Exhibit 5

Kiddie Beach Bacteria TMDL Reduction Feasibility Analysis

May 2023

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List of Abbreviations

A	Amps
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
BEP	Best Efficiency Point
C	Hazen Williams Friction Coefficient
CDP	Coastal Development Permit
CEQA	California Environmental Quality Act
CIBCSD	Channel Islands Beach Community Services District
CSWRCB	California State Water Resources Control Board
CWA	Clean Water Act
DOSH	Department of Safety and Health
EA	Environmental Assessment
EIR	Environmental Impact Report
FM	Force Main
FPS	Feet per Second
GPD	Gallons per Day
GPM	Gallons per Minute
HOA	Hand-Off-Auto
HP	Horsepower
IS/MND	Initial Study/ Mitigated Negative Declaration
kVA	Kilovolt Amps
LARWQCB	Los Angeles Regional Water Quality Control Board
LS	Lift Station
MCC	Motor Control Center
MGD	Million Gallons per Day
MTS	Manual Transfer Switch
NMFS	National Marine Fisheries Service
NPDES	National Pollutant Discharge Elimination System
NSSCW	Northerly Silver Strand Community Watershed
OIT	Operator Interface Terminal
OSHA	Occupational Safety and Health Administration
PDWF	Peak Dry Weather Flow
PHMSA	Pipeline and Hazardous Materials Safety Administration
PLC	Primary Logic Controller
POC	Point of Connection
PVC	Polyvinyl Chloride
PW	Potable Water
ROW	Right of Way
RPM	Revolutions per Minute
RWQCB	Regional Water Quality Control Board
SCAQMD	South Coast Air Quality Management District
SNPS	San Nicholas Pump Station
SS	Sanitary Sewer
SSCW	Silver Strand Community Watershed
TMDL	Total Maximum Daily Load
UPS	Uninterruptable Power Supply
USEPA	United States Environmental Protection Agency
USFWS	U.S. Fish and Wildlife Service
V	Volts
VPS	Voorhees Wastewater Pump Station
WQBELs	Water Quality-Based Effluent Limitations
WW	Wastewater
WWPS	Wastewater Pump Station
WWTP	Wastewater Treatment Plant

1 Executive Summary

1.1 Project Summary

The County of Ventura (County) is the governing faction over the County unincorporated Silver Strand Community Watershed (SSCW) and its County storm drain system. This consists of 108 acres (approximately 0.17 square miles) of low-lying residential neighborhoods adjacent to the Pacific Ocean, South from Channel Island Harbor. The northernmost subwatershed, accounting for approximately 33% of the entire SSCW, drains to the ocean near Kiddie Beach. In November 2007, the Regional Water Quality Control Board (RWQCB) Los Angeles Region (Los Angeles Water Board) adopted the Bacteria Total Maximum Daily Load (TMDL) assigning water quality limits at both Kiddie and Hobie Beaches for the County, Ventura County Watershed Protection District (VCPWA-WP), and other responsible parties to meet during dry and wet weather. In the 2021 Regional Municipal Stormwater (MS4) Permit, the effective date for both dry and wet weather compliance is September 11, 2021.

VCPWA-WP operates the three pump stations discharging stormwater runoff from the County storm drain system to the adjacent beaches, including Kiddie Beach, which is subject to the Bacteria TMDL. To comply with dry weather TMDL requirements, VCPWA-WP installed a dry weather diversion system in the San Nicholas Pump Station (SNPS), which currently discharges low flows of 70 GPM to the Channel Islands Community Services District (CIBCSD) sewer system for treatment by the City of Oxnard Wastewater Treatment Plant. However, during wet weather, defined as precipitation of 0.1” or greater within 24 hours and for 72 hours after the precipitation, the stormwater discharge onto Kiddie Beach contributes to the bacteria exceedances.

The feasibility study objective is to identify viable project concepts to reduce or eliminate bacteria loading induced by storm events from entering Kiddie Beach. For this feasibility study, the goal of the joint effort by the County and VCPWA-WP is to ensure that the 85th percentile of storm flows are diverted from release into the ocean for MS4 Permit and TMDL compliance. Storm volumes were obtained from 20 years of data on a local rain gauge (Oxnard Airport 168) within the City of Oxnard, near the Oxnard Airport; flowrates were obtained via the rational method, as defined by the Ventura County Hydrology Manual and Technical Guidance Manual for Stormwater Quality Control Measures. Section 4.1.3 provides further information on the rational method.

1.2 Alternatives Evaluated

The County hired PACE to evaluate viable alternatives for compliance with TMDL requirements. A total of nine (9) alternatives were analyzed for their ability to reduce or eliminate stormwater discharges from the County storm drain system via VCPWA-WP’s SNPS from entering Kiddie Beach. These alternatives are listed as follows, with a brief description of what each entails:

- Store and Divert to CIBCSD Sewer: Runoff reaching the SNPS would be diverted to an underground storage tank/wet well, where it could be held until demand on the Channel Islands sewer collection system is reduced. At that time, the stored water would be diverted to the CIBCSD in a manner that would minimize the impact on the existing sewer system.
- Pump, Store, and Percolate (US Navy ROW): Runoff reaching the SNPS would be diverted to an additional pump station and valve vault within the nearby parking lot. This station would then pump the stormwater to an open plot of land parallel to Panama Drive owned by the United States (US) Navy, where it could be held for percolation and evaporation. The land will be built up to have two separate 3-foot-high berms extending approximately 1,650 feet in the perimeter for each berm.

- Divert to Sewer (Replace CIBCSD Pump Station & Force Main): Runoff reaching the SNPS would be diverted to an additional pump station and valve vault within the nearby parking lot. The pump station would be capable of diverting flows to the CIBCSD at the flow rates needed to meet the bacteria TMDL limits of the MS4 Permit. Sewer infrastructure located downstream of the San Nicolas Pump Station would also require upgrades to accommodate the increased load on the system.
- Divert to Sewer (Lift Station 29 on Patterson Road): Runoff reaching the SNPS would be diverted to an additional pump station and valve vault within the nearby parking lot. The pump station would bypass the smaller sewer lift stations and gravity pipes to pump directly into a trunk line located on Patterson.
- Store and Treat for Off-Site Reuse: Runoff reaching the SNPS would be diverted to a storage facility, where the water would then be treated to acceptable "purple pipe" or Title 22 water standards. After treatment, the water would be diverted to an open space where it could be utilized for irrigation or other alternate use.
- Treat for Off-site Reuse: Runoff reaching the SNPS would be treated in-line with acceptable "purple pipe" or Title 22 water standards. After treatment, the water would be diverted to an open space where it could be utilized for irrigation or other alternate use. This option would require a higher rate of treatment than the "Store and Treat for Off-Site Reuse" option, since no storage facility is involved.
- Treat for Release: Runoff reaching the SNPS would be treated in line. After treatment, the water would be released through the existing San Nicholas pump station. This option would have a higher rate of treatment than the "Store and Treat for Off-Site Reuse" option since no storage facility is involved.
- Divert to Santa Paula Pump Station: Runoff reaching the SNPS would be diverted to the Santa Paula Pump Station instead of the CIBCSD sewer system. The diverted runoff would be conveyed to a nearby existing storm drain collection inlet to the Santa Paula ocean outfall, which has some excess capacity during storm events and where the receiving waters are far more turbulent than at the existing San Nicolas ocean outfall. Pollutants associated with the diverted runoff would become mixed/diluted at the Santa Paula Pump Station outfall, whereas pollutants reaching the more stagnant waters at the SNPS remain more concentrated.
- San Nicolas Pump Station Ocean Outfall: Runoff reaching the SNPS would remain unchanged, however the existing outfall would be modified. The existing ocean outfall outlets to a particularly stagnant area of water, resulting in a concentrated area of pollutants. If the outfall were relocated to an area where the receiving water is more turbulent, the discharged pollutants would be better able to disperse and be diluted.

A more extensive description and analysis for each of these alternatives are listed in sections 7 through 15 below.

1.3 Feasibility Analysis

Initially, PACE incorporated the usage of an eight (8) criterion ranking matrix to evaluate the feasibility of each alternative, which included analyzing: the capital cost, the 50-year life cycle cost, the cost per acre-foot per year, the TMDL reduction performance, the anticipated public agency/ regulatory board support, the likelihood of additional regulatory requirements, the impact on the public and the foreseeable issues pertaining to the constructability.

Since VCPWA-WP aims to compare alternatives at the same 85th percentile of storm flows, the criterion of “cost per acre-foot per year” was removed. The County recommended an additional matrix for “Operations & Maintenance,” which is incorporated herein. A weighted score was applied per criterion to score each alternative based on anticipated importance. Note that this implies that an alternative with the lowest capital cost does not necessarily imply favorability with respect to the other criteria.

The score per criterion was calculated by analyzing all foreseeable issues for each respective criterion. A summary defining each of these criteria, as well as their respective ranking weights, can be found within section 3.2.1.

1.4 Feasibility Analysis Conclusion

PACE believes the most feasible alternatives reviewed within this analysis is the **Pump, Store, and Percolate (US Navy ROW)** and the **Diversion to Santa Paula Pump Station** options. Both of these alternatives incorporate approximately the same amount of infrastructure improvements, they would require a minimal change in the existing operational procedure and ultimately, both options would ensure that the 85th percentile of storm flows are diverted from Kiddie beach. Furthermore, both options would be significantly easier to construct in comparison to the underground storage options due to the high groundwater in the area of the parking lot adjacent to the SNPS, as discussed in sections 7 and 11. Implementation of an underground storage tank within the adjacent parking lot of the SNPS would have constructability concerns due to the sandy soil profile and shallow groundwater (approximately 5 feet below ground surface) that was measured via temporary groundwater monitoring wells (See section 9 of the technical memorandum in Appendix C). Additionally, since the proposed underground storage system would connect to the existing pump station wet well via a gravity line, the depth of the proposed storage system must be deeper than the existing SNPS, which could be problematic for shoring during construction or keeping the storage system water-tight. Another problem that can arise from an underground storage facility is the interception of existing utility lines that are within the project footprint. The majority of these issues can be avoided, however, if an above-ground storage option is implemented, such as the US Navy ROW option, or the flows are diverted to the Santa Paula Pump Station.

Assessor maps and other parcel data were gathered in order to identify open spaces that might be utilized for one or more of the alternatives that will be evaluated in this feasibility study. One of the locations identified was the section of land between Panama Drive and West Road, within the US Navy right of way. This land provides enough width for the construction of berms to act as percolation basins. There is only one section of the above-ground pipeline in the middle of the available land, which would require the construction of two separate berms unless the pipeline can be relocated. In order to withhold the 85th percentile of storm volumes, the current geometry of the berms is to be 750’ long, 80’ wide, and 3’ high. Out of the nine (9) alternatives covered within Sections 7 through 15, this option appears to be one of the most favorable and feasible.

An equally viable alternative is the diversion to the Santa Paula Pump Station. Given that this option utilizes much of the existing infrastructure, it is the most cost-conservative as well. This option would consist of a force main sending flows to the nearest storm drain in the Santa Paula watershed, along Bardsdale Avenue. The Santa Paula pumps can withstand the additional flows, as the dual pump capacity is approximately 26,000 GPM, as found within the 2014 Silver Strand Deficiency Study conducted by PACE (See Appendix E for excerpts of this study). This is more than 8 times the 85th percentile flow rates for the San Nicholas Watershed. A hydraulic analysis that extrapolated data from the 2014 study showed that the existing 30” storm drain along Bardsdale Avenue and Ocean Drive had sufficient pipe capacity to convey the 85th percentile of flows diverted from the San Nicholas watershed. A description of this process is found in section 14.2.

Several alternatives were eliminated and not ranked because they had inherent flaws or had extremely high costs. A comprehensive breakdown of the feasibility of the remaining alternatives can be seen via the ranking matrix in Table 1-1 below. Note that the number within the parentheses for each alternative references the alternative number that describes the design approach in detail. The section in which each of these alternatives is described in further detail is always 6 more than the alternative number. For example, section 7 describes the design approach for alternative 1. Each alternative was weighed via the criteria listed in the first column. The “total weighted score” is the product of the alternative’s score per criteria and the relative weight per criteria, as discussed in section 3. The score breakdown per criteria for each alternative is discussed in section 16. The total weighted scoring scale varies from a minimum score of 8, to a maximum score of 135. The alternatives are ranked in favorability by their total weighted score in the last row of the table. While the Pump, Store and Percolate (Navy) option has the highest score, some of the considerations that would negatively affect its score were not addressed within Table 3-1 (Criteria Ranking Points System). These considerations include the potential for delays that occur with the legislative processes associated with federal agencies. Since PACE does not believe these concerns affect the feasibility of an alternative, they were not implemented within the overall ranking matrix. That said, to address these concerns, PACE recommends the top two alternatives so the County may ultimately decide between an option that incorporates a federal agency or not.

Table 1-1: 85th Percentile Evaluation Conclusion

Criteria	Weig ht	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
		Weighted Scoring Scale 8-135				
Capital Cost	5	10	20	10	25	5
50 Year Life Cycle Cost	3	9	12	6	15	3
Performance	5	25	25	10	15	20
Public Agency/Regulatory Board Support	5	15	20	5	5	0
Constructability	3	6	9	9	9	3
Public Perception/ Impact	2	10	8	8	8	6
Regulatory Requirements	1	4.0	4.0	2.0	4.0	0.0
Operations & Maintenance	3	15.0	15.0	3.0	15.0	9.0
Total Weighted Score		**94.0**	**113.0**	**53.0**	**96.0**	**46.0**
Overall Rank		**3**	**1**	**4**	**2**	**5**

2 Project Overview

2.1 Project Background & Objectives

The Silver Strand Community Watershed (SSCW) consists of approximately 108 acres (Approximately 0.17 square miles) of the low-lying residential neighborhood adjacent to the Pacific Ocean. Three similar size subareas with three separate stormwater pump stations discharge directly to the adjacent beaches. The pollutants from the existing dry weather flow from the Northerly Silver Strand Community Watershed (NSSCW) are pumped to the existing sewer system through a diversion system at the San Nicholas Pump Station (SNPS) by Kiddie Beach. The existing sewer conveyance system is owned and operated by the Channel Islands Beach Community Services District (CIBCSD). Sewage is directed to the City of Port Hueneme and the City of Oxnard's Sanitary Sewer Collection System, eventually reaching the Oxnard Wastewater Treatment Plant (WWTP). Since the dry weather flow pollution being diverted is insufficient in reducing the bacteria pollution from being released to the ocean, the project objective for this sub-tributary area is to identify viable concepts to reduce or eliminate bacteria load from being discharged to Kiddie Beach via the SNPS.

2.2 Project Team

PACE worked as the primary consultant to the County to prepare this Kiddie Beach Bacteria TMDL Reduction Feasibility Analysis. In addition, PACE collaborated with the CIBCSD to receive information regarding their existing sewer system.

The PACE primary contact is:

- Project Manager: Duncan Lee, P.E.

The County/VCPWA-WP primary contact is:

- Project Manager: Hayley Luna, P.E.

2.3 MS4 Permit

The current MS4 Permit applicable to discharges at Kiddie Beach is Order R4-2021-0105, adopted July 23, 2021, effective on September 11, 2021, and expiring on September 11, 2026. The permit includes discharge prohibitions and requirements to meet TMDL requirements. The water quality limits dictated by the TMDL applicable to the study area are listed in Attachment L of the MS4 Permit and are also summarized in **Table 2-1** below.

Table 2-1: MS4 TMDL Effluent Limitations

Pollutant	Applicable TMDL Effluent Limitations (MPN or cfu)	
	Daily Maximum	Geometric Mean
Bacteria (Total Coliform)	10,000/100 mL	1,000/100 mL
Bacteria (Fecal Coliform)	400/100 mL	200/100 mL
Bacteria (Enterococcus)	104/100 mL	35/10 mL

Retention of stormwater runoff volume from the County storm drain system up to and including the 85th percentile, 24-hour rain event for the drainage area tributary to the Kiddie Beach will be considered compliance with the Kiddie Beach Bacteria TMDL. As such, the study will focus on the 85th percentile storm event as the preferred design condition when analyzing alternative methods for achieving bacteria TMDL compliance in the Kiddie Beach watershed.

2.4 Regulatory Requirements and Permits

The purpose of this section is to identify potential permit requirements prior to the start of construction. Before the start of construction, the Contractor will be responsible for obtaining all required permits. The following is a list of potential Federal, State, and local agencies that should or may have permitting requirements:

- California Coastal Commission
- California Department Fish and Wildlife
- National Marine Fisheries Service (NMFS)
- U.S. Fish and Wildlife Service (USFWS)
- U.S. Army Corps of Engineers
- Los Angeles Regional Water Quality Control Board (LARWQCB)
- Channel Islands Beach Community Services District (CIBCSD)
- Ventura County Public Works Agency (VCPWA)
- VCPWA-Watershed Protection (VCPWA-WP), and
- City of Oxnard.

3 Feasibility Criterion

3.1 Feasibility Ranking Criterion

As previously mentioned under Section 1.3 above, the following eight (8) criterions are used to rank the Kiddie Beach alternatives:

- Capital Cost
- Life Cycle Cost
- Performance
- Public Agency / Regulatory Board Support
- Regulatory Requirements
- Public Perception and Impact
- Constructability
- Operations & Maintenance

3.2 Alternative Review and Ranking

In order to select the best complete TMDL reduction system for Kiddie Beach, PACE, in partnership with VCPWA-WP, will review and evaluate each alternative. Each alternative was rated on a weighted scale, with the highest score indicating the most feasible. Please note that while cost is an important criterion, the evaluation may result in a selection of an alternative that does not have the lowest capital cost or life cycle cost. Criteria definition and weight are provided below. The numerical scoring/ranking for the various criterion within each alternative was calculated by foreseeable considerations necessary for the implementation of each alternative (See section 16). Each alternative has a written description of some of these considerations within their respective subsection, which can be found within sections 7 through 15.

*Note that in the original technical memorandum (See Appendix D), a criterion was considered for the "Cost / Acre-Foot / Year" as it was defined initially with the intention to highlight alternatives that may be more cost-efficient solutions to TMDL requirements at lower storm percentiles. However, since the existing MS4 permit stipulates much reduced annual monitoring requirements if the proposed improvement is designed to address 85^{th} percentile storm flows/volume, the VCPWA-WP directed PACE to only evaluate potential alternatives at the 85^{th} percentile. Therefore, the "Cost / Acre-Foot / Year" criterion will not be helpful in finding an optimal alternative and was removed from the ranking matrix. Subsequently, the weight for "Capital Cost" was increased from "4" to "5".

As suggested by the County, an additional criterion was considered "Operations & Maintenance," which is included at the bottom of this list. Note that this addition was included after the original technical memorandum was submitted, so these changes will only be reflected within this study.

3.2.1 <u>Criteria Definitions and Requirements (Criterion Weight in Parenthesis)</u>

*Capital Cost (5): The total cost to construct the proposed alternative. The score for this criterion per alternative will be evaluated via the range of values listed in A1-A5, in Table 3-1 below. The evaluation of capital costs will encompass purchasing all equipment, land, material, and labor associated with the installation of an alternative.

