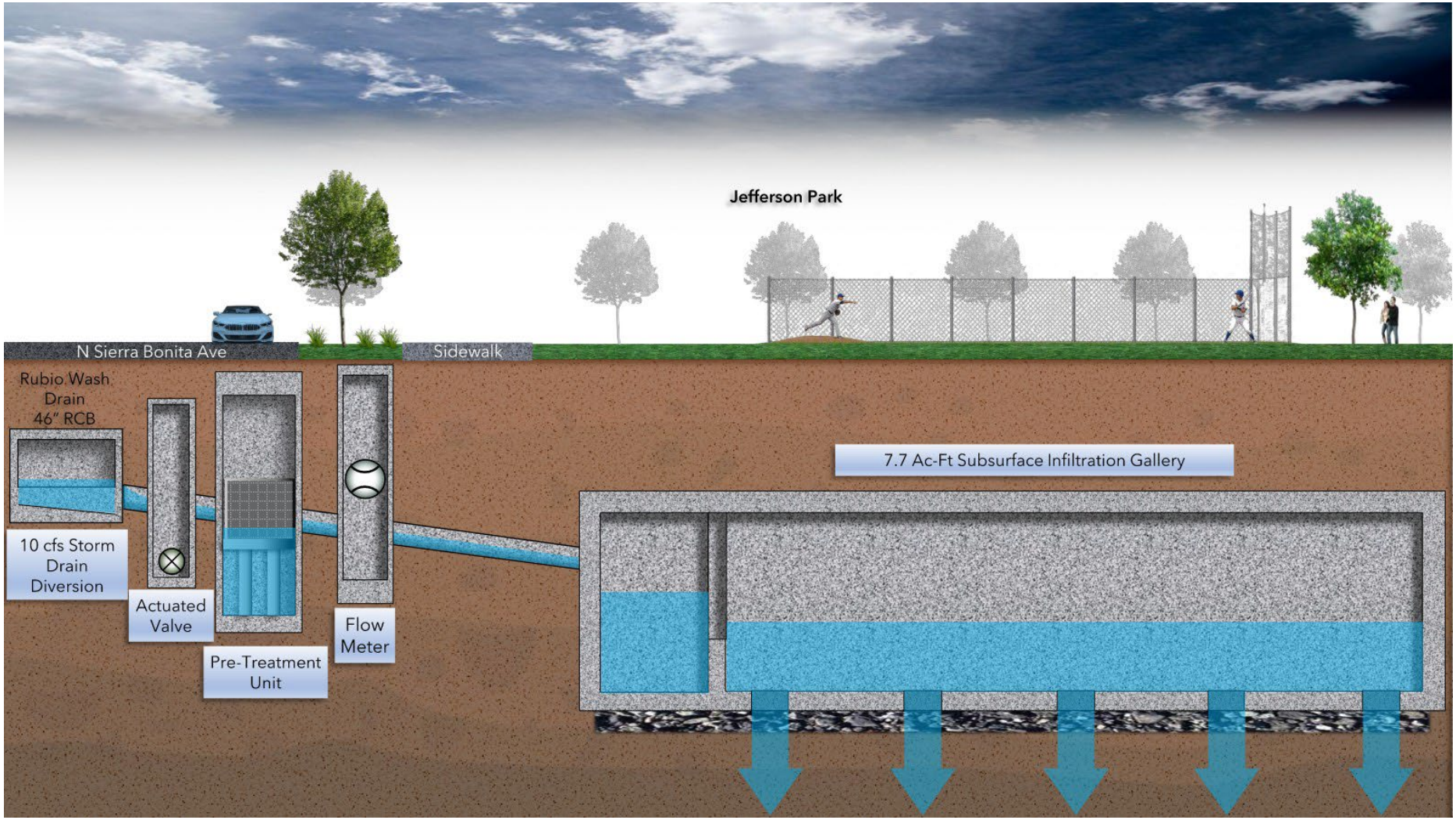
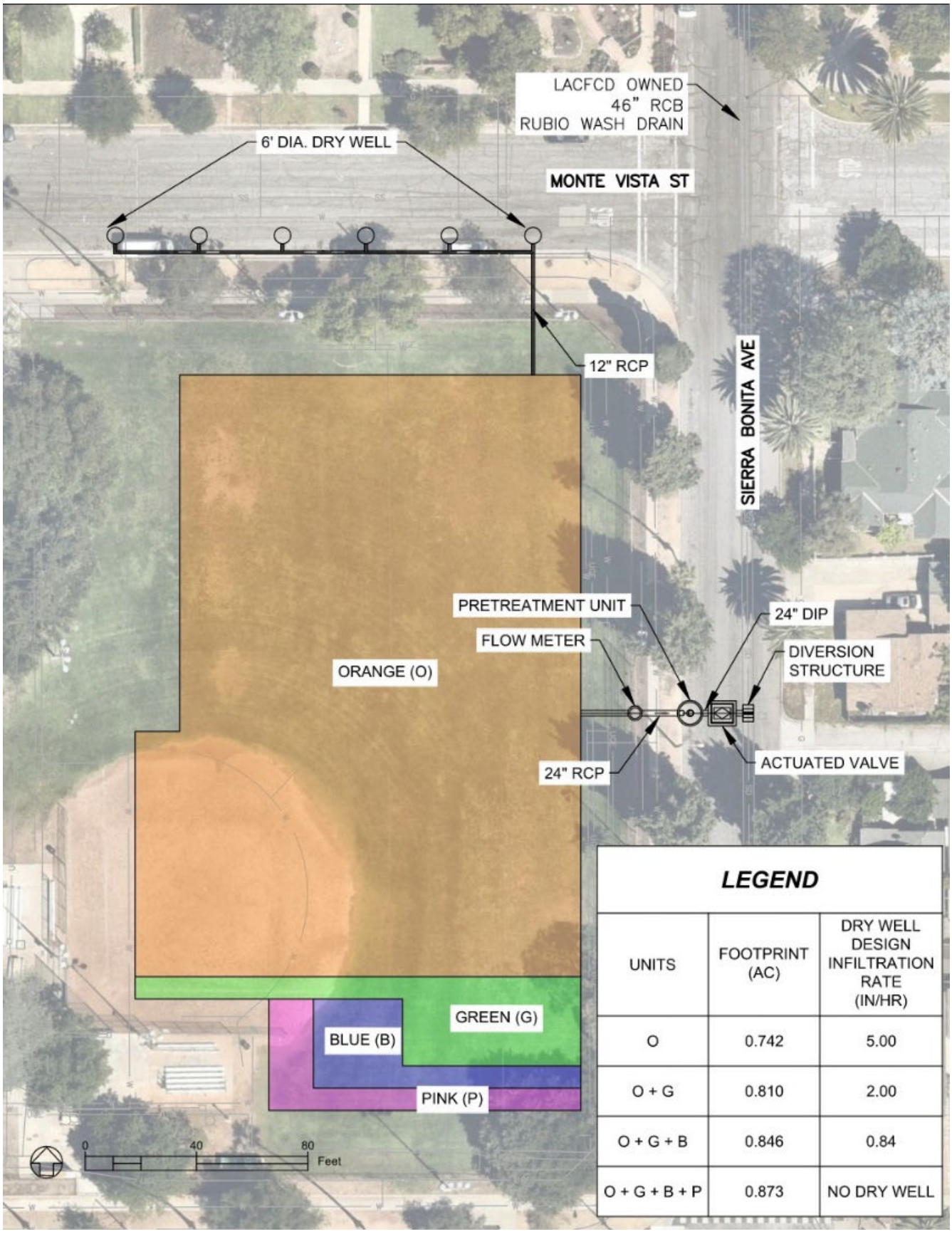