50-Year Life Cycle Cost (3): The evaluation of the combination of capital cost, operation, maintenance over the expected useful life of an alternative in today's value. Automation, equipment hours of operation, operation man-hour requirements, required scheduled maintenance, maintenance requirements, and reliability will be factored into the life cycle cost. This is calculated in two (2) parts (in present value), consisting of 50 years of operation and maintenance cost and the capital replacement cost after 50 years. PACE assumed the annual cost of

	operation and maintenance to be 3% of the total capital cost. 50 times such value equates to the operation and maintenance portion of this analysis. The next step is to calculate the cost of replacing such a system after it has reached its useful life. For instance, if a pipe has a capital cost of $1,000,000 with a useful life of 80 years, the capital portion of the 50 Year life cycle cost will be [$1,000,000 x (50/80) = $625,000]. The operation and maintenance portion of the cost will be [$1,000,000 x 3% x 50 years = $1,500,000]. The sum of these two figures would be the 50 Year life cycle cost.
Performance (5):	Effectiveness of the alternative to reduce bacteria TMDL as defined in the MS4 Permit. The alternatives will be analyzed at various flow rates to determine if they can meet the design requirements specified. Meeting higher flow rates will benefit an alternative's performance.
Public Agency / Regulatory Board Support (5):	The support of public agencies and regulatory boards for an alternative. Depending on the location of project, the California Coastal Commission may have a large impact on the feasibility of an alternative.
Regulatory Requirements (1):	Low need for an EIR, CEQA Analysis, and involvement of the California Coastal Commission will be scored favorably.
Public Perception / Impact (2):	Support of an alternative. The public perception of a project allows for residents who are impacted to feel that their needs are addressed. Alternatives that minimize the impact on the public will be scored favorably.
Constructability (3):	Feasibility of being able to construct the proposed alternative. The complete system must be able to be constructed at the scale required to meet the bacteria TMDL compliance requirements. The more complex a project is to implement, the lower the alternative will be scored. This includes impacts to existing infrastructure.
Operations and Maintenance (3):	Feasibility of being able to operate and maintain the proposed alternative. The complete system must be maintained to withstand the expected flow rates, and volumes for TMDL compliance requirements. The more stringent the requirements for maintenance crews to undergo, the lower the alternative will be scored.

3.2.2 Criteria Considerations and Scoring

In order to score each of these criteria per alternative, the following list of factors was considered. Note that the total amount of score for each criterion is out of 5, in which the higher the score, the more favorable an alternative is ranked with respect to that criterion. Table 3-1 below lists how ranking points are assigned for each criterion.

Table 3-1: Criteria Ranking Points System

Capital Cost		
	Weight Factor	5
	Description	**Point**
A1	≤ $3,000,000 (+5)	5
A2	>$3,000,000≤ $4,000,000 (+4)	4
A3	>$4,000,000≤ $5,000,000 (+3)	3
A4	>$5,000,000≤ $6,000,000 (+2)	2
A5	>$6,000,000 (+1)	1

50 Year Life Cycle Cost		
	Weight Factor	3
	Description	**Point**
B1	≤ $7,000,000 (+5)	5
B2	>$7,000,000≤ $10,000,000 (+4)	4
B3	>$10,000,000≤ $13,000,000 (+3)	3
B4	>$13,000,000≤ $16,000,000 (+2)	2
B5	>$16,000,000 (+1)	1

Performance		
	Weight Factor	5
	Description	**Point**
C1	Captures/ Eliminates the 85th Percentile from Discharge to Kiddie Beach (+1)	1
C2	Reduces Pollutant Load at Kiddie Beach (+1)	1
C3	Improvements Will Not Discharge to Active Public Areas (+1)	1
C4	Improvements Will Not Require Specialized Operation and Maintenance (+1)	1
C5	Eliminates Pollutant Loads from Discharge (+1)	1

Public Agency/Regulatory Board Support		
	Weight Factor	5
	Description	**Point**
D1	Grant Competitiveness (+2)	2
D2	Potential for Multiple Benefits (e.g., Water Recycling or Groundwater Recharge) (+1)	1
D3	Permitted Under the Counties Local Coastal Plan (LCPs) (+1)	1
D4	Qualified for Categorical Exclusions / Statutory Exemption with The California Coastal Commission (+1)	1

Constructability		
	Weight Factor	3
	Description	**Point**
E1	Additional Third-party Public Infrastructure Improvement (-1)	-1
E2	Pipeline Trenching/ Underground Storage in High Groundwater Environment (-2)	-2
E3	Underwater Construction in the Harbor (-2)	-2

Public Perception/ Impact		
	Weight Factor	2
	Description	**Point**
F1	Construction In the Harbor (-1)	-1
F2	Construction In the Beach (-1)	-1
F3	Loss of Parking (-1)	-1
F4	Full Street Closure During Construction (-1)	-1
F5	Increased Flood Risk (-1)	-1

Regulatory Requirements		
	Weight Factor	1
	Description	**Point**
G1	California Coastal Commission (-1)	-1
G2	California Department Fish and Wildlife (-1)	-1

G3	National Marine Fisheries Service (NMFS) (-1)	-1
G4	U.S. Fish and Wildlife Service (USFWS) (-1)	-1
G5	U.S. Army Corps of Engineers (-0.333)	-0.333
G6	California Environmental Quality Act (CEQA) (-0.333)	-0.333
G7	Regional Water Quality Control Board (RWQCB) (-0.333)	-0.333
Operations & Maintenance		
	Weight Factor	**3**
	Description	**Point**
H1	Infrequent Maintenance (+1)	1
H2	Alternative Does Not Require Replacing Specialized Equipment (+1)	1
H3	Easy Access for Maintenance (+1)	1
H4	Does Not Require Continuous Annual Regulatory Compliance Monitoring (+1)	1
H5	Improvements Will Not Require Specialized Safety Training/ Procedures (+1)	1

4 VCPWA-WP Existing Stormwater Infrastructure

4.1 Stormwater Collection System

4.1.1 Infrastructure

The Silver Strand Beach Community consists of approximately 108 acres of the low-lying, residential neighborhood adjacent to the Pacific Ocean. The watershed is relatively flat, with average slopes ranging from 0.1% to 2.2%. To facilitate stormwater drainage, the tract was developed with saw tooth grading, which creates natural low spots to collect surface water from localized sub-watersheds. The localized sub-watersheds are drained by three networks of underground storm drains, each of which discharges to the ocean via one of three pump stations (San Nicholas, Santa Paula, and Santa Monica).

4.1.2 Watershed Drainage

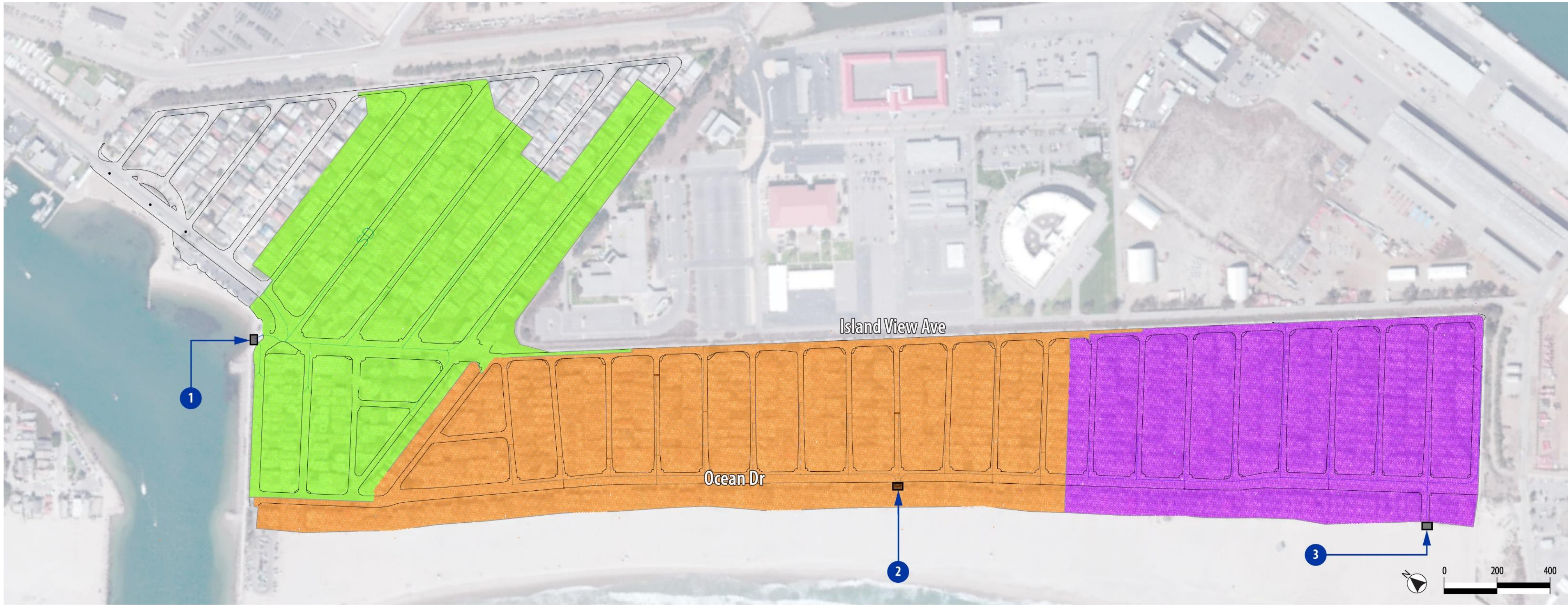
There are 55 catch basin inlets within the SSCW. The distribution of these catch basins within the subwatershed areas is summarized in Table 4-1 below.

Table 4-1: Subwatershed Catch Basin Data

Subwatershed Name	Subwatershed Area	Number of Catch Basin Inlets
San Nicholas	31.1 acres	15
Santa Paula	36.4 acres	21
Santa Monica	26.0 acres	19

The stormwater is conveyed within the three (3) subwatershed by a network of storm drains. The overall watershed map, with each respective sub-watershed, is shown in Figure 4-1 below. For catch basin locations, refer to the 2014 Silver Strand Pump Station Deficiency Study 100-year storm excerpts in Appendix E.

Figure 4-1: Silver Strand Subwatersheds



LEGEND

1. San Nicholas Pump Station (Watershed Area = 31.1 ac)
2. Santa Paula Pump Station (Watershed Area = 36.4 ac)
3. Santa Monica Pump Station (Watershed Area = 26.0 ac)

4.1.3 Flow Determination

To determine the project's wet weather flow into the SNPS, the rational method was utilized in conjunction with the data from the Oxnard Airport rain gauge. The Rational Method and the runoff coefficient are defined within the Ventura County Hydrology Manual and the Technical Guidance Manual for Stormwater Quality Control Measures (See Section 3 from the technical memorandum in Appendix D). Only the SNPS was analyzed for flow determination. Santa Monica Pump Station is outside the local vicinity of Kiddie Beach and provided no beneficial analysis for its subwatershed. However, Santa Paula Pump Station will play a role under one of the project alternatives. Therefore, information about the San Nicholas and Santa Paula Pump Stations is provided below.

4.1.3.1 Rain Gauge Data Analysis

Oxnard Airport's hourly rainfall, recorded from the rain gauge for the last 20 years (10/01/1999 – 10/01/2020), was analyzed to determine the runoff flow rate and the accumulated rainfall runoff per rain event. Independent rain events were defined as any amount of precipitation measured via the Oxnard Airport rain gauge with a minimum of 72 hours between rain events. Since the rain events were largely influenced by low/no precipitation readings within the 72-hour window, a 24-hour analysis was utilized, which redefined an independent rain event as being a minimum of 24 hours between any amount of recorded precipitation. This allowed for the analysis of rainfall data that is less skewed from low cumulative precipitation rain events. In order to determine these values, the rainfall flow rates and accumulated volumes were analyzed at the respective percentiles. Table 4-2 provides the accumulative rain event in inches and the estimated volume of stormwater within the SNPS watershed.

Table 4-2: Accumulated Rainfall per Rain Event (24-hour Analysis)

A =	31.1	acres
C =	0.72	
Rainfall	**Inches**	**MG**
20th Percentile	0.010	0.0060
30th Percentile	0.021	0.0128
40th Percentile	0.088	0.0531
50th Percentile	0.149	0.0898
60th Percentile	0.260	0.1568
70th Percentile	0.450	0.2716
80th Percentile	0.760	0.4583
85th Percentile	0.961	0.5795
90th Percentile	1.239	0.7471
95th Percentile	1.841	1.1101
98th Percentile	2.505	1.5107
99th Percentile	3.666	2.2108
100th Percentile	5.700	3.4372

4.1.3.2 Flow Selection

Table 4-3 summarizes the 20-year analysis and provides a large variety of stormwater flow into the existing station in the form of percentile.

Table 4-3: San Nicholas Influent Flow (24-hour Analysis)

A =	31.1	acres
C =	0.72	
Rainfall	**Flow (cfs)**	**Flow (GPM)**
20th Percentile	0.2	100.5
30th Percentile	0.7	301.5
40th Percentile	1.3	603.0
50th Percentile	2.2	1,005.0
60th Percentile	3.4	1,507.5
70th Percentile	4.4	1,959.8
80th Percentile	6.0	2,713.6
85th Percentile	7.2	3,216.1
90th Percentile	10.1	4,520.6
95th Percentile	12.7	5,708.5
98th Percentile	14.6	6,532.6
99th Percentile	17.7	7,939.7
100th Percentile	65.4	29,346.7

4.2 San Nicholas Pump Station

4.2.1 Mechanical Equipment

The SNPS consists of two (2) 20 HP Cascade axial pumps, one (1) 5 HP WEMCO submersible pump, and one (1) 5 HP Vaughan submersible chopper pump. The pump characteristics are summarized below.

Table 4-4: San Nicholas Pump Characteristics

Description	Make & Model	Design Capacity	Qty
20 HP Main Pumps	Cascade – 20P Propeller Pump (700 RPM)	7600 GPM at 7.5' TDH (± 16.9 CFS x 2)	2
5 HP Sump Pump	WEMCO 4S3 Submersible Pump	450 GPM at 13' TDH (± 1.0 CFS)	1
5 HP Sewer Pump	Vaughan - SE3F Submersible Chopper Pump	120 GPM at 22' TDH (± 0.27 CFS)	1

The addition of the 5 HP sewer pump was a part of an upgrade performed in 2004, in which VCPWA-WP installed a diversion to the sewer collection system owned and operated by the CIBCSD. The upgrades to the station consisted of the following:

- 5 HP Sewer Pump and Associated Appurtenances.
- 3-Inch Sch. 40 PVC Pipe to Sanitary Sewer Manhole.
- Modifications to the Existing Wet Well.
- Associated Electrical and Controls Equipment.

4.2.2 Electrical Equipment

The SNPS is powered by Southern California Edison (SCE), most likely with a 100-amp service (will need to be field verified). The electrical service and motors starters are shown below.

Table 4-5: San Nicholas Electrical

Description	Voltage	Circuit Protection
Service – SCE Meter #256000-164988	480/277V 3 phase	100A
Manual Transfer Switch	480/277V 3 phase	200A
20 HP Main Pumps Motor Starters	480/277V 3 phase	-
5 HP Sump Pump Motor Starter	480/277V 3 phase	-
5 HP Sewer Pump Motor Starter	480/277V 3 phase	-

4.2.3 Instrumentation and Controls

The SNPS level set point control system for the main and sump pumps references an air bubbler controller. The liquid level in the wet well is measured by sensing the backpressure of compressed air that is continuously bubbling through a tube that extends down to the near bottom of the wet well. The system utilizes sensitive pressure sensors to measure the backpressure in terms of liquid level to control the operation of the pumps.

The gravity lines could not be evacuated during pump operation before the lead pump "OFF" control set point was reached. This indicates that the pump capacity exceeds the rate at which stormwater can enter the station.

Table 4-6: San Nicholas Station Control Set Point

Description	Level Set Point
High Water Alarm	120"
Lag Pump ON	85"
Lead Pump ON	80"
Lag Pump OFF	50"
Lead Pump OFF	45"
Sump Pump ON	36"
Sump Pump OFF	12"

The existing sewer diversion pump is run off a control system that is independent of the main and sump pumps. The automatic controls are an intrinsically safe relay housed in a NEMA 7 enclosure that senses the operation of two liquid level switches, a "high switch" turns the pump on, and a "low switch" shuts it off.

The station is outfitted with a basic Remote Telemetry Unit (RTU) that provides alarm dialing and data logging capability. MISSION Model 110 RTU uses wireless communication through cellular digital data networks to transmit data to MISSION's secure website.

4.2.4 Pump Station Discharge and Outfall

The outfall for the SNPS is discharged to the rocky shoreline in the harbor with a "duckbill" check valve system; and, therefore, does not have an outfall structure. This will need to be field verified.

4.2.5 Condition Assessment for Station Reuse or Replacement

4.2.5.1 Building

The existing structure comprises 8-inch CMU walls supported by the cast-in-place concrete wet well below. There is a 5" thick, tapered to 4" thick (for slope), precast concrete slab roof that attaches to the masonry wall with steel angle clips. The precast concrete roof is detachable and removed in two separate pieces in order to gain access to the equipment housed beneath. The existing building appears to be in a fair condition.

4.2.5.2 Wet Well

From a past project evaluation, the fixed manual trash screen was in failed condition, and a repair or replacement of the screens was needed to protect the pumps from future damage. The below-grade structure was found to be in fair condition. This will need to be field verified.

4.3 Santa Paula Pump Station

4.3.1 Mechanical Equipment

The Santa Paula Pump Station consists of two (2) 50 HP Johnston axial pumps and one (1) 5 HP WEMCO submersible pump. The pump characteristics are summarized below.

Table 4-7: Santa Paula Pump Characteristics

Description	Make & Model	Design Capacity	Qty
50hp Main Pumps	Johnston – 20PO Propeller Pump (880rpm)	11,700GPM @ 10.5' tdh	2
5hp Sump Pump	WEMCO 4S3 Submersible Pump	450GPM @ 13' tdh	1

4.3.2 Electrical Equipment

The Santa Paula Pump Station is believed to be a 200-amp service. This will need to be field verified. This capacity is adequate for the horsepower of the pumps installed at the station.

Table 4-8: Santa Paula Electrical

Description	Voltage	Circuit Protection
Service – SCE Meter #256000-063471	480/277V 3 phase	100A
Manual Transfer Switch	480/277V 3 phase	200A
50hp Main Pumps Motor Starters	480/277V 3 phase	-
5hp Sump Pump Motor Starter	480/277V 3 phase	-

4.3.3 Instrumentation and Controls

The Santa Paula Pump Station level set point control system for the main and sump pumps references an air bubbler controller. The liquid level in the wet well is measured by sensing the backpressure of compressed air that is continuously bubbling through a tube that extends down to the near bottom of the wet well. The system utilizes sensitive pressure sensors to measure the backpressure in terms of liquid level to control the operation of the pumps.

The gravity lines were unable to be evacuated during pump operation before the lead pump "OFF" control set point was reached. This indicates that the pump capacity exceeds the rate at which stormwater can enter the station.

The station is outfitted with a basic Remote Telemetry Unit (RTU) that provides alarm dialing and data logging capability. MISSION Model 110 RTU uses wireless communication through cellular digital data networks to transmit data to MISSION's secure website.

Table 4-9: Santa Paula Station Control Set Point

Description	Level Set Point
High Water Alarm	110"
Lag Pump ON	85"
Lead Pump ON	80"
Lag Pump OFF	50"
Lead Pump OFF	45"
Sump Pump ON	36"
Sump Pump OFF	12"

4.3.4 Pump Station Discharge and Outfall

The Santa Paula Pump Station has a large (6-feet wide, 6-feet long, and 10-feet deep) concrete headwall structure for erosion and settlement protection. Santa Paula has a beach outfall with a stainless-steel slide gate that frequently is impacted with sand. The slide gate can be opened and function as intended once the sand has been removed. VCPWA-WP had PACE separately review a preliminary design of an outfall relocation that aimed to address issues with operator safety and mitigate the buildup of sand between storm events. The design relocated the outfall 60 feet upstream to gain an additional foot of head. The outfall was subsequently adjusted to discharge 1 foot higher, thus, keeping the head condition on the pump consistent.

4.3.5 Condition Assessment for Station Reuse or Replacement

4.3.5.1 Building

The existing structure is constructed of 8-inch CMU walls supported by the cast-in-place concrete wet well. There is a 5-inch thick, tapered to 4-inch thick (for slope), precast concrete slab roof that attaches to the masonry wall with steel angle clips. The precast concrete roof is detachable and removed in two separate pieces in order to gain access to the equipment housed beneath.

From a separate past inspection, the building is experiencing various structural issues as follows (this will need to be field verified):

- Cracking throughout CMU walls.
- Precast concrete roof slab is experiencing excessive cracking

4.3.5.2 Wet Well

From a separate past inspection, PACE observed that the fixed manual trash screen was in poor condition and that repair or replacement of the screens is needed to protect the pumps from future damage. The below-grade structure was found to be in fair condition. This will need to be field verified.

5 Feasibility Alternative Outline

5.1 Methodology

5.1.1 Flow and Volume Selection and Analysis

This section will discuss the approach to the analysis of the flow data. In order to accommodate the 85th percentile of storm flows, the following flows and volumes listed in Table 5-1 are goal markers for each of the alternatives. Note that each alternative will be evaluated without the existing diversion system, as the impact is relatively minor, and some alternatives are restricted to the currently emplaced diversion flows of 70 GPM.

Table 5-1: 85th Percentile Flow and Volume Amounts

Condition	Flow (GPM)	Volume (MG)
Without Diversion	3216	0.58
With Diversion	3146	0.57

5.1.2 Typical Components

The majority of the alternatives require a wet well and pump system. Some alternatives require the usage of a temporary storage system to capture the diverted stormwater. The hydrodynamic trash will be utilized in most of the options. See Section 6 for additional information regarding hydrodynamic separators.

5.2 Feasibility Alternatives

The following nine alternatives within Table 5-2 were considered. Note that throughout this study, the section in which each alternative description is found is the alternative number plus six.

Table 5-2: Description of Analyzed Alternatives

Alternative Number	Description	Section number
1	Store and Divert to CIBCSD Sewer	7
2	Pump, Store, and Percolate (US Navy ROW)	8
3	Divert to Sewer (Replace CIBCSD Pump Station & Force Main)	9
4	Divert to Sewer (Lift Station 29 on Patterson Road)	10
5	Store and Treat for Off-Site Reuse	11
6	Treat for Off-Site Reuse	12
7	Treat for Release	13
8	Divert to Santa Paula Pump Station	14
9	San Nicholas Pump Station Ocean Outfall	15

6 Hydrodynamic Separator

6.1 Design Approach

A hydrodynamic separator is a proven method to separate trash, and other floatable debris, from stormwater runoff. Various alternatives discussed in this report will require the use of such trash separators.

6.1.1 Hydrodynamic Trash Separator

According to record drawings, the SNPS has two storm drain inlet pipes that are individually connected to the pump station's wet well (15" and 36"). For the purpose of this concept-level feasibility study, PACE is recommending two separate hydrodynamic trash separators for each of the storm drain inlet pipes to the SNPS, which would be required for seven of the nine alternatives. Another possibility would be combining the 15" line and 36" line with a junction box and having the outlet line lead to a singular hydrodynamic separator. An overflow could be added within the new pump station, with the return line leading back to the SNPS. This would be used for flows exceeding the 85th percentile. For the purpose of this feasibility study, two hydrodynamic separators were implemented within each of the proposed options. The two alternatives that listed the separator units as optional items are those that divert to the sewer collection system without any detention basin:

- Divert to Sewer (Replace CIBCSD Pump Station & Force Main)
- Divert to Sewer (Lift Station 29 on Patterson Road)

While PACE recommends the implementation of a hydrodynamic trash separator, it is intended for floatable debris and will not contribute towards compliance with the bacteria TMDL requirements. If VCPWA-WP decides not to implement trash separators, then any proposed alternatives that have a wet well for a pump station can help eliminate some, but not likely a significant amount, of such debris.

7 Store and Divert to CIBCSD Sewer (Alternative 1)

7.1 Design Approach

The store and divert to CIBCSD approach would redirect incoming water to be stored in a separate equalization storage tank that would allow for a greater accumulation of incoming dry and wet weather flows prior to being pumped at a low flow rate into the adjacent CIBCSD sewer system. This option would allow for a greater volume of rainfall to be sent to the CIBCSD sewers following a respective rain event.

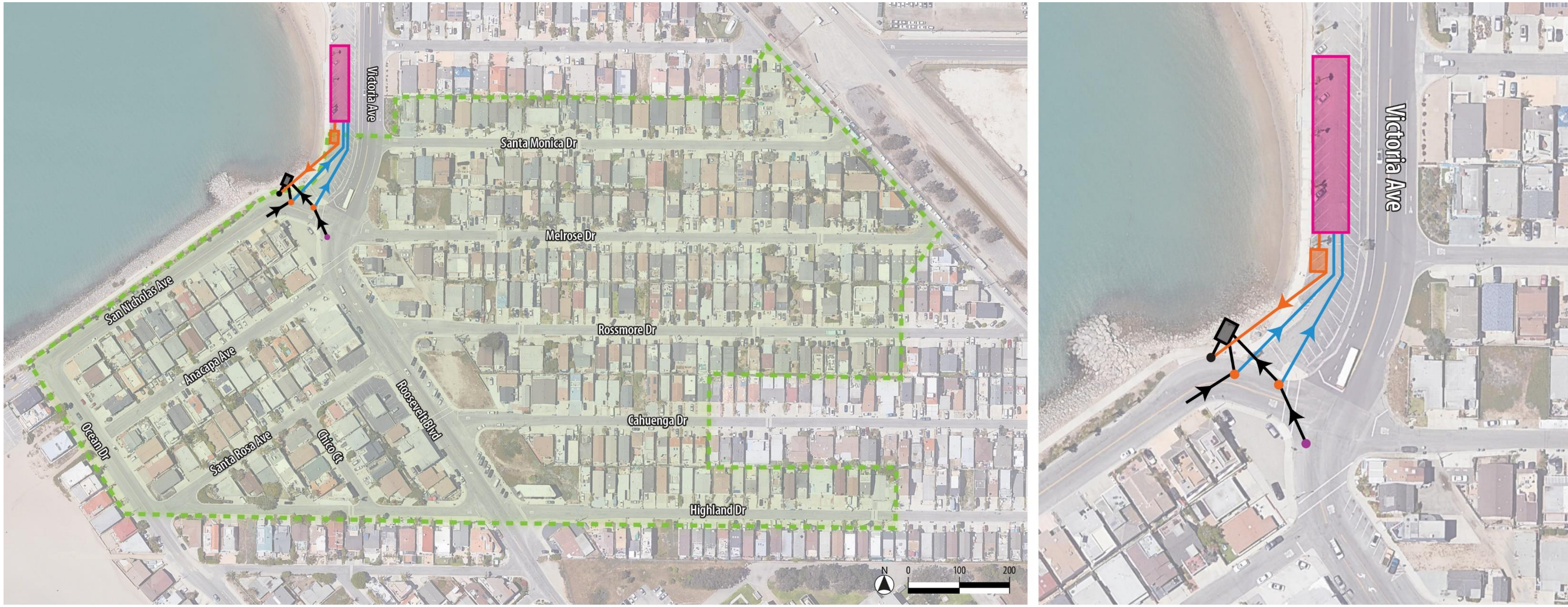
The CIBCSD currently restricts the incoming diversion flows to 70 GPM. Furthermore, stormwater diversion stops entirely once 0.1 inches of rain has fallen within a respective rain event. If this option were to be implemented, a larger volume of stormwater runoff could be captured and slowly diverted to the CIBCSD sewer system while adhering to the original flow restrictions instilled in the existing system.

7.2 Design Criteria

In order to account for wet weather flows within the 85th percentile of storms, a large storage tank will need to be incorporated within the design. The assumption is to use the adjacent parking lot for the location of the storage basin. A few dimensions can be assumed for the sizing. The first is restricting the storage tank's width to that of the parking lot, which is approximately 50 feet. Expanding the tank in the northern direction is possible since it would remain within the designated parking lot. Potential limitations on expanding northernly can be accounted for by increasing the depth of the proposed tank. It is worth noting, however, that the deeper the tank, the more complex the construction would be due to the local soil profile and potential groundwater seepage from shallow groundwater shown from the monitoring well data (See section 9 in Appendix D). Furthermore, since the tank will be connected via a gravity line to the existing wet well, the system will require the tank to be at a lower elevation than the existing SNPS wet well.

The volume can be accounted for by using a large enough storage tank within the extents of the parking lot. However, the flow rate is restricted to the limitations instilled by CIBCSD. Refraining from allowing more than 70 GPM to be released into the sewer system would result in some backlogging between respective 85th percentile storm events, based on the definition given by the MS4 permit, which is 72 hours between rain events. In order to completely evacuate the tank within each respective rain event, an additional 64.1 GPM would need to be accounted for with the volume associated with a filled 85th percentile tank. Currently, with the flow restrictions given by CIBCSD, it would take approximately 138 hours to fully drain at 70 GPM, which is near twice the duration of a storm event as defined by the MS4 permit. Whereas there are several means of mitigating the flow rate, it may be worth considering pumping higher flow into the existing CIBCSD's lift station, where there could be additional conveyance capacity. However, there are design challenges associated with this approach due to the risk of the lift station not having sufficient capacity for the additional flows. Therefore, it is not evaluated at this time. The volume restrictions instilled by CIBCSD of 0.1" of rain would also be problematic, as 85th percentile storm volumes are nearly 10 times that restriction. It is worth noting that two 85th percentile storm events happening subsequently within the time frame required to drain the storage tank is extremely unlikely. To account for that possibility, an overflow could be installed that would drain additional flows to the SNPS, for discharge. This however, would not encapsulate the 85th percentile for the subsequent storm, and thus would affect the feasibility ranking via the criteria listed in Table 3-1. This can be avoided all together if CIBCSD allowed higher flows to be diverted to the sewer system during periods of low sewer flows. A preliminary layout can be seen below in Figure 7-1.

Figure 7-1: Store and Divert to Sewer Concept Layout



LEGEND

- Proposed Improvements/ Force Main (FM)
- Proposed Gravity Main (GM)
- Approximate Boundary of San Nicholas Watershed
- Existing
- Proposed Valve Vault (Slow Diversion to CIBCSD Sewer System)
- Proposed Storage Tank and Low Flow Pump Station (40′ x 200′ x 12′)
- Existing San Nicholas Low Flow Diversion Station
- Proposed Hydrodynamic Separator
- Existing Sanitary Sewer Manhole
- Existing Storm Drain Manhole

7.3 Feasibility Analysis

7.3.1 Capital Cost

The capital cost of this option would include the storage tank within the adjacent parking lot, the submersible pump, the modified piping, the maintenance valve vault, and any necessary additional appurtenances. The capital cost associated with the installation of a new pump and the modified piping is insignificant when compared to the cost of upgrading the existing CIBCSD sewer collection system. This, however, would only be needed if additional diversion flows were sent to the CIBCSD sewer system. The projected capital cost of major improvement items is shown in Table 7-1 below. Note that the costs associated with upgrading the downgradient sewer system are not implemented within the following table, as currently, only flows of 70 GPM are proposed to be sent to the CIBCSD sewer system. For higher flow rates, an upgrade to the existing system may be needed. However, additional information from the CIBCSD sewer system is required to know the remaining flow capacity.

Table 7-1: Projected Capital Cost for Store and Divert to CIBCSD

Description	Costs
Hydrodynamic Separator (2)	$300,000
Underground Storage Tank and Low Flow Pump Station (40' x 200' x 12')	$2,000,000
Soil Stabilization, Shoring, Sheeting, Bracing, and Dewatering	$2,000,000
Valve Vault (Low Flow Diversion to CIBCSD)	$150,000
Estimated Cost (Concept Level)	**$4,450,000**
20% Concept Level Contingency	**$890,000**
Prelim. Construction Cost w/Contingency	**$5,340,000**

7.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 7-2: Projected Life Cycle Cost for Store and Divert to CIBCSD

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
Underground Storage Tank and Low Flow Pump Station (40' x 200' x 12')	80	$4,250,000
Soil Stabilization, Shoring, Sheeting, Bracing, and Dewatering	80	$4,250,000
Valve Vault (Low Flow Diversion to CIBCSD)	50	$375,000
Estimated Cost (Concept Level)		**$9,513,000**
20% Concept Level Contingency		**$1,903,000**
Prelim. Construction Cost w/Contingency		**$11,416,000**

7.3.3 Performance

This alternative ranked well in all five categories identified under Table 3-1 (Criterion Ranking Points System). It would capture the 85th percentile from entering Kiddie Beach, it would reduce pollutant load at Kiddie Beach, it would not involve discharging untreated stormwater in publicly active areas, and will not require specialized staff for operation and maintenance. Finally, it would effectively eliminate the pollutant discharge from entering the ocean, since all flows would be eventually diverted to the sewer.

7.3.4 Public Agency / Regulatory Board Support

This alternative ranked relatively well in four categories identified under Table 3-1. The water diverted to the sewer system can contribute to future water recycling efforts, the project would be expected to be permitted under the County's Local Coastal Plan, and it should qualify for various CEQA exclusions or exemptions. Grant competitiveness would not be increased, however, since this option would not involve working with a federal agency.

7.3.5 Regulatory Requirements

The construction of the storage tank and proposed pipeline would be located within the County of Ventura's Local Coastal Program Area and could require a Coastal Development Permit (CDP). Depending on the extent of the storage tank and proposed gravity main construction footprint, the project could require a Clean Water Act (CWA) Section 404 permit from the U.S. Army Corps of Engineers, a CWA 401 Water Quality Certification from the Los Angeles Regional Water Quality Control Board (LARWQCB), a California Department Fish and Wildlife Streambed Alteration Agreement 1602, and a NPDES construction permit from the LARWQCB. The pipeline would not exceed the 1-mile threshold. The project would be in compliance with Statutory Exemption 15282 (k) of the CEQA guidelines. The following regulatory requirements for this alternative are summarized as follows:

Required (Highly Probable):

- Coastal Development Permit
- Categorical Exemption Class 1
- City of Oxnard Encroachment Permit

Potentially Required (Not Probable):

- U.S. Army Corps of Engineers CWA Section 404
- LARWQCB CWA 401 Water Quality Certification and NPDES construction permit
- California Department Fish and Wildlife Streambed Alteration Agreement 1602

7.3.6 Public Perception and Impact

This alternative ranked well in all five categories identified under Table 3-1.It will not involve construction in the harbor or the beach, no long-term loss of parking, no full street closure during construction, or any perceived increased flood risk.

7.3.7 Constructability

This alternative did not rank well in the area of constructability for the three categories identified under Table 3-1. With known shallow groundwater in sandy soil, adjacent to the Kiddie Beach, soil stabilization is difficult to achieve prior to excavation and construction activities. If a contractor could dewater and stabilize the site, the next major hurdle to overcome would be to design a concrete structure that would not float when it was empty and would not allow outside groundwater to seep into the underground tank under typical dry weather conditions. If the underground tank leaked over time, the submersible pumps would run continuously, pumping seawater into the sewer collection system. If this were to occur, the City of Oxnard and CIBCSD would likely not allow the diversion system to operate. Additional CIBCSD sewer infrastructure improvements may be required if diversion flows become more constricted from the current diversion flow rate. Fortunately, however, this option does not involve any underwater construction, which would otherwise drastically complicate the process.

7.3.8 Operations & Maintenance

This alternative ranked well in the categories identified under Table 3-1. This option would require relatively infrequent maintenance visits to prevent clogging and buildup of sediment that may have surpassed the trash separating units. Standard pump maintenance would be required to ensure efficient pumping from the tank to the diversion manhole. This option would not require any specialized equipment, or safety training, and maintenance crews would have easy access to the entire system. Finally, since all 85th

percentile flows are to be diverted from Kiddie beach, there would be no additional need for TMDL compliance monitoring.

8 Pump, Store, and Percolate (US Navy ROW) (Alternative 2)

8.1 Design Approach

The pump, store, and percolate approach would allow the redirected water to bypass the existing small CIBCSD sewer pumping and conveyance system altogether. Stormwater would be diverted to and detained within basins along an open plot of land between Panama Drive and West Road, which is owned by the US Navy. All diverted flows to the basins will remain there for percolation and evaporation. An emergency overflow could also be implemented to discharge flows greater than the 85th percentile to the nearby channel along Logistics Way. A geotechnical investigation would need to confirm the feasibility of using this land for percolation basins in its current condition. However, this study assumed it was possible. If infiltration rates are limited by the high groundwater, raising the elevation in which the water would reside with permeable soils would allow for effective percolation. Given a large amount of usable land, the available soil for berm elevation adjustments would be abundant. Alternatively, flows could be discharged to the nearby channel along Logistics Way, which would eventually make it to the Port of Hueneme. As a last resort, the design could allow flows of up 70 GPM to be diverted to the CIBCSD sewer system within the nearest adjacent sewer manhole. Since this is already set as the current diversion flow rate, the additional flow will not cause any issues for the existing system. Both of these secondary options affect the feasibility of this alternative, notably categories C5, D2, and E1, from Table 3-1, depending on design pursuit. However, the feasibility of this option is relatively unaffected in the overall ranking, so for the purposes of this study, the design currently includes the usage of percolation basins. Any concerns with this approach can be addressed within the design phase.

8.2 Design Criteria

The required pump(s) would need to be capable of pumping approximately 3,200 GPM, in order to ensure 85th percentile storm flows are diverted. The force main would be required to travel approximately 1,400 feet to reach the proposed berm locations. The pump station will be a wet well with submersible pumps, thereby preserving all of the existing parking spaces. The pipe will need to be properly sized such that the head requirements from the pump can be met without unnecessarily oversizing the pump. This is because pump head requirements dictate the required motor, which is directly related to the pump cost. However, while using a larger pipeline would mitigate the required head of the pump, the cost per linear foot of pipeline would also increase proportional to size. Therefore, balancing the cost of the required pump with the size of the force main is ideal. This is a typical consideration that would need to be reviewed for all options utilizing a pump station. Utilizing the Hazen-Williams equation, the major headloss that would need to be accounted for per pipe diameter is summarized in Table 8-1 below. Note that pipelines 12" and larger mark the point in which proposed pump costs are offset by pipeline size costs. Additionally, minor losses were not accounted for within the following table, the values listed only account for frictional headloss.

Table 8-1: Headloss and Velocity per Varying Pipe Diameters

Pipe inner diameter (in)	Headloss (ft)	Velocity (ft/s)
4	5760.23	82.11
6	801.06	36.49
8	197.60	20.53
10	66.72	13.14
12	27.48	9.12
14	12.98	6.70
16	6.78	5.13
18	3.82	4.05

The approximate elevation difference between the existing SNPS finished grade and the designated location for the percolation basins is approximately 3 feet. One of the significant benefits of this alternative is that the site is not open to the public. The location of the proposed berms is located in between the fences along both Panama Drive and West Road. Currently the geometry of the basins will be 750’ in length, 80’ wide with water levels at 1.17’ high for 85th percentile volumes. The geometry of the berms is currently proposed at 14’ wide along the base, 2’ wide along the top, a 2/1 horizontal to vertical slope and 3’ in height. The elevation where the water will be kept for both berms must be consistent so that the flow is distributed between them equally. The existing grade elevations for the locations of the proposed berms ranges from 7’ to 12’ throughout. To utilize the existing grade for cut/fill requirements a berm base elevation of 10’ was determined for the design. However, as mentioned previously, the elevations can be raised further, if necessary, to account for a high ground water table. A preliminary layout can be seen below in Figure 8-1. It should be noted that some of the preliminary layouts for the alternatives mention a '3,200 GPM pump station', this value is merely an estimation based on the 85th percentile values discussed in section 5.1.1, and the actual required capacity of the pump station utilized in the design will reflect the true 85th percentile flows.

Figure 8-1: Pump, Store, and Percolate (US Navy ROW) Concept Layout

LEGEND

- Proposed 1400' Force Main (FM) *(Dia. TBD)*
- Proposed Gravity Main (GM)
- Approximate Boundary of San Nicholas Watershed
- Existing
- Proposed 3,200 GPM Pump Station and Valve Vault
- Proposed 1660' Berms for Detention Basin
- Existing San Nicholas Low Flow Diversion Station
- Proposed Hydrodynamic Separator
- Existing Sanitary Sewer Manhole
- Existing Storm Drain Manhole

8.3 Feasibility Analysis

8.3.1 Capital Cost

The capital cost of this option would include the wet well, the submersible pump, the modified piping, the maintenance valve vault, the force main, any necessary additional appurtenances, and the grading required to construct the berms. The capital cost associated with installing a new pump station and force main is significantly offset by the low cost of creating storage. It is worth noting that the US Navy's interest in collaborating for a regional improvement project opens more opportunities to obtain funding through grants. Historically, grant competitiveness increases whenever projects incorporate multiple agencies, especially federal agencies. The projected capital cost of major improvement items is shown in Table 8-2 below.