Figure 3-8. *Profile schematic for Alternative 2.*

4 CONCEPT LAYOUT

Concept level layouts were developed to illustrate how each BMP configuration would be implemented within the baseball field, with the alternative capable of capturing the entire 85th percentile storm presented in **Figure 4-1** and the optimized long term BMP shown in **Figure 4-2**. Both alternatives utilize the same diversion point from Rubio Wash Storm Drain and convey flows west into the park, where diversion and pretreatment infrastructure would be located near the eastern edge of the field prior to discharge into the subsurface storage gallery. The different colors represent the reduction in required project footprint resulting from the addition of six dry wells under varying assumed dry well infiltration rates. It is important to note that the baseline scenario for each alternative without the addition of six dry wells is represented by the "O + G + B + P" footprint.

Both configurations would require substantial excavation and temporary closure of the baseball field during construction. Excavation would necessitate full removal and reconstruction of existing field surfaces. Following construction, the field would be restored and returned to its existing recreational use, with the BMP functioning entirely below grade. The reduced footprint associated with Alternative 2 may offer minor benefits in terms of construction duration, material quantities, and restoration effort, but both alternatives represent similar overall levels of site disruption.



UNITS	FOOTPRINT (AC)	DRY WELL DESIGN INFILTRATION RATE (IN/HR)
O	0.742	5.00
O + G	0.810	2.00
O + G + B	0.846	0.84
O + G + B + P	0.873	NO DRY WELL