Table 8-2: Projected Capital Cost for Pump, Store and Percolate to (US Navy ROW)

Description	Costs
Hydrodynamic Separator (2)	$300,000
1,400 Feet of 12" Force Main	$600,000
Pump Station & Valve Vault (w/Dewatering)	$1,500,000
3,300 Lineal Feet of Berm (~1' Max. Storage + 2' Freeboard)	$400,000
Low-Flow Gravity Diversion to Sewer by the US Navy Site (Pending Geotechnical Investigation)	$150,000
Estimated Cost (Concept Level)	**$2,950,000**
20% Concept Level Contingency	**$590,000**
Prelim. Construction Cost w/Contingency	**$3,540,000**

8.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation. Note that sewer diversion fees were not considered in the life cycle cost since the intended goal is percolation. If percolation is not possible with the current grade and the groundwater elevation, designs could be adjusted as discussed in 8.1. Sewer diversion is currently seen as a last resort and therefore should not be incorporated in life cycle costs.

Table 8-3: Projected Life Cycle Cost for Pump, Store and Percolate to (US Navy ROW)

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
1,400 Feet of 12" Force Main	80	$1,275,000
Pump Station & Valve Vault (w/Dewatering)	50	$3,750,000
3,300 Lineal Feet of Berm (~1' Max. Storage + 2' Freeboard)	100	$800,000
Low-Flow Gravity Diversion to Sewer by the US Navy Site (Pending Geotechnical Investigation)	80	$319,000
Estimated Cost (Concept Level)		**$6,782,000**
20% Concept Level Contingency		**$1,356,000**
Prelim. Construction Cost w/Contingency		**$8,138,000**

8.3.3 Performance

This alternative ranked well in all five categories identified under Table 3-1 (Criterion Ranking Points System). It would capture the 85th percentile flows and reduce pollutant load from entering Kiddie Beach. Additionally it would not involve discharging untreated stormwater to active public areas, nor would it require specialized staff for operation and maintenance. Finally, it would effectively eliminate the pollutant discharge from entering the ocean, since all flows would be diverted to the basins. This however, would not be the case if a design that discharges to the Port of Hueneme was pursued. However, since this is not the current design intent, it was ranked with the assumption that discharging to the port would not be pursued.

The system would divert to the CIBCSD only when necessary for interim maintenance. This would be done through the existing diversion within the SNPS, which would send 70 GPM flows to the CIBCSD sewer system. This alternative, aside from maintenance intervals, could ultimately avoid the monthly fees that could be charged by CIBCSD, with the current diversion system. This is because the proposed design sends all flows to the percolation basins, leaving the existing SNPS diversion system relatively unused, below the 85th percentile flows. The fees that could be imposed by CIBCSD would be for the usage of their sewer system in diverting untreated stormwater, which is then treated at the Oxnard WWTP.

8.3.4 Public Agency / Regulatory Board Support

This alternative ranked relatively well in three categories identified in Table 3-1. The project would be expected to be permitted under the County's Local Coastal Plan, and it would have increased grant competitiveness due to collaboration with a federal agency. Additionally, since the force main is projected to be less than a mile long, it could also qualify for CEQA statutory exemption. However, since the water is not being reused or recycled, there would not be any potential benefits aside from the bacteria TMDL reduction.

8.3.5 Regulatory Requirements

Construction of the proposed pipeline would be located within the County of Ventura's Local Coastal Program Area and would require a CDP. Depending on the extent of the construction footprint, the project could require a CWA Section 404 permit from the U.S. Army Corps of Engineers, a CWA 401 Water Quality Certification from LARWQCB, a California Department Fish and Wildlife Streambed Alteration Agreement 1602, and a NPDES construction permit from the LARWQCB. The pipeline would not exceed the 1-mile threshold for a Statutory Exemption 15282 (k) of the CEQA guidelines. The project would still require Initial Study/ Mitigated Negative Declaration (IS/MND) for the construction of the pipeline and detention basin and an Environmental Assessment (EA), since the detention basin is located on US Navy property. It can be assumed that the necessary permits for the work carried out on the base will be prepared by the US Navy based on their previous collaborative efforts to do so. The following regulatory requirements for this alternative are summarized as follows:

Required (Highly Probable):

- Coastal Development Permit
- Initial Study / Mitigated Negative Declaration / Environmental Assessment
- LARWQCB Discharge Permit
- City of Oxnard (With applicable associated permits)
- County of Ventura Encroachment permit

Potentially Required (Not Probable):

- U.S. Army Corps of Engineers CWA Section 404
- LARWQCB CWA 401 Water Quality Certification and NPDES construction permit
- California Department Fish and Wildlife Streambed Alteration Agreement 1602

8.3.6 Public Perception and Impact

This alternative ranked well in all five categories identified under Table 3-1. It will not involve construction in the harbor or the beach, no long-term loss of parking and no full street closure during construction. There are several streets that would endure partial street closures for the length of the force main, however the berm construction would be out of the way from public streets and walkways. There is a slight increase to flood risk in the event that the diversion pumps send too much water, or the berms leak. This can be accounted for however, by allowing the berms to have an overflow into the nearby channel along logistics way, that would discharge into the Port of Hueneme.

8.3.7 Constructability

This alternative ranked well in two categories identified under Table 3-1. It will not require any additional third-party infrastructure improvement (such as upsizing the CIBCSD sewer system), no large underground storage tank in a high groundwater environment, and no construction along the beach or in the harbor. However, this option would require pipeline trenching for the proposed force main.

8.3.8 Operations & Maintenance

This alternative ranked well in the categories identified in Table 3-1. Maintenance would be relatively infrequent, as compared to the other options, there would be no specialized equipment or safety training requirements. Additionally, since the storage system is above ground, maintenance would be easier than the other alternatives. Finally, since all 85^{th} percentile flows are being diverted from Kiddie Beach, there would no requirement for annual regulatory compliance monitoring. Regular inspection and cleaning of the above ground detention basin to ensure efficient percolation, as well as maintenance of the discharge pumps to the basin, are necessary considerations for this alternative. This option would also require the regular maintenance for the trash separators per manufacturer's recommendations.

9 Divert to Sewer (Replace CIBCSD Pump Station & Force Main) (Alternative 3)

9.1 Design Approach

The divert to sewer approach would redirect incoming water directly into the CIBCSD sewer collection and pumping system. However, per the sewer study conducted on the CIBCSD sewer system (see Appendix B), the existing sewer pipes do not have any surplus capacity for increased conveyance. In addition, CIBCSD recently required the dry weather diversion pump to operate at a reduced flow rate (i.e., from 120 GPM to 70 GPM). Therefore, sending the 85th percentile flow, at an estimated rate of 3,200 GPM, would require replacement of CIBCSD's sewer pump station and over 2 miles of force main. This could also require infrastructure improvement further downstream, including at the City of Oxnard's wastewater treatment plant.

9.2 Design Criteria

To convey wet weather flows for the 85th percentile of storms, a new 3,200 GPM below-ground pump station and force main would be needed to convey storm flows to the CIBCSD lift station. As mentioned above, the existing CIBCSD lift station and force main would need to be upsized and replaced as well. This would have significant added operational expense for CIBCSD and would put greater responsibility onto CIBCSD staff, presenting a major obstacle for this alternative. Furthermore, the City of Oxnard's wastewater treatment plant would also potentially have negative impacts from such a large design flow rate. For these reasons, this alternative was deemed infeasible and was not ranked against other potential projects. A preliminary layout is shown below in Figure 9-1.

Figure 9-1: Divert to Sewer (Replace CIBCSD Pump Station & Force Main) Concept Layout

9.3 Feasibility Analysis

As mentioned above, this alternative was deemed infeasible, as it would require costly third-party infrastructure improvements. Therefore, it was not ranked using the criterion and ranking matrix established in Section 3. Included below are the Capital Cost (Table 9-1) and the 50-Year Life Cycle Cost (Table 9-2) estimates for the alternative.

9.3.1 Capital Cost

Table 9-1: Projected Capital Cost for Divert to Sewer (Replace CIBCSD Pump Station & FM)

Description	Costs W/ Separators	Costs W/O Separators
Hydrodynamic Separator (2) (Optional)	$300,000	$0
750 Feet of 12" Force Main	$320,000	$320,000
3,200 GPM Pump Station & Valve Vault	$1,500,000	$1,500,000
Estimated Cost (Concept Level)	**$2,120,000**	**$1,820,000**
20% Concept Level Contingency	**$424,000**	**$364,000**
Prelim. Construction Cost w/Contingency *	**$2,544,000**	**$2,184,000**

**** Not Including Cost to Replace CIBCSD Pump Station & Force Main***

9.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 9-2: Projected Life Cycle Cost for Divert to Sewer (Replace CIBCSD Pump Station & FM)

Description	Est. Life (Yrs)	Costs W/ Separators	Costs W/O Separators
Hydrodynamic Separator (2) (Optional)	80	$638,000	$0
750 Feet of 12" Force Main	80	$680,000	$680,000
3,200 GPM Pump Station & Valve Vault	50	$3,750,000	$3,750,000
Estimated Cost (Concept Level)		**$5,068,000**	**$4,430,000**
20% Concept Level Contingency		**$1,014,000**	**$886,000**
Prelim. Construction Cost w/Contingency *		**$6,082,000**	**$5,316,000**

**** Not Including Cost to Replace CIBCSD Pump Station & Force Main***

10 Divert to Sewer (Lift Station 29 on Patterson Road) (Alternative 4)

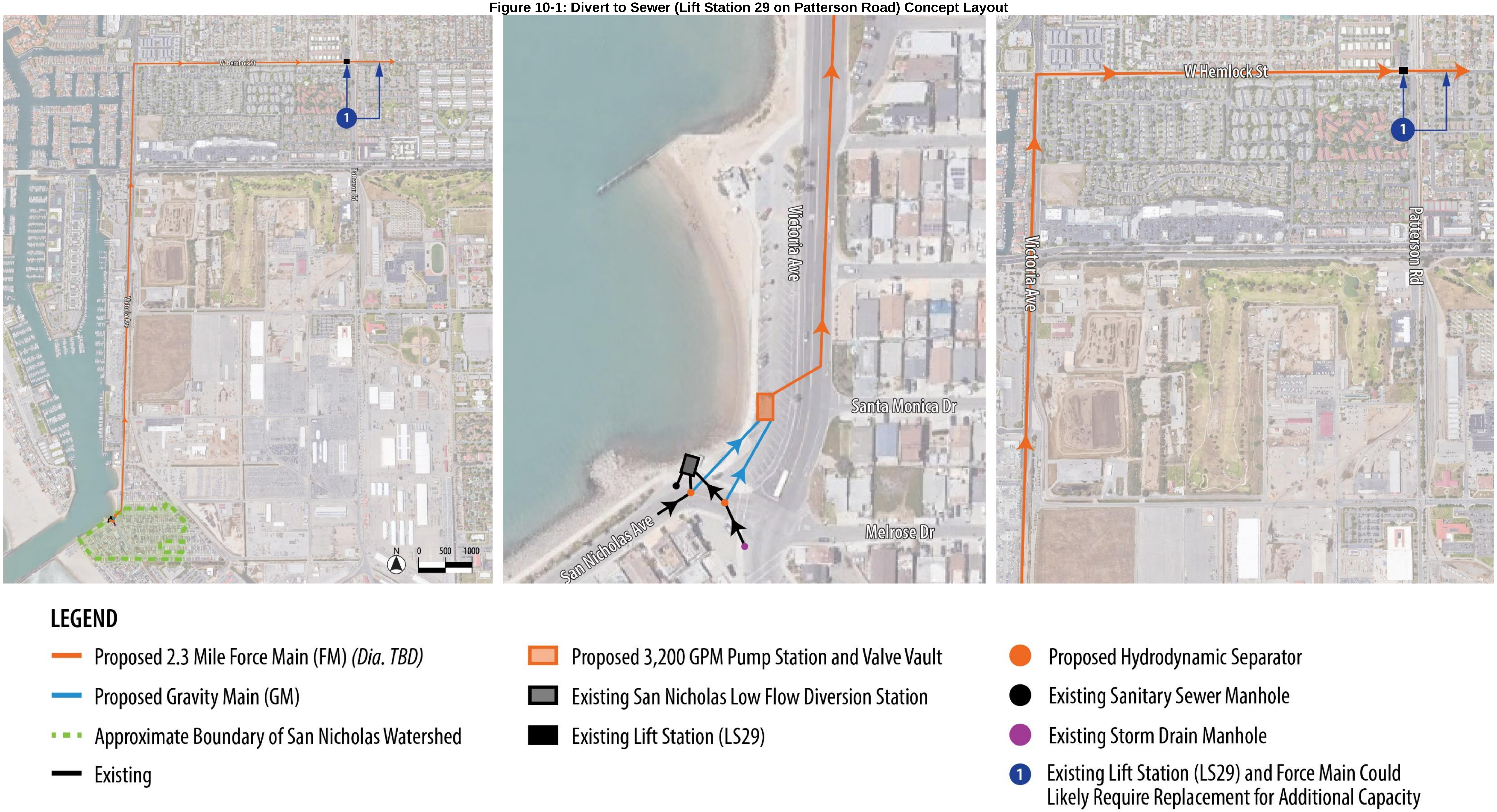
10.1 Design Approach

The divert to sewer approach would redirect incoming water directly into the City of Oxnard's sewer Lift Station 29, on Patterson Road. Lift Station 29 is located over 2 miles from the SNPS. This approach would eliminate the connection with the CIBCSD system, but instead, connect to the City's system further downstream. Similar to the previous diversion alternative in Section 9, capacity to accommodate the 85th percentile flow, at an estimated rate of 3,200 GPM, may require infrastructure improvement further downstream, including at the City of Oxnard's wastewater treatment plant.

10.2 Design Criteria

To convey wet weather flows within the 85th percentile of storms, a new 3,200 GPM below-ground pump station and force main would be needed to convey storm flows to the City of Oxnard's existing sewer Lift Station 29, located on Patterson Road. This lift station is over 2 miles northeast of the SNPS. The City of Oxnard's existing system would be negatively impacted by the large design flow rate associated with the 85th percentile of storms. For this reason, this alternative was deemed infeasible and was not ranked against other potential projects. A preliminary layout can be seen below in Figure 10-1.

Figure 10-1: Divert to Sewer (Lift Station 29 on Patterson Road) Concept Layout



10.3 Feasibility Analysis

As mentioned above, this alternative was deemed infeasible as it could require costly third-party infrastructure improvements. The capital cost for the diversion to Lift Station 29 is high even without the potential added financial impact of upgrading downgradient infrastructure. Therefore, this alternative was not ranked using the criterion and ranking matrix established in Section 3. Included below are the Capital Cost (Table 10-1) and the 50-Year Life Cycle Cost (Table 10-2) estimates.

10.3.1 Capital Cost

Table 10-1: Projected Capital Cost for Divert to Sewer (Lift Station 29 on Patterson Road)

Description	Costs W/ Separators	Costs W/O Separators
Hydrodynamic Separator (2) (Optional)	$300,000	$0
2.3 Miles of 12" Force Main	$5,200,000	$5,200,000
3,200 GPM Pump Station & Valve Vault	$1,500,000	$1,500,000
Estimated Cost (Concept Level)	**$7,000,000**	**$6,700,000**
20% Concept Level Contingency	**$1,400,000**	**$1,340,000**
Prelim. Construction Cost w/Contingency	**$8,400,000**	**$8,040,000**

**** Not Including Potential Cost to Upsize Oxnard's Pump Station / FM / WWTP***

10.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 10-2: Projected Life Cycle Cost for Divert to Sewer (Lift Station 29 on Patterson Road)

Description	Est. Life (Yrs)	Costs W/ Separators	Costs W/O Separators
Hydrodynamic Separator (2) (Optional)	80	$638,000	$0
2.3 Miles of 12" Force Main	80	$11,050,000	$11,050,000
3,200 GPM Pump Station & Valve Vault	50	$3,750,000	$3,750,000
Estimated Cost (Concept Level)		**$15,438,000**	**$14,800,000**
20% Concept Level Contingency		**$3,088,000**	**$2,960,000**
Prelim. Construction Cost w/Contingency		**$18,526,000**	**$17,760,000**

**** Not Including Potential Cost to Upsize Oxnard's Pump Station / FM / WWTP***

11 Store and Treat for Off-Site Reuse (Alternative 5)

11.1 Design Approach

The store and treat for off-site reuse approach would redirect incoming water to be stored in an equalization storage tank that would accumulate incoming dry and wet weather flows before being pumped by a low flow rate pump station. When there is a water demand, the water would be treated and distributed to a user(s) through a distribution force main. The treatment system may be required to meet Title 22 standards for unrestricted reuse. This approach assumes the user(s) would have water storage to meet their unique demands. Since dry and wet weather flow will be reused, the existing diversion system that discharges into the CIBCSD sewer system will become a system backup.

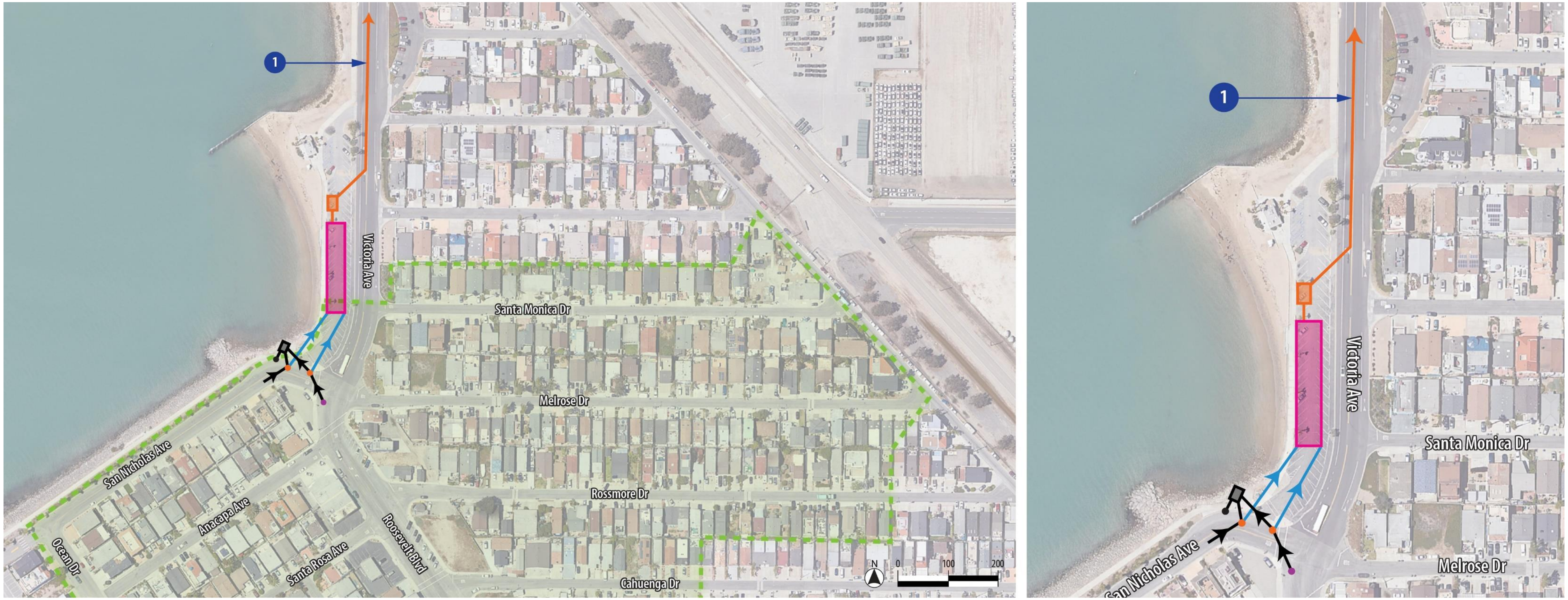
It is important to note that the Title 22 regulatory requirement for unrestricted reuse is for water with a wastewater source origin, not stormwater. Since PACE is unaware of formal documented requirements for water with a stormwater source, treatment projects are commonly designed conservatively to meet Title 22 standards. Title 22 of the California Code of Regulations lays out the standards and requirements for the treatment of recycled water for various uses, including irrigation, industrial processes, and toilet and urinal flushing. The State Water Resources Control Board oversees the implementation and enforcement of Title 22 regulations, which specify treatment processes and disinfection requirements to ensure that the recycled water is safe and meets the necessary quality standards. The regulations also address monitoring, reporting, and record-keeping requirements to ensure that the recycled water continues to meet the required standards over time. The specific treatment requirements depend on the intended usage of the recycled water.

11.2 Design Criteria

To provide storage for wet weather flows within the 85th percentile of storms, a large storage tank would need to be incorporated within the design (refer to Section 7 for the placement, sizing, and challenges with the tank concept). A low-flow below-ground pump station, above-ground treatment system, and new force main would convey treated water to its user(s). The most feasible treatment option could potentially be an ozone system, as it is relatively reliable, performs well, and requires significantly less land when compared to a conventional treatment system that would require chlorine contact chambers. Ozone or chlorine contact chambers are less expensive to install and operate compared to UV treatment systems. UV treatment requires a significant amount of energy to operate, and the lamps and sleeves used in the UV systems require regular replacement. In contrast, ozone or chlorine contact chambers can be relatively low-maintenance and cost-effective over the long term. It was for these reasons that UV treatment was disregarded as a feasible solution for the alternatives proposing stormwater treatment.

The potential viability of this alternative depends heavily on whether there are nearby users for the treated water. Assessor maps and other parcel data were gathered to identify open spaces that could be future users of treated water (See Section 11 of the technical memorandum in Appendix C). The only potential user identified was the golf course, owned by the US Navy. Subsequently, VCPWA-WP and PACE met with representatives from the US Navy and discovered that they already have a water source with a treatment system and would not be interested in additional water from this project. Since there are no other potential nearby user(s) for treated water, this alternative was deemed infeasible and was not ranked against other potential projects. A preliminary layout can be seen below in Figure 11-1.

Figure 11-1: Store and Treat for Off-Site Reuse Concept Layout



LEGEND

- Proposed Improvements/ Force Main (FM)
- Proposed Gravity Main (GM)
- Approximate Boundary of San Nicholas Watershed
- Existing
- Proposed Above-Ground Treatment System and Valve Vault
- Proposed Storage Tank and Low Flow Pump Station (40′ x 200′ x 12′)
- Existing San Nicholas Low Flow Diversion Station
- Proposed Hydrodynamic Separator
- Existing Sanitary Sewer Manhole
- Existing Storm Drain Manhole
- 1 Recycled Water to Potential User (No Designated Customer)

11.3 Feasibility Analysis

As mentioned above, this alternative was deemed infeasible because potential user(s) of the treated water were not identified. Even if a user(s) were identified, the significant additional cost of a new distribution system would make this alternative uncompetitive when compared to other lower-cost options. Therefore, this alternative was not ranked using the criterion and ranking matrix established in Section 3. Included below are the Capital Cost (Table 11-1) and the 50-Year Life Cycle Cost (Table 11-2) estimates.

11.3.1 Capital Cost

Table 11-1: Projected Capital Cost for Store, Treat and Reuse

Description	Costs
Hydrodynamic Separator (2)	$300,000
Underground Storage Tank and Low Flow Pump Station (40' x 200' x 12')	$2,000,000
Soil Stabilization, Shoring, Sheeting, Bracing, and Dewatering	$2,000,000
Above-Ground Treatment System with Valve Vault	$2,000,000
Estimated Cost (Concept Level)	**$6,300,000**
20% Concept Level Contingency	**$1,260,000**
Prelim. Construction Cost w/Contingency	**$7,560,000**

**** Not including distribution system cost***

11.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 11-2: Projected Life Cycle Cost for Store, Treat and Reuse

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
Underground Storage Tank and Low Flow Pump Station (40' x 200' x 12')	80	$4,250,000
Soil Stabilization, Shoring, Sheeting, Bracing, and Dewatering	80	$4,250,000
Above-Ground Treatment System with Valve Vault	50	$5,000,000
Estimated Cost (Concept Level)		**$14,138,000**
20% Concept Level Contingency		**$2,828,000**
Prelim. Construction Cost w/Contingency		**$16,966,000**

**** Not Including Force Main Cost***

12 Treat for Off-Site Reuse (Alternative 6)

12.1 Design Approach

The approach to the treat for off-site reuse alternative is very similar to the store and treat for off-site reuse alternative described in Section 11. The primary difference is that storage at the parking lot would be removed from the process flow and the below-ground pump station would be larger, designed for 3,200 GPM.

12.2 Design Criteria

To account for wet weather flows within the 85th percentile of storms, a below-ground pump station, above-ground treatment system, and new force main would convey treated water to the user(s). The system would be designed to satisfy the 3,200 GPM flow parameter. The most feasible potential treatment option would be an ozone system, as it is relatively reliable, performs well, and requires significantly less space compared to a conventional treatment system with chlorine contact chambers. UV treatment was not considered a viable alternative for the same reasons discussed in 11.2.

Similar to the store and treat for off-site reuse alternative described in Section 11, the potential viability of this alternative depends heavily on whether there are nearby users for the treated water. Assessor maps and other parcel data were gathered to identify open spaces that could be future users of treated water (See Section 11 of the technical memorandum in Appendix C). The only potential user identified was the golf course owned by the US Navy. Subsequently, VCPWA-WP and PACE met with representatives from the US Navy and discovered that they already have a water source with a treatment system and would not be interested in additional water from this project. Since there are not any other potential nearby user(s) for treated water, this alternative was deemed infeasible and was not ranked against other potential projects. A preliminary layout can be seen below in Figure 12-1.

Figure 12-1: Treat for Off-Site Reuse Concept Layout

12.3 Feasibility Analysis

As mentioned above, this alternative was deemed infeasible because the potential user(s) of the treated water was not identified. Even if a user(s) were identified, the significant additional cost of a new distribution system would make this alternative uncompetitive when compared to other lower-cost options. Therefore, this alternative was not ranked using the criterion and ranking matrix established in Section 3. Included below are the Capital Cost (Table 12-1) and the 50-Year Life Cycle Cost (Table 12-2) estimates.

12.3.1 Capital Cost

Table 12-1: Projected Capital Cost for Treat for Off-Site Reuse

Description	Costs
Hydrodynamic Separator (2)	$300,000
3,200 GPM Pump Station & Valve Vault	$1,500,000
Above-Ground Treatment System with Valve Vault	$2,500,000
Estimated Cost (Concept Level)	**$4,300,000**
20% Concept Level Contingency	**$860,000**
Prelim. Construction Cost w/Contingency	**$5,160,000**

**** Not including distribution system cost***

12.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 12-2: Projected Life Cycle Cost for Treat for Off-Site Reuse

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
3,200 GPM Pump Station & Valve Vault	50	$3,750,000
Above-Ground Treatment System with Valve Vault	50	$6,250,000
Estimated Cost (Concept Level)		**$10,638,000**
20% Concept Level Contingency		**$2,128,000**
Prelim. Construction Cost w/Contingency		**$12,766,000**

**** Not Including Force Main Cost***

13 Treat and Release (Alternative 7)

13.1 Design Approach

The treat and release alternative would be very similar to the treat for off-site reuse alternative described in Section 12. The primary difference would be that the wet well for the 3,200 GPM below-ground pump station would increase in size to have two separate hydraulic chambers, separated by an over-flow weir, and a short force main to send treated water back to the existing SNPS wet well. This alternative would take advantage of the existing SNPS outfall and would not have a separate discharge point to Kiddie Beach.

It is important to note this alternative would be the most complicated to both design and operate, as the treatment system and two pump stations would need to operate in unison. These concerns are addressed within section 13.3.8.

13.2 Design Criteria

To account for wet weather flows within the 85th percentile of storms, the system would require a below-ground pump station with a two-chamber wet well, an above-ground treatment system, and a new force main to convey treated water into the existing SNPS wet well. The system would be designed to satisfy the 3,200 GPM flow parameter. The most feasible potential treatment option would be an ozone system, as it is relatively reliable, performs well, and requires significantly less space than a conventional treatment system with chlorine contact chambers. UV treatment was not considered a viable alternative for the same reasons discussed in 11.2. A preliminary site layout can be seen below in Figure 13-1.

Figure 13-1: Treat and Release Concept Layout

13.3 Feasibility Analysis

13.3.1 Capital Cost

The capital cost of this option would include the two-chamber wet well with an over-flow weir, the submersible pump, the above-ground treatment system, the modified piping, the maintenance valve vault, the force main, control modification at the existing SNPS, and any necessary additional appurtenances (Each of these components are grouped within the "pump station and weir diversion" listing within Table 13-1, aside from the above-ground treatment system listed separately). From the capital cost perspective, this alternative ranked relatively well. The projected capital cost of major improvement items is shown in Table 13-1 below.

Table 13-1: Projected Capital Cost for Treat and Release

Description	Costs
Hydrodynamic Separator (2)	$300,000
Above-Ground Treatment System	$2,500,000
Pump Station & Weir Diversion	$2,000,000
Estimated Cost (Concept Level)	**$4,800,000**
20% Concept Level Contingency	**$960,000**
Prelim. Construction Cost w/Contingency	**$5,760,000**

13.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 13-2: Projected Life Cycle Cost for Treat and Release

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
Above-Ground Treatment System	50	$6,250,000
Pump Station & Weir Diversion	50	$5,000,000
Estimated Cost (Concept Level)		**$11,888,000**
20% Concept Level Contingency		**$2,378,000**
Prelim. Construction Cost w/Contingency		**$14,266,000**

13.3.3 Performance

This alternative ranked poorly in the categories identified under Table 3-1 (Criterion Ranking Points System). It would reduce pollutant load at Kiddie Beach and does not involve discharging untreated stormwater in active public areas. However, this alternative would be the most complicated to design and maintain and would likely require specialized staff for the operation and maintenance of the designated treatment system. Additionally, this option would not eliminate total pollutant load from discharge and there's a possibility that 85th percentile storm discharges would not receive the required treatment per Title 22 compliance requirements at the higher flow rates.

13.3.4 Public Agency / Regulatory Board Support

This alternative ranked below average in the four categories identified under Table 3-1. The only points scored for this project would be the ability to permit under the County's Local Coastal Plan.

13.3.5 Regulatory Requirements

The construction of the proposed pump station and weir diversion would be located within the County of Ventura's Local Coastal Program Area and would require a CDP. The changes on the existing discharge into the marina would require a Section 10 and CWA 404 from the U.S. Army Corps of Engineers, a CWA 401 Water Quality Certification, and California Department Fish and Wildlife Streambed Alteration Agreement 1602. Depending on the project footprint, an NPDES construction permit from the LARWQCB may be required. CEQA compliance may require an IS / MND for the treatment system. The following regulatory requirements for this alternative are summarized as follows:

Required (Highly Probable):

- Coastal Development Permit
- IS / MND
- U.S. Army Corps of Engineers Rivers and Harbors Act Section 10 and CWA Section 404
- LARWQCB CWA 401 Water Quality Certification
- California Department Fish and Wildlife Streambed Alteration Agreement 1602
- City of Oxnard
- County of Ventura Encroachment Permit

Potentially Required (Not Probable):

- LARWQCB NPDES construction permit

13.3.6 Public Perception and Impact

This alternative ranked average in the five categories identified under Table 3-1. It will not involve construction in the harbor or the beach, will not need full street closure during construction, and will not create any perceived increased flood risk. However, the above-ground treatment system will create a long-term loss of parking.

13.3.7 Constructability

This alternative ranked well in two of the three categories identified under Table 3-1. It will not require Third-party infrastructure improvement, construction of a large underground storage tank in a high groundwater environment, or underwater construction. However, it will require pipeline trenching and installation of a wet well within a high groundwater environment.

13.3.8 Operations & Maintenance

This alternative ranked poorly in four of the five categories identified under Table 3-1. Treating stormwater that will be released to the ocean requires regular operations and maintenance to ensure compliance with regulatory requirements and prevent equipment failures. Concerns include the frequency of maintenance needed to keep the treatment systems running efficiently, the requirement of replacing specialized equipment, such as filters, pumps, and valves, as they wear out or become clogged, and the need for specialized training for operators to understand the treatment systems, perform maintenance tasks and operate the system effectively. The only notable benefit for this approach is the greater accessibility associated with having all the components of the treatment localized within the extents of the parking lot.

14 Divert to Santa Paula Pump Station (Alternative 8)

14.1 Design Approach

The divert to the Santa Paula Pump Station alternative would redirect water from the SNPS to the Santa Paula Pump Station to discharge up to the 85th percentile storm flows through the existing ocean outfall. Anecdotal wisdom suggests that the water in the harbor at Kiddie Beach, even with daily tidal influence, is far more stagnant than at the beach outfall for the Santa Paula Pump Station. It is possible that the turbulent water at the outfall for the Santa Paula Pump Station may dissipate pollutants much more effectively. The Santa Paula Pump Station has extra capacity at the 85th percentile storm events that can be utilized to discharge the stormwater collected in the San Nicholas sub-watershed. The Santa Paula outfall is currently being retrofitted to discharge further up along the beach, to alleviate sand impaction at the outlet. This would allow some flows discharged from Santa Paula to percolate into the sand before being discharged into the ocean.

14.2 Hydraulic Analysis

In order to determine the 85th percentile storm flows within the Santa Paula Watershed, the same hydraulic process discussed in section 4.1.3 was adopted. Given that the total Santa Paula Watershed spans approximately 17% more area than the San Nicholas Watershed, the anticipated flow rate was calculated at approximately 3,800 GPM. A 100-year storm analysis was conducted in the 2014 Silver Strand Pump Station Deficiency Study (see Appendix E for excerpts of this study) to identify the outflow capacity of the Santa Paula Pump Station, which was 58 cfs, or approximately 26,000 GPM. Given that the Santa Paula Watershed flow rates for 85th percentile storms are approximately 15% of this maximum design flow, all flows measured within this model were scaled by that factor. This allowed an analysis of the existing storm drains that can withstand the additional diverted flows during an 85th percentile storm. With this approach, the closest available storm drain with the additional capacity for the diverted flows is along the intersection of Bardsdale Avenue and Ocean Drive. The depth of flow within the 30” storm drain before and after the diversion during an 85th percentile storm event can be seen in Table 14-1 below.

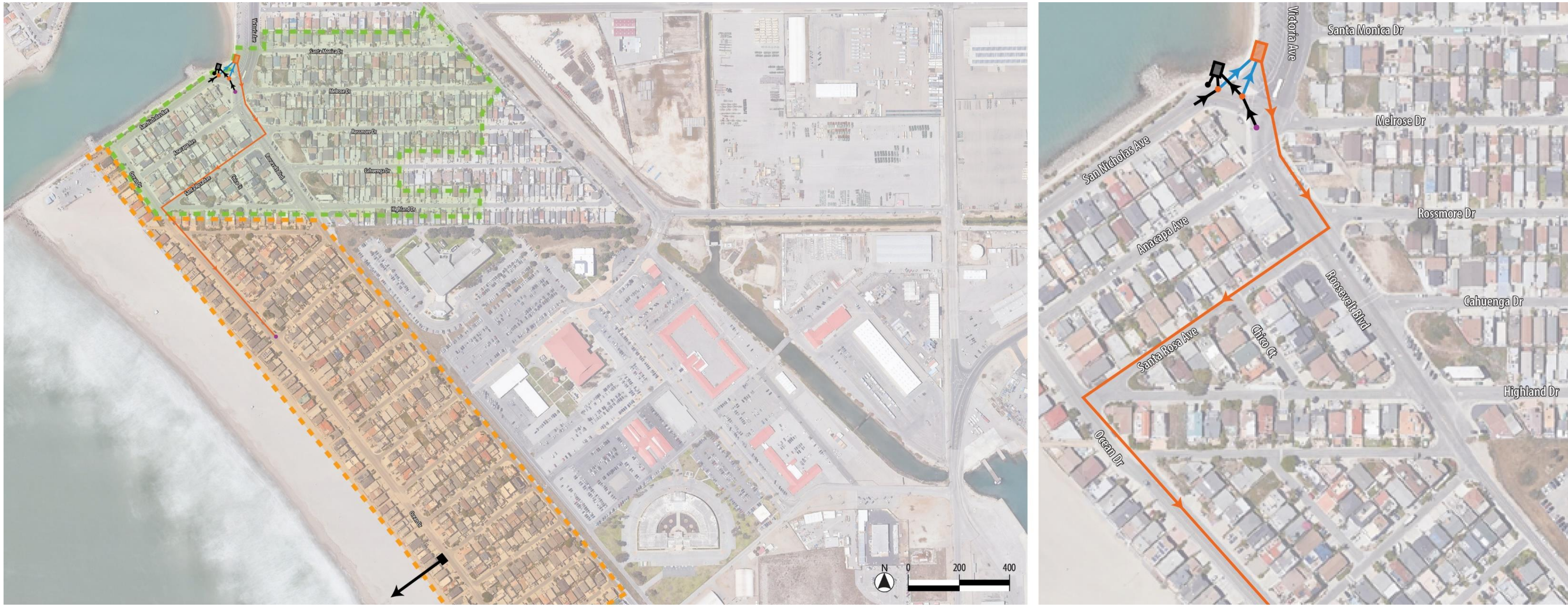
Table 14-1: Bardsdale Avenue Storm Drain Capacity Analysis

Size of Storm Drain (ft)	Before Diversion Depth of Flow (ft)	After Diversion Depth of Flow (ft)
2.50	1.09	2.20

14.3 Design Criteria

The required pump(s) would need to be capable of pumping the 85th percentile storm flows to the nearest storm drain in the Santa Paula watershed, which can withstand the additional flows during a rain event. An approximate 1,500-foot force main would be required to connect the proposed diversion pump station to this storm drain. The pump station would be a wet well with submersible pumps, thereby preserving the existing parking spaces. The force main pipe would need to be properly sized to balance pump costs versus the capital cost to install the new force main. When the storm flow intensity exceeds the 85th percentile rate, the diversion pumps will stop to allow both existing stormwater pump stations to perform per their current design. A preliminary site layout can be seen below in Figure 14-1. For locations of the existing storm drains within the Santa Paula watershed, refer to 2014 Silver Strand Pump Station Deficiency Study 100-Year storm excerpts in Appendix E.

Figure 14-1: Divert to Santa Paula Pump Station Concept Layout



LEGEND

- Proposed 1,300' Force Main (FM) *(Dia. TBD)*
- Proposed Gravity Main (GM)
- Approximate Boundary of San Nicholas Watershed
- Approximate Boundary of Santa Paula Watershed
- Existing
- Proposed 3,200 GPM Pump Station and Valve Vault
- Existing San Nicholas Low Flow Diversion Station
- Existing Santa Paula Pump Station
- Proposed Hydrodynamic Separator
- Existing Sanitary Sewer Manhole
- Existing Storm Drain Manhole

14.4 Feasibility Analysis

14.4.1 Capital Cost

The capital cost of this alternative would include the wet well, the submersible pumps, the modified piping, the maintenance valve vault, the force main, and any necessary additional appurtenances. From the capital cost perspective, this alternative ranked the highest of the alternatives. The projected capital cost of major improvement items is shown in Table 14-2 below.