Figure 4-1. *Concept Layout Depicting Alternative 1.*

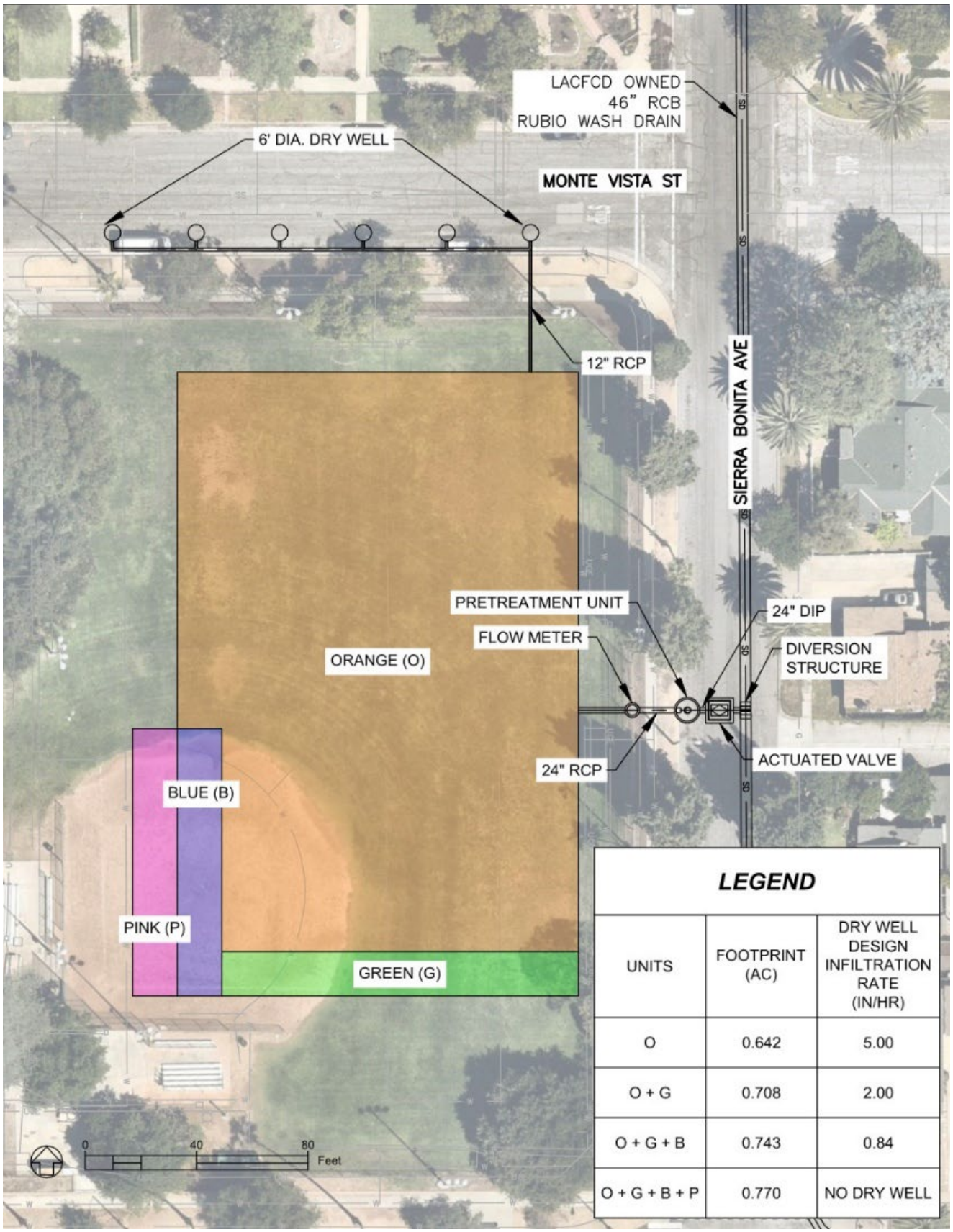


***Figure 4-2**. Concept Layout Depicting Alternative 2.*

5 CONCLUSION AND NEXT STEPS

The two configurations developed through the optimization process represent distinct design approaches rather than incremental variations of a single solution. A summary of key design parameters and performance metrics is provided in **Table 5-1**. Alternative 1 is configured to fully capture the 85th percentile, 24-hour design storm and reflects a capture-based design objective, while Alternative 2 is optimized for long-term performance, balancing pollutant reduction and cost-effectiveness.

***Table 5-1**. Comparison of BMP Alternatives*

CATEGORY	ALT 1 – FULL CAPTURE	ALT 2 – OPTIMIZED
Primary Objective	Maximize 85th capture	Maximize long-term performance
Zinc Reduction[3]	Lower	Higher
Storm Capture	100%	88%
Cost	Higher	Lower
Design Approach	Design storm driven	Balanced approach considering multiple objectives
Strength	Full compliance with design storm	Cost-efficient pollutant removal
Limitation	Higher cost, diminishing returns	Does not fully capture 85th

As shown in **Table 5-1**, Alternative 1 achieves complete storm capture but at a higher cost and lower zinc removal efficiency. Alternative 2 provides improved zinc reduction and reduced cost while maintaining substantial storm capture. As a result, selection of a preferred alternative is not solely a technical determination, but a decision that reflects project priorities.

Based on the project's primary water quality driver of zinc reduction, as well as the emphasis on cost-effective performance within the Safe Clean Water Program framework, Alternative 2 is considered the more efficient configuration for advancement. However, Alternative 1 remains a viable option if maximizing capture of the 85th percentile design storm is determined to be the overriding project objective.

The next phase of work will include advancement of the selected alternative into feasibility-level design, including refinement of diversion and BMP layout and development of more detailed cost estimates.

[3] Based on SCW criteria.

6 REFERENCES

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