Table 14-2: Projected Capital Cost for Santa Paula Diversion

Description	Costs
Hydrodynamic Separator (2)	$300,000
1,500 Feet of 12" Force Main	$650,000
Pump Station & Valve Vault	$1,500,000
Estimated Cost (Concept Level)	**$2,450,000**
20% Concept Level Contingency	**$490,000**
Prelim. Construction Cost w/Contingency	**$2,940,000**

14.4.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 14-3: Projected Life Cycle Cost for Santa Paula Diversion

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
1,500 Feet of 12" Force Main	80	$1,381,000
Pump Station & Valve Vault	50	$3,750,000
Estimated Cost (Concept Level)		**$5,769,000**
20% Concept Level Contingency		**$1,154,000**
Prelim. Construction Cost w/Contingency		**$6,923,000**

14.4.3 Performance

This alternative ranked average in the five categories identified under Table 3-1 (Criterion Ranking Points System). It would divert the 85th percentile of storm flows and effectively reduce the pollutant load from entering Kiddie Beach. Additionally, this option would require only standard operational and maintenance procedures with a typical pump station. However, this alternative would discharge to an active public area along the Silver Strand State Beach. Additionally, since this option only displaces the bacterial load, it would not effectively eliminate the pollutant load from discharge.

14.4.4 Public Agency / Regulatory Board Support

This alternative ranked relatively well in three categories identified under Table 3-1. The project can be permitted under the County's Local Coastal Plan, it should qualify for various CEQA exclusions or exemptions. However, this option would not increase grant competitiveness, since there would be no collaboration with a federal agency, such as the US Navy.

14.4.5 Regulatory Requirements

As mentioned previously, there is a possibility that a TMDL could be adopted at the discharge location of the Santa Paula Pump Station sub-watershed (i.e., Silver Strand Beach) in the future, if pollutant dissipation at the Santa Paula Pump Station outfall was ineffective, thereby risking this proposed alternative from becoming obsolete.

The construction of the proposed pump station and pipeline would be located within the County of Ventura's Local Coastal Program Area and would require a CDP. Depending on the extent of the proposed gravity main and pump station construction footprint, the project could require a CWA 404 from the U.S. Army Corps of Engineers, a CWA 401 Water Quality Certification, and California Department Fish and Wildlife Streambed Alteration Agreement 1602. In addition, an NPDES construction permit from the LARWQCB could be required. CEQA compliance would require a Class 1 Categorical Exemption 15301 Existing Facilities for the pump station and Statutory Exemption 15282 (k) for the pipeline. The following regulatory requirements for this alternative are summarized as follows:

Required (Highly Probable):

- Coastal Development Permit
- California Department Fish and Wildlife Streambed Alteration Agreement 1602
- Categorical Exemption Class 1
- Statutory Exemption 15282 (k)
- County of Ventura Encroachment Permit
- City of Oxnard

Potentially Required (Not Probable):

- U.S. Army Corps of Engineers CWA Section 404
- LARWQCB CWA 401 Water Quality Certification and NPDES construction permit

14.4.6 Public Perception and Impact

This alternative ranked relatively well in four categories identified under Table 3-1. It would not involve construction in the harbor or the beach, long-term loss of parking, or full street closure during construction; however, the perception of an increased flood risk from adding flows to the Santa Paula subwatershed could be a hurdle to overcome.

14.4.7 Constructability

This alternative ranked well in the categories identified under Table 3-1. It would not require Third-party infrastructure improvement, construction of a large underground storage tank in a high groundwater environment, or construction along the beach or in the harbor. However, it would require pipeline trenching and installation of a wet well within a high groundwater environment.

14.4.8 Operations & Maintenance

This alternative ranked well in all five categories identified under Table 3-1. The frequency of required maintenance is minimal, with regular inspections and occasional cleaning of the separator and valve vault. Replacing specialized equipment is not a concern with this system, as the equipment required is standard and readily available. Additionally, no specialized safety training procedures would be required, as the system is very typical for pump station crews. Maintenance crews will have ease of access within the pump station and valve vault whenever necessary. Compliance monitoring is not a concern since the discharge would not be within the extents of Kiddie Beach.

15 San Nicholas Pump Station Ocean Outfall (Alternative 9)

15.1 Design Approach

The new ocean outfall alternative would direct all the flows from the SNPS beyond the riprap breakers located west of the mouth of the harbor and would allow dry and wet weather flows to dilute with open ocean water. This alternative ranked the lowest among the alternatives considered. This alternative was kept in contention because it would satisfy regulatory compliance and would be the easiest option to manage by staff. The outfall pipe would be connected to the existing discharge pumps so that no additional wet well would be necessary.

15.2 Design Criteria

The construction of an ocean outfall would be exceptionally challenging, requiring dredging to lay the pipe beneath the surface of the ocean floor, and it will impact the day-to-day use of the harbor and beach. A preliminary site layout for this option is shown in Figure 15-1 below.

Figure 15-1: San Nicholas Pump Station Ocean Outfall Concept Layout

15.3 Feasibility Analysis

15.3.1 Capital Cost

The capital cost of a new outfall pipeline would include a dredging operation to create a trench to lay the pipe beneath the harbor floor. Outfall projects typically trigger environmentally sensitive topics, requiring both short-term and long-term mitigation and monitoring. The projected capital cost of major improvement items is shown in Table 15-1 below.

Table 15-1: Projected Capital Cost for Ocean Outfall

Description	Costs
Hydrodynamic Separator (2)	$300,000
3,000 Feet of 20" Outfall	$6,000,000
Permit Compliance & Annual Monitoring	$500,000
Estimated Cost (Concept Level)	**$6,800,000**
20% Concept Level Contingency	**$1,360,000**
Prelim. Construction Cost w/Contingency	**$8,160,000**

15.3.2 50-Year Life Cycle Cost

Refer to Section 3.2.1 for the method and breakdown of the life cycle cost calculation.

Table 15-2: Projected Life Cycle Cost for Ocean Outfall

Description	Est. Life (Yrs)	Costs
Hydrodynamic Separator (2)	80	$638,000
3,000 Feet of 20" Outfall	100	$12,000,000
Permit Compliance & Annual Monitoring	100	$1,000,000
Estimated Cost (Concept Level)		**$13,638,000**
20% Concept Level Contingency		**$2,728,000**
Prelim. Construction Cost w/Contingency		**$16,366,000**

15.3.3 Performance

This alternative ranked well in the five categories identified under Table 3-1 (Criterion Ranking Points System). It would divert the 85th percentile of storm flows and effectively reduces the pollutant load from entering Kiddie Beach. Additionally, this option would require only standard operational and maintenance procedures with a typical pump station. This alternative would not discharge to an active public area, since the proposed discharge is several thousand feet from any beach head. However, since this option only displaces the bacterial load, it would not eliminate the pollutant load from discharge.

15.3.4 Public Agency / Regulatory Board Support

This alternative ranked low, as it did not meet any of the categories identified under Table 3-1.

15.3.5 Regulatory Requirements

Construction of the proposed pipeline and outfall would be located within the County of Ventura's Local Coastal Program Area and would require a CDP. Construction of the outfall would require Section 10 and CWA Section 404 permits from the U.S. Army Corps of Engineers, a CWA 401 Water Quality Certification, California Department Fish and Wildlife Streambed Alteration Agreement 1602, and a National Pollutant

Discharge Elimination System (NPDES) construction permit from the Los Angeles Regional Water Quality Control Board. The project would require an IS/MND or EIR. The outfall would trigger National Marine Fisheries Service (NMFS) and a U.S. Fish and Wildlife Service (USFWS) reviews. The following regulatory requirements for this alternative are summarized as follows:

Required (Highly Probable):

- Coastal Development Permit
- Initial Study / Mitigated Negative Declaration or Environmental Impact Report (EIR)
- NMFS Endangered Species Act Section 7
- USFWS Endangered Species Act Section 7
- U.S. Army Corps of Engineers Rivers and Harbors Act Section 10 and CWA Section 404
- LARWQCBCWA 401 Water Quality Certification and NPDES construction permit
- California Department Fish and Wildlife Streambed Alteration Agreement 1602

Potentially Required (Not Probable):

- NPDES construction permit and Ocean NPDES Discharge Permit

15.3.6 Public Perception and Impact

This alternative ranked average in the five categories identified under Table 3-1. It is expected that construction activities that greatly impact the use of the harbor and beach would not be well received by the general public. However, there would be no loss of parking, no street closures or increase of flood risks.

15.3.7 Constructability

This alternative ranked low in the three categories identified under Table 3-1. Underwater construction activities in the harbor and beach would be extremely challenging. Pipeline trenching would be submerged within the ocean, implying the challenges associated with water infiltration and soil instability. However, no third party infrastructure would need to be improved for this system to function as proposed.

15.3.8 Operations & Maintenance

This alternative ranked average in the three categories identified under Table 3-1. The anticipated maintenance would be relatively infrequent, there would be no specialized equipment necessary for replacement, nor any specialized training requirements. However, addressing repairs on the outfall pipeline would be difficult. Additionally, annual compliance monitoring would be required to confirm the discharge pipeline is not leaking within the extents of Kiddie Beach.

16 Feasibility Ranking Summary & Recommendation

16.1 Feasibility Ranking Summary

16.1.1 Alternatives Removed from Ranking Analysis

Of the nine alternatives covered within Sections 7 through 15, only five of these options were ranked. Four alternatives were eliminated from the following ranking matrices due to various significant factors, such as an inherent flaw concept, extremely high cost, or likely requiring a considerable amount of third-party agency infrastructure improvement. This topic is further discussed within each respective section for the nine alternatives.

As a summary, the **Divert to Sewer (Replace CIBCSD Pump Station & Force Main)** and to **Divert to Sewer (Lift Station 29 on Patterson Road)** alternatives were deemed infeasible as they would likely require a considerable amount of third-party agency infrastructure improvement. The **Store and Treat for Off-Site Reuse** and to **Treat for Off-Site Reuse** alternatives were deemed infeasible as no potential user of the treated water was identified. The remaining options, alternatives 1, 2, 7, 8 and 9, are ranked within the summary tables below.

16.1.2 Capital Cost

The sub-ranking of the five alternatives based on capital cost is summarized in Table 16-1. Note that from section 3.2.1, the weight for this criterion is (5) within the overall ranking matrix. The Sub-Rank score is input as well to demonstrate relative ranking between the alternatives for each of the following criterion.

Table 16-1: 85th Percentile Capital Cost

Capital Cost	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 1 – 5**				
Cost Estimate	**$5,340,000**	**$3,540,000**	**$5,760,000**	**$2,940,000**	**$8,160,000**
≤ $3,000,000 (+5)	0	0	0	5	0
>$3,000,000≤ $4,000,000 (+4)	0	4	0	0	0
>$4,000,000≤ $5,000,000 (+3)	0	0	0	0	0
>$5,000,000≤ $6,000,000 (+2)	2	0	2	0	0
>$6,000,000 (+1)	0	0	0	0	1
Sub-Weighted Score	**10**	**20**	**10**	**25**	**5**
Sub-Rank Score	**3**	**2**	**3**	**1**	**5**

16.1.3 50-Year Life Cycle Cost

The sub-ranking of the five alternatives based on the 50-year life cycle cost is summarized in Table 16-2. Note that the weight for the criterion in the overall ranking matrix is (3) and its calculation method is explained in Section 3.2.1.

Table 16-2: 85th Percentile 50-Year Life Cycle Cost

50 Year Life Cycle Cost	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 1 – 5**				
Cost Estimate	**$11,416,000**	**$8,138,000**	**$14,266,000**	**$6,923,000**	**$16,366,000**
≤ $7,000,000 (+5)	0	0	0	5	0
>$7,000,000≤ $10,000,000 (+4)	0	4	0	0	0
>$10,000,000≤ $13,000,000 (+3)	3	0	0	0	0
>$13,000,000≤ $16,000,000 (+2)	0	0	2	0	0
>$16,000,000 (+1)	0	0	0	0	1
Sub-Weighted Score	**9**	**12**	**6**	**15**	**3**
Sub-Rank Score	**3**	**2**	**4**	**1**	**5**

16.1.4 Performance

The sub-ranking of the five alternatives based on performance is summarized in Table 16-3. Note that from section 3.2.1, the weight for this criterion is (5) within the overall ranking matrix.

Table 16-3: 85th Percentile Performance

Performance	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 0 – 5**				
Captures/ Eliminates the 85th Percentile from Discharge to Kiddie Beach (+1)	1	1	0	1	1
Reduces Pollutant Load at Kiddie Beach (+1)	1	1	1	1	1
Improvements Will Not Discharge to Active Public Areas (+1)	1	1	1	0	1
Improvements Will Not Require Specialized Operation and Maintenance (+1)	1	1	0	1	1
Eliminates Pollutant Loads from Discharge (+1)	1	1	0	0	0
Sub-Weighted Score	**25**	**25**	**10**	**15**	**20**
Sub-Rank Score	**1**	**1**	**5**	**4**	**3**

16.1.5 Public Agency / Regulatory Board Support

The sub-ranking of the five alternatives based on public agency / regulatory board support is summarized in Table 16-4. Note that from section 3.2.1, the weight for this criterion is (5) within the overall ranking matrix.

Table 16-4: 85th Percentile Public Agency / Regulatory Board Support

Public Agency/Regulatory Board Support	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 0 – 5**				
Grant Competitiveness (+2)	0	2	0	0	0
Potential for Multiple Benefits (e.g., Water Recycling or Groundwater Recharge) (+1)	1	1	0	0	0
Permitted Under the Counties Local Coastal Plan (LCPs) (+1)	1	0	0	0	0
Qualified for Categorical Exclusions / Statutory Exemption With The California Coastal Commission (+1)	1	1	1	1	0
Sub-Weighted Score	**15**	**20**	**5**	**5**	**0**
Sub-Rank Score	**2**	**1**	**3**	**3**	**5**

16.1.6 <u>Regulatory Requirements</u>

The sub-ranking of the five alternatives based on regulatory requirements is summarized in Table 16-4. Note that from section 3.2.1, the weight for this criterion is (1) within the overall ranking matrix.

Table 16-5: 85th Percentile Regulatory Requirements

Regulatory Requirements	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 0 – 5**				
California Coastal Commission (-1)	0	0	0	0	-1
California Department Fish and Wildlife (-1)	0	0	0	0	-1
National Marine Fisheries Service (NMFS) (-1)	0	0	-1	0	-1
U.S. Fish and Wildlife Service (USFWS) (-1)	0	0	-1	0	-1
U.S. Army Corps of Engineers (-0.333)	-0.333	-0.333	-0.333	-0.333	-0.333
California Environmental Quality Act (CEQA) (-0.333)	-0.333	-0.333	-0.333	-0.333	-0.333
Regional Water Quality Control Board (RWQCB) (-0.333)	-0.333	-0.333	-0.333	-0.333	-0.333
Sub-Weighted Score	**4.0**	**4.0**	**2.0**	**4.0**	**0.0**
Sub-Rank Score	**1**	**1**	**4**	**1**	**5**

16.1.7 Public Perception and Impact

The sub-ranking of the five alternatives based on public perception / impact is summarized in Table 16-6. Note that from section 3.2.1, the weight for this criterion is (2) within the overall ranking matrix.

Table 16-6: 85th Percentile Public Perception / Impact

Public Perception/ Impact	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 0 – 5**				
Construction In The Harbor (-1)	0	0	0	0	-1
Construction In The Beach (-1)	0	0	0	0	-1
Loss of Parking (-1)	0	0	-1	0	0
Full Street Closure During Construction (-1)	0	0	0	0	0
Increased Flood Risk (-1)	0	-1	0	-1	0
Sub-Weighted Score	**10**	**8**	**8**	**8**	**6**
Sub-Rank Score	**1**	**2**	**2**	**2**	**5**

16.1.8 Constructability

The sub-ranking of the five alternatives based on public perception / impact is summarized in Table 16-7. Note that from section 3.2.1, the weight for this criterion is (3) within the overall ranking matrix.

Table 16-7: 85th Percentile Constructability

Constructability	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 0 – 5**				
Additional Third-Party Public Infrastructure Improvement (-1)	-1	0	0	0	0
Pipeline Trenching/ Underground Storage in High Groundwater Environment (-2)	-2	-2	-2	-2	-2
Underwater Construction in the Harbor (-2)	0	0	0	0	-2
Sub-Weighted Score	**6**	**9**	**9**	**9**	**3**
Sub-Rank Score	**4**	**1**	**1**	**1**	**5**

16.1.9 Operations & Maintenance

The sub-ranking of the five alternatives based on Operations & Maintenance is summarized in Table 16-8. Note that from section 3.2.1, the weight for this criterion is (3) within the overall ranking matrix.

Table 16-8: 85th Percentile Operations & Maintenance

Operations & Maintenance	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
Description	**Scoring Scale 0 – 5**				
Infrequent Maintenance (+1)	1	1	0	1	1
Alternative Does Not Require Replacing Specialized Equipment (+1)	1	1	0	1	1
Easy Access For Maintenance (+1)	1	1	1	1	0
Does Not Require Continuous Annual Regulatory Compliance Monitoring (+1)	1	1	0	1	0
Improvements Will Not Require Specialized Safety Training/ Procedures (+1)	1	1	0	1	1
Sub-Weighted Score	**15**	**15**	**3**	**15**	**9**
Sub-Rank Score	**1**	**1**	**5**	**1**	**4**

16.2 Recommendation

Of the five alternatives that are ranked through seven criteria, the most optimal and feasible options are the **Pump, Store, and Percolate (US Navy ROW)** and the **Divert to Santa Paula Pump Station** options (alternatives 2 & 8). The overall ranking of the five alternatives is summarized in Table 16-8. There are many reasons these options are favorable compared to the other options, the most noteworthy being the following:

- Large above-ground detention basin – not significantly impacted by local high-ground water for both the construction and operation of the storage facility (Alternative 2)
- Unlikely to have major hurdles to overcome in obtaining the required permits (Both alternatives)
- Meets TMDL objectives (Both alternatives)
- No loss in parking with the proposed below-ground pump station (Both alternatives)
- Redundancy by converting the existing dry weather diversion system to the CIBCSD from primary to backup instead (Both alternatives)
- Collaboration with the US Navy can improve competitiveness when seeking grants as a regional project (Alternative 2)

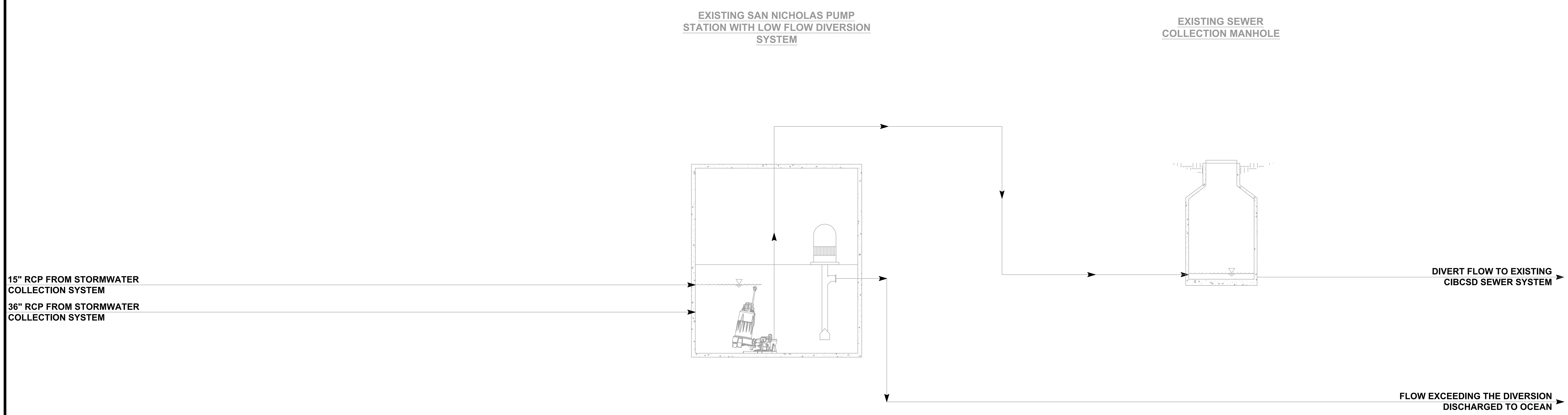
If the VCPWA-WP agrees with PACE's recommendation to convey up to 85th percentile wet weather flow to the US Navy right-of-way, or the Santa Paula Pump Station, PACE's next task will be to develop concept plans for both options.

While the Pump, Store and Percolate (Navy) option has the greatest score, some of the considerations that would negatively affect its feasibility were not addressed within Table 3-1 (Criteria Ranking Points System). These considerations include the potential for delays that occur with the legislative processes associated with federal agencies. Since PACE does not believe these concerns affect the feasibility of an alternative, they were not implemented within the overall ranking matrix. That said, to address these concerns, PACE recommends the top two alternatives so the County may ultimately make the decision between an option that incorporates a federal agency or not.

Table 16-9: 85th Percentile Evaluation Conclusion

Criteria	Weight	Store and Divert to CIBCSD Sewer (1)	Pump, Store and Percolate (Navy) (2)	Treat for Release (7)	Divert to Santa Paula (8)	SNPS Ocean Outfall (9)
		Weighted Scoring Scale 8-135				
Capital Cost	5	10	20	10	25	5
50 Year Life Cycle Cost	3	9	12	6	15	3
Performance	5	25	25	10	15	20
Public Agency/Regulatory Board Support	5	15	20	5	5	0
Constructability	3	6	9	9	9	3
Public Perception/ Impact	2	10	8	8	8	6
Regulatory Requirements	1	4.0	4.0	2.0	4.0	0.0
Operations & Maintenance	3	15.0	15.0	3.0	15.0	9.0
Total Weighted Score		**94.0**	**113.0**	**53.0**	**96.0**	**46.0**
Overall Rank		**3**	**1**	**4**	**2**	**5**

Appendix A: Hydraulic Process Flow Diagrams



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PROJECT ENGINEER
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CHECKED X.X.X. | DATE FEB 2022

TITLE
EXISTING PROCESS FLOW DIAGRAM

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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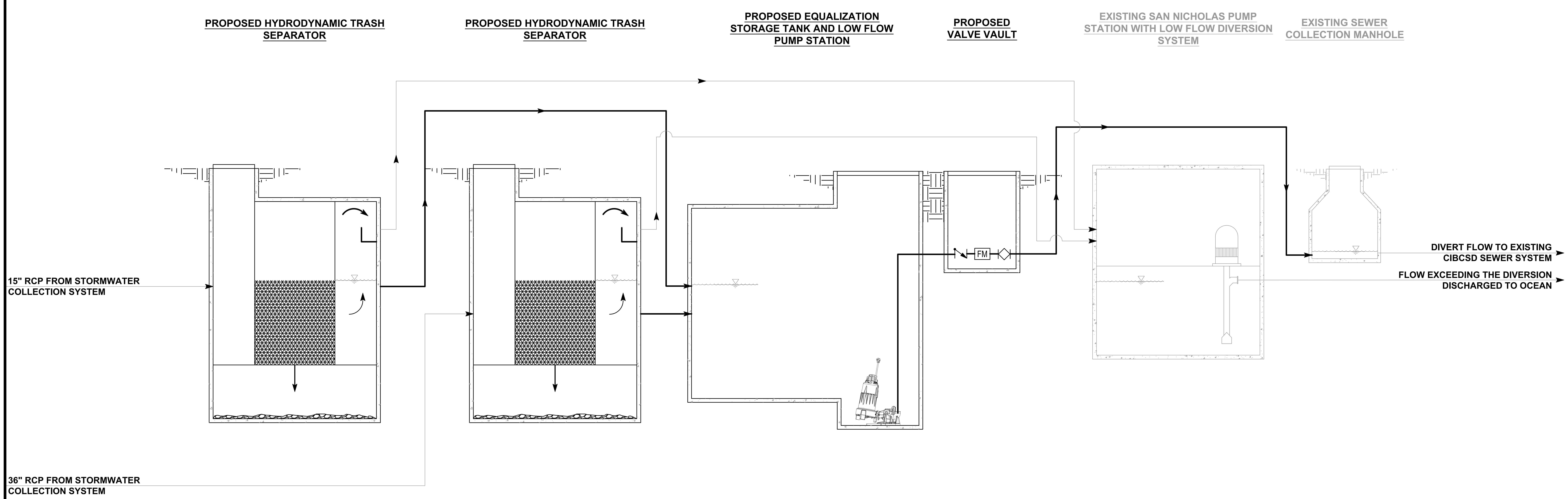

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DESIGNED X.X.X.
CHECKED X.X.X.
SCALE AS NOTED
DATE FEB 2022

TITLE
PROCESS FLOW DIAGRAM: STORE AND DIVERT TO CIBCSD SEWER (ALTERNATIVE 1)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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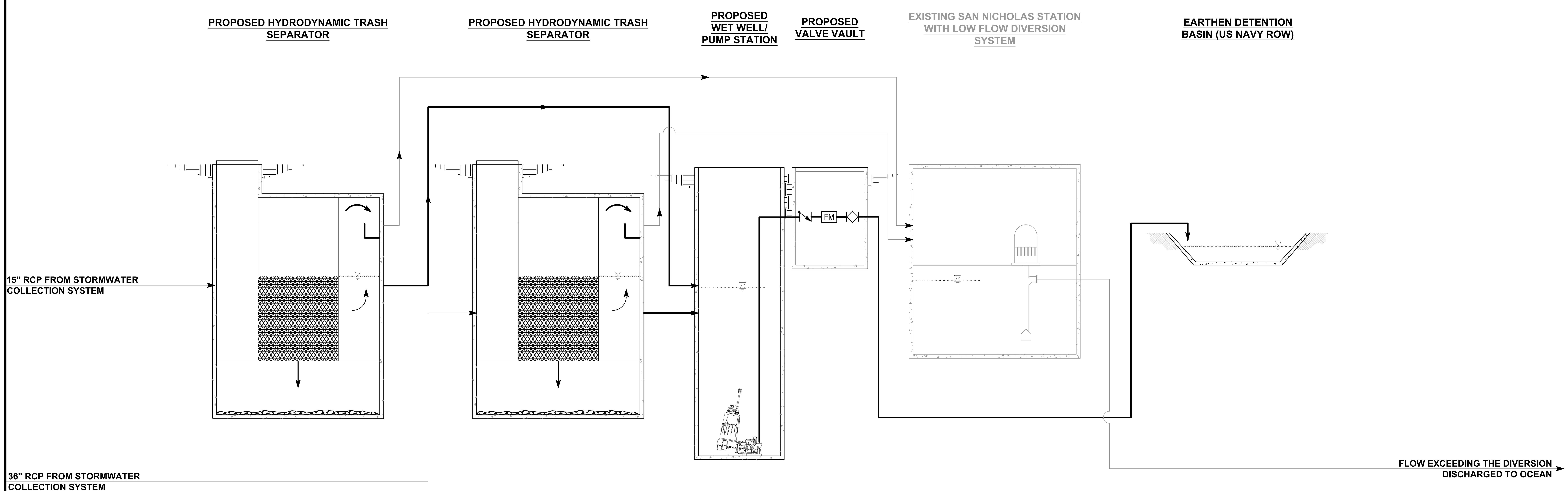

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TITLE
PROCESS FLOW DIAGRAM: PUMP, STORE AND PERCOLATE (US NAVY ROW) (ALTERNATIVE 2)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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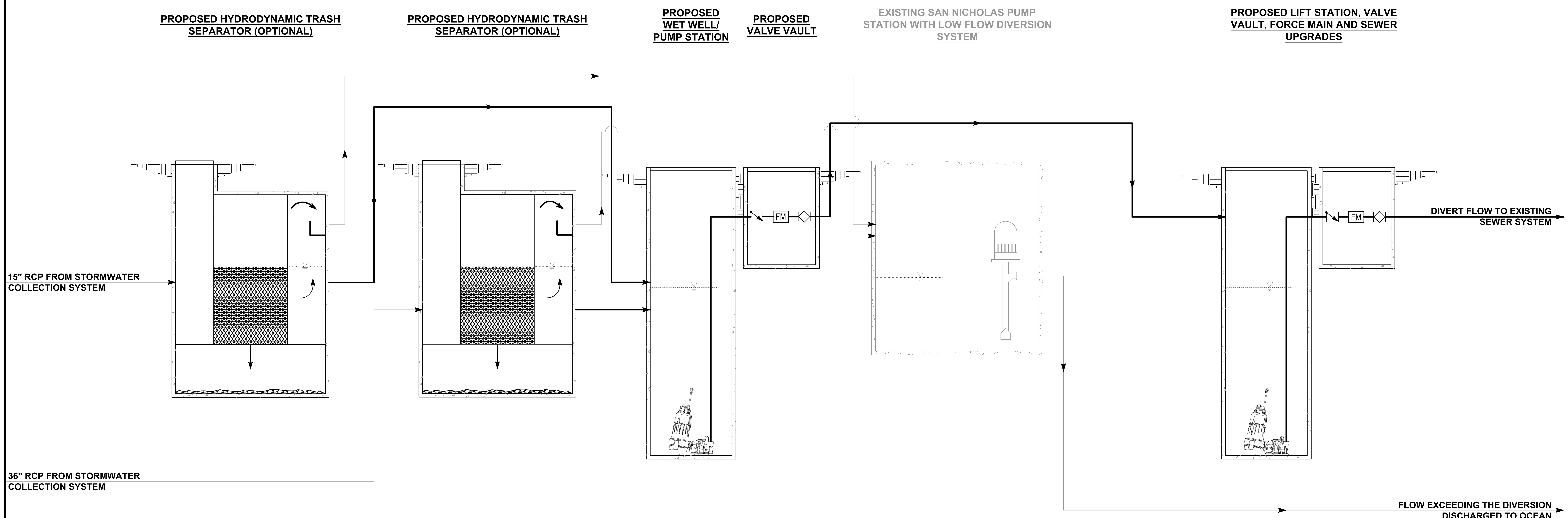

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TITLE
PROCESS FLOW DIAGRAM: DIVERSION TO SEWER (REPLACE CIBCSD PUMP STATION & FORCE MAIN) (ALTERNATIVE 3)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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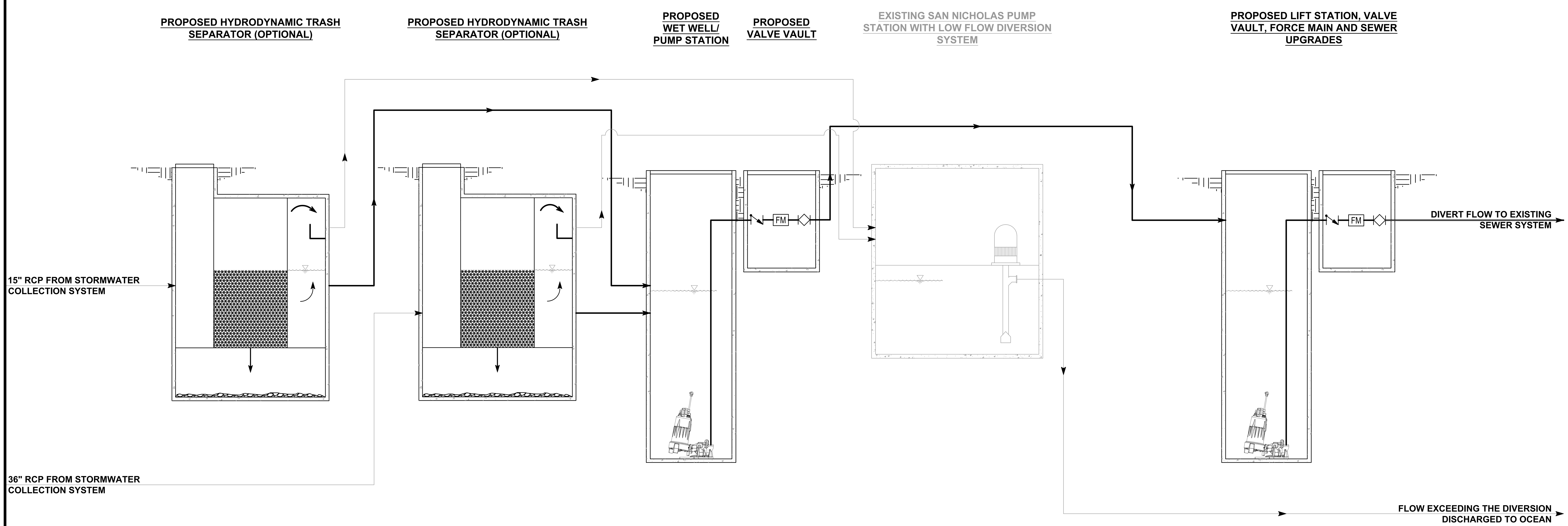

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TITLE
PROCESS FLOW DIAGRAM: DIVERT TO SEWER (LIFT STATION 29 ON PATTERSON ROAD) (ALTERNATIVE 4)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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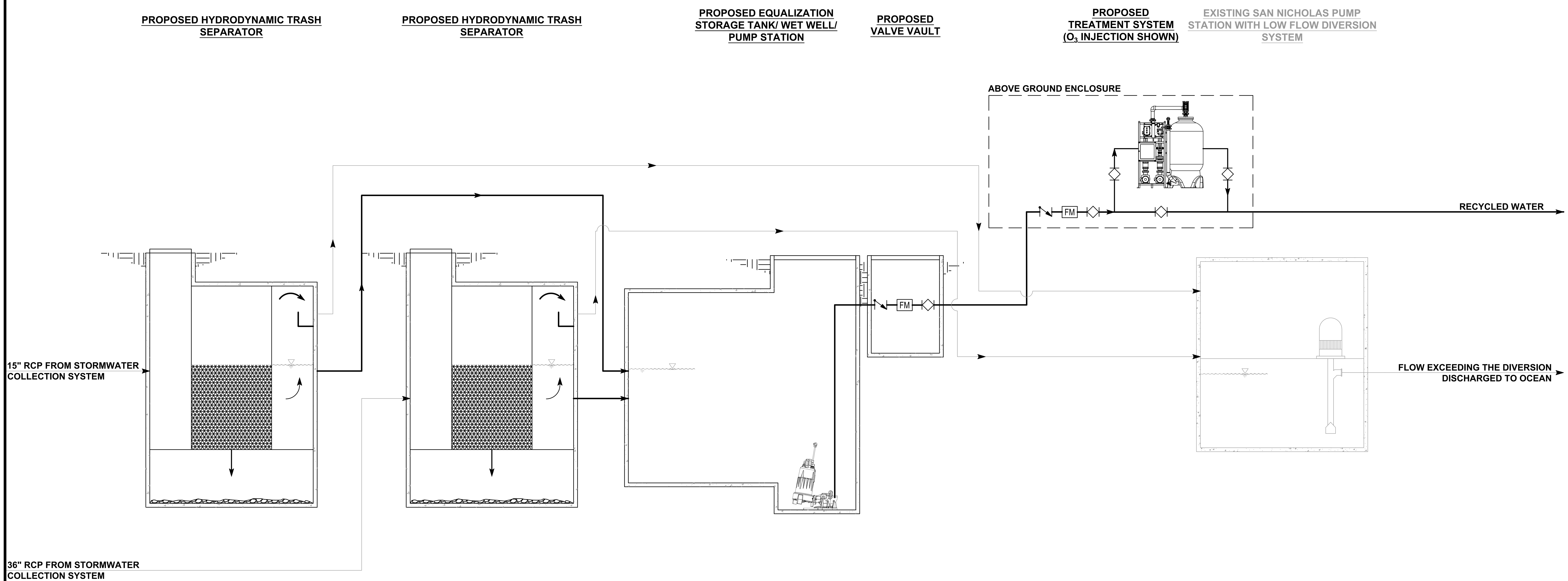

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EXP. XX-XX-XX

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DESIGNED X.X.X.
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DATE FEB 2022

TITLE
PROCESS FLOW DIAGRAM: STORE AND TREAT FOR OFF-SITE REUSE (ALTERNATIVE 5)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

PACE
Advanced Water Engineering
17520 Newhope Street, Suite 200 | Fountain Valley, CA 92708
P: (714) 481-7300 | www.pacewater.com

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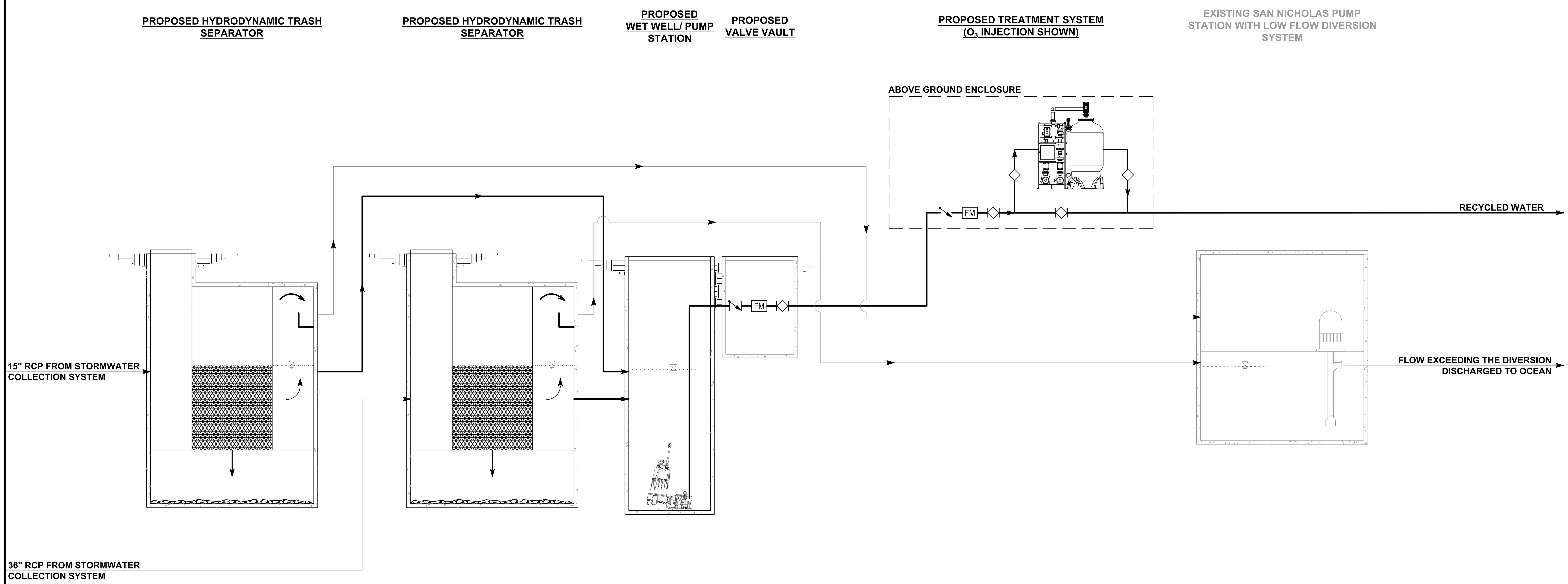
JOB NO. B804



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TITLE
PROCESS FLOW DIAGRAM: TREAT FOR OFF-SITE REUSE (ALTERNATIVE 6)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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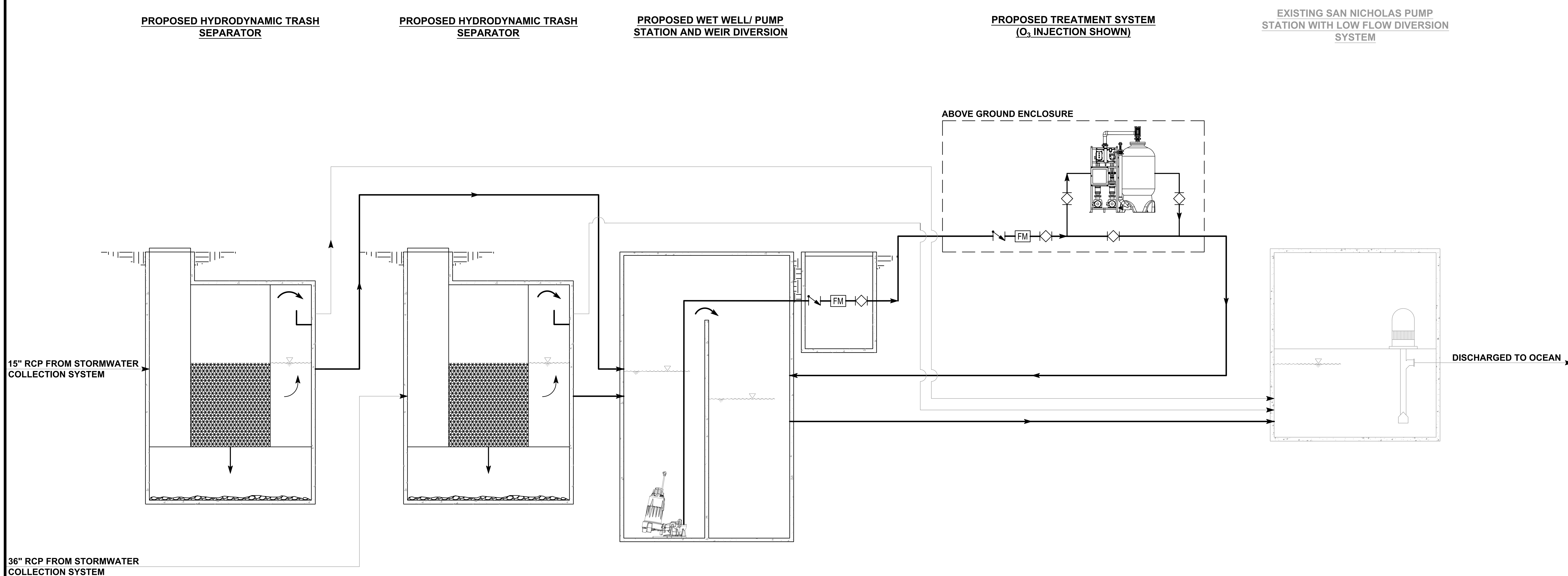

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TITLE
PROCESS FLOW DIAGRAM:
TREAT AND RELEASE
(ALTERNATIVE 7)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
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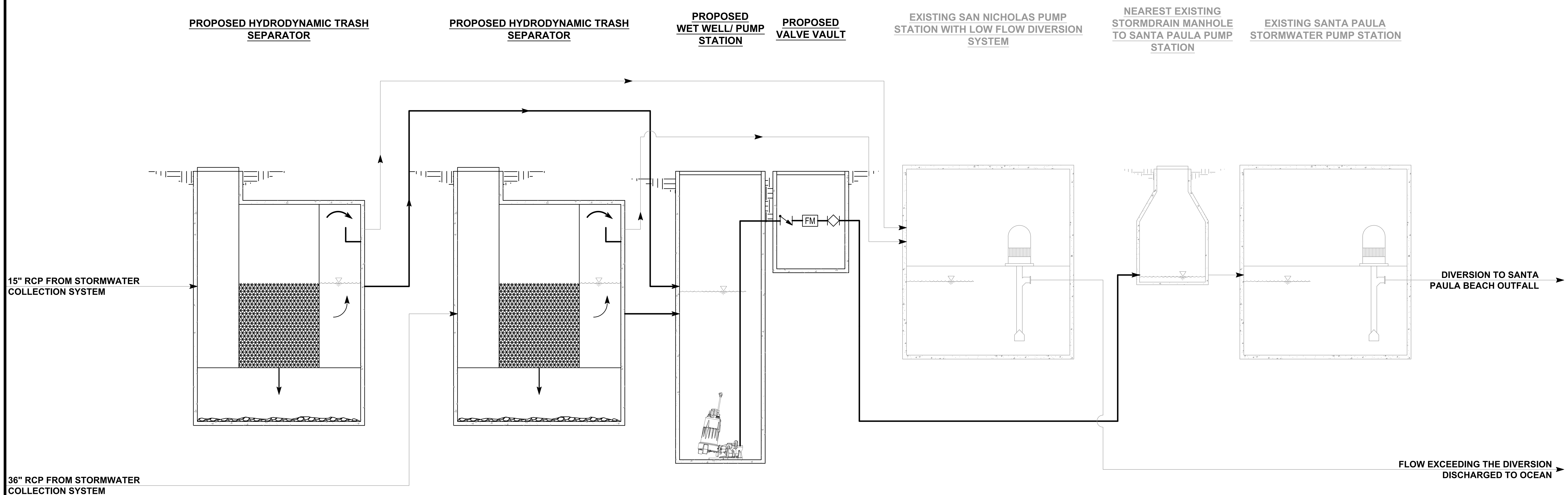

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TITLE
PROCESS FLOW DIAGRAM: DIVERT TO SANTA PAULA PUMP STATION (ALTERNATIVE 8)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
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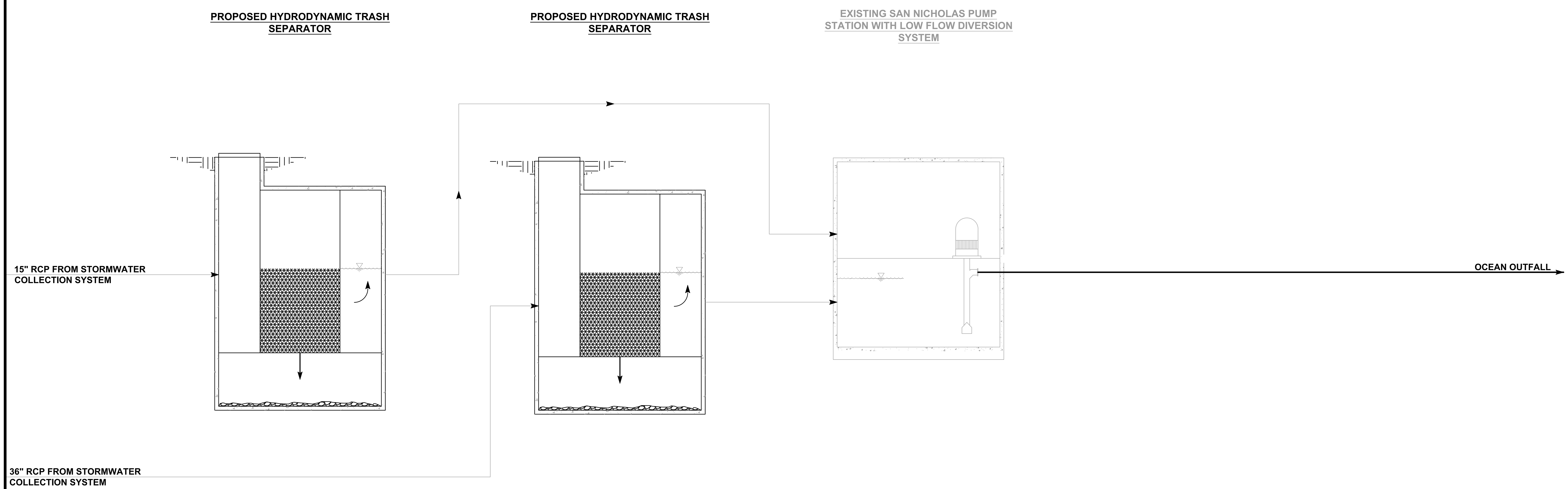

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TITLE
PROCESS FLOW DIAGRAM: SAN NICHOLAS PUMP STATION OCEAN OUTFALL (ALTERNATIVE 9)

JOB
VENTURA COUNTY PUBLIC WORKS AGENCY - WATERSHED PROTECTION
KIDDIE BEACH DISCHARGE FEASIBILITY STUDY
VENTURA CA

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Appendix B: CIBCSD Sewer Study

800 S Victoria Ave. Ventura, CA 93009

Kiddie Beach Sewer Capacity Study

Prepared for:

Ventura County Public Works Agency

Prepared by:

Pacific Advanced Civil Engineering, Inc.
17520 Newhope Street #200
Fountain Valley, CA 92708

August 29, 2022
#B804

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Appendix A – Standard Plan 040: Sewer Design General Requirements Criteria
Appendix B – US^3 Temporary Wastewater Flow Monitoring

List of Abbreviations

County	Ventura County
City Standards	City of Oxnard Standard Design Criteria
Client	Ventura County Public Works Agency
CFS	Cubic Feet per Second
D/d	Depth to Pipe Diameter Ratio
FPS	Feet per Second
ft	Feet
GIS	Geographical Information Systems
GPD	Gallons per Day
GPM	Gallons per Minute
in	Inches
MG	Million Gallons
MGD	Million Gallons per Day
MH	Manhole
PACE	Pacific Advanced Civil Engineering
Site	2974 S Victoria Ave, Oxnard, CA, 93035
TMDL	Total Maximum Daily Loads
US^3 or Sub Consultant	Utility Systems Science and Software, Inc.
VCP	Vitrified Clay Pipe

1. Introduction

1.1 Background

2974 S Victoria Ave, Oxnard, CA, 93035 (Site) – located on the corner of Victoria Avenue and Roosevelt Boulevard – is a 1.64-acre beach and parking lot which contains the San Nicholas Pump Station that pumps the Northernmost Silver Strand Community Watershed into the ocean. Currently, this station diverts up to 50 GPM of flow to the Channel Islands Beach Community Services District (CIBSD) sewer system during wet weather flows for treatment at the Oxnard Wastewater Treatment Plant (WWTP).

The CIBCSD sewers intake the diverted rain water via a diversion pump that discharges into the nearest manhole (MH020 as shown in **Figure 1-1**) located on San Nicholas Avenue. The sewage will continue to flow south east and then discharge into the existing lift station near the intersection of Highland Dr. and Roosevelt Blvd. PACE is tasked to prepare a sewer study to analyze that the existing sewer mains potentially have adequate available capacity to handle additional diversion flows from the proposed bacteria TMDL reduction feasibility alternative.

Five existing manholes (MH) along Victoria Ave., Roosevelt Blvd., and Highland Dr. were monitored per the CIBCSD from June 28, 2022 to July 13, 2022, a total of 17 days. MH056 and MH030 (**Figure 1-1**) are located downstream of MH020 on Roosevelt Blvd. MH056 is at the intersection of Roosevelt Blvd. and Melrose Dr., and MH030 is at the intersection of Roosevelt Blvd. and Cahuenga Dr. MH037 is located upstream at the intersection of Sunset Dr. and Victoria Ave. MH026 is located on Highland Dr. directly upstream of an existing lift station. MH060 is located at the intersection of Malibu Ave. and Island View Ave.

This project lies under the jurisdiction of the Ventura County Public Works Agency (County). The CIBCSD allowed monitoring of their sewer manholes to analyze the feasibility of increasing the diversion flows.

1.2 Report Objectives

The objectives of the 2974 S Victoria Ave Site Sewer Capacity Study are as follows:

- Provide a minimum 14-day continuous flow monitoring study at the indicated sewer Manholes.
- Monitor and analyze the average flow rate (Q_{avg}) from the Manhole outlets in accordance with typical municipal standards.
- Monitor and analyze the peak flow rate (Q_{peak}) from the Manhole outlets in accordance with typical municipal standards.
- Calculate and analyze the peak flow rate (Q_{peak}) without the stormwater diversion flow in accordance with typical municipal standards.
- Monitor and analyze the depth to diameter ratio (d/D) for the peak flow rate for the Manhole outlets in accordance with typical municipal standards.
- Calculate and analyze the depth to diameter ratio (d/D) without the stormwater diversion flow in accordance to typical municipal standards.

Figure 1-1: Ventura County Sewer Maps Exhibit 2721 S Victoria Ave, Oxnard, CA 93035

CHANNEL ISLANDS BEACH
COMMUNITY SERVICES DISTRICT
Legend
Sewer Manhole
Sewer CleanOut
Sewer Lift Station
TYPE
Force Main
Gravity
Sheet: 4
1 inch = 200 feet
0
100
200
Monitored MH037 (Victoria MH)
Storm Water Diversion Station
Monitored MH056 (Roosevelt MH)
Monitored MH030 (Cahuenga MH)
Lift Station "A"
Monitored MH026 (Highland MH)
Monitored MH060 (Malibu MH)
Lift Station
Channel Islands Harbor
MH040
MH038
MH039
MH037
MH036
MH016
MH035
MH034
MH020
MH056
MH033
MH017
MH004
MH085
MH029
MH028
MH032
MH018
MH057
MH058
MH030
MH031
MH024
MH059
MH003
MH027
MH026
MH025
MH019
MH060
MH063
MH062
MH061
MH021
MH013
MH053
8" VCP 224'
8" VCP 195'
8" VCP 451'
8" VCP 193'
8" VCP 325'
8" VCP 526'
8" VCP 167'
8" VCP 200'
8" VCP 214'
8" VCP 528'
8" VCP 263'
8" VCP 177'
8" VCP 536'
8" VCP 378'
8" VCP 206'
8" VCP 244'
8" VCP 544'
8" VCP 224'
8" VCP 433'
8" VCP 451'
8" VCP 258'
8" VCP 145'
8" VCP 429'
8" VCP 529'
8" VCP 293'
8" VCP 581'
4" PVC 48'
8" VCP 233'
8" VCP 194'
8" VCP 241'
8" VCP 149'
8" VCP 469'
8" VCP 341'
8" VCP 482'
8" VCP 228'
8" VCP 208'
8" VCP 184'
8" VCP 227'
8" VCP 195'
8" VCP 412'
8" VCP 488'
8" VCP 444'
8" VCP 641'
8" VCP 232'
10" VCP 253'
8" VCP 478'
10" VCP 228'
8" VCP 294'
10" VCP 183'
8" VCP 162'
8" VCP 284'
8" VCP 194'
8" VCP 210'
8" VCP 283'
8" VCP 118'
8" VCP 104'
Lake Shore Dr
Hollywood Blvd
Santa Monica Dr
Rossmore Dr
Cahuenga Dr
Highland Dr
Malibu Ave
Roosevelt Blvd
Panama Dr
West Rd
Harbor Blvd
Piru Ave
Lehman
Talos Rd
Kiddie Beach
Hobie Beach
Silver Strand Beach

2. Sewer Impact Analysis

2.1 Existing Sewer Conditions

The City Standards, provided as a reference to typical municipal standards, can be viewed in **Appendix A.** This establishes the minimum design and performance requirements for any sewer facility or collection system in the City. The capacity analysis entailed for the Site focuses on the collection system and whether or not the pipe capacity is sufficient enough to convey additional diversion flow. The following requirements taken from the City Standards provided the evaluation criteria for the sewer capacity study.

The current conditions of the monitored sewers are shown in **Table 2-1** below:

Table 2-1: Existing Sewer Conditions

Manhole	037	056	030	026	060
Number of Outlet Pipes	1	1	1	1	1
Outlet Pipe Diameter (in)	8	8	8	8	10
Number of Inlet Pipes	3	4	2	1	2
Inlet Pipe Diameters (in)	8, 8, 8	8, 8, 8, 8	8, 8	8	8, 10

2.1.1 Existing Sewer Conditions – Existing Peak Diverted Flow from Stormwater Runoff

At the range of the monitoring data, the diversion pumped up to 70 GPM of stormwater into the CIBCSD sewer system. This is equivalent to **0.101 MGD** of diverted stormwater when pumps are operational. The diversion is minimal as it typically starts & stops one (1) time daily, operates continuously for around 40 minutes, and only diverts approximately 3,000 gallons each day.

Table 2-2: Existing Diverted Stormwater Flow into Downstream Manholes

Manhole (MH)	Existing Diverted Stormwater Flow into Downstream Manholes (GPM/MGD)
037	-
056	**70 / 0.101**
030	**70 / 0.101**
026	-
060	-

2.1.2 Existing Sewer Conditions – Existing Average Monitored Flow from Sewer Mains

The 14-day continuous flow monitoring was successfully conducted from June 28th, 2022 to July 13th, 2022. The monitoring data, provided by Utility Systems Science and Software, Inc. (US^3), reported the existing average flow rates as shown in **Table 2-3**.

Table 2-3: Monitored Average Flow Rate from the Existing Sewer Mains

Manhole (MH)	Average Flow Rate Monitored, $Q_{average}$ (GPM/MGD)
037	17 / 0.025
056	22 / 0.031
030	67 / 0.097
026	8 / 0.011
060	63 / 0.091

2.1.3 Existing Sewer Conditions – Monitored Peak Flow from Sewer Mains

As conducted previously, the actual peak flow rates were measured during the 14-day period. The results are shown in **Table 2-4** below.

Table 2-4: Monitored Peak Flow Rate from the Existing Sewer Mains

Manhole (MH)	Peak Flow Rate Monitored, Q_{peak} (GPM/MGD)
037	42 / 0.061
056	93 / 0.134
030	180 / 0.259
026	86 / 0.124
060	228 / 0.329

According to a CIBCSD representative, MH027, located directly downstream of MH030, has been previously recorded to have temporary pipe inundation due to the existing Lift Station “A” operational procedures. According to the flow monitoring data at MH030, it showed the flow depth exceeded the pipe diameter during peak flow, implying pipe design capacity had been exceeded. However, monitoring data showed the velocity at the time the peak depth ratio was recorded was less than the average velocity monitored throughout the monitoring period. Typically, the average velocity would remain the same or increase with an increase in the depths of flow. However, since this is not the case with flow monitoring data at MH030, it appears occasionally inundated, similar to MH027.

2.1.4 Existing Sewer Conditions – Monitored Peak Flow Rate Without Stormwater Diversion

To analyze existing sewer capacity without stormwater diversion, the diversion flow was subtracted from the peak monitored flow. Compiling the diverted flow rate and the monitored flow rate from the 14-day flow monitoring, the total peak flow rate without diversion is obtained as shown in **Table 2-5**. The additional flow from the San Nicholas watershed is only removed from MH056 and MH030 as both are the only monitored manholes located downstream from the diversion station.

Table 2-5: Total Peak Flow Rate Without Stormwater Diversion

Manhole (MH)	Peak Diversion Flow Rate Calculated, Q_{peak} (GPM/MGD)	Peak Flow Rate Monitored, Q_{peak} (GPM/MGD)	Total Peak Flow Rate Without Stormwater Diversion, $Q_{total\ peak}$ (GPM/MGD)
037	-	42 / 0.061	**42 / 0.061**
056	70 / 0.101	93 / 0.134	**23 / 0.033**
030	70 / 0.101	180 / 0.259	**110 / 0.158**
026	-	86 / 0.124	**86 / 0.124**
060	-	228 / 0.329	**228 / 0.329**

3. Sewer Collection Design and Performance Requirements

3.1 Impact of Diversion on Pipe Capacity

3.1.1 Pipe Capacity – Depth to Diameter Ratio

One parameter commonly referenced for available pipe capacity is the depth-to-diameter ratio (d/D). While the stringency on the restrictions of this ratio varies per governing faction, it is commonly utilized as a method for analyzing available pipe capacity. It is for this reason that the peak depth-to-diameter ratios were monitored in addition to the flow rate. Per Section 40-2.2 in the City Standards, the design peak flow rate in pipes 10-inches and smaller is limited by a depth-to-diameter ratio of 0.5.

Note: d/D is the ratio of the calculated flow depth to the measurement of the pipes inside diameter.

To analyze the effect of the current stormwater diversion effect on the available pipe capacity, a calculation of the depth-to-diameter ratio was done without the additional 0.101 MGD of diversion flow. Since the diversion is flowing through MH056 and MH030, diversion flow was removed from these two manholes for the computation. However, as seen in red in **Table 3-1** below, MH030 was monitored to have a depth-to-diameter ratio exceeding one, implying a brief flooding period. Since the CIBCSD staff has already indicated to PACE that the nearby downstream MH027 has also been known to flood temporarily, this is likely to be the case at MH030. Since the method to calculate pipe capacity assumes a partially filled pipe, further information on the CIBCSD sewer system is required to analyze the current effect of diversion flows on Manhole 030. To determine the effect on flow capacity from diversion flows on Manhole 056, Manning's Equation was used at the calculate d/D without diversion flow, along with the following sewer design parameters provided by the City Standards:

$$Manning's\ Equation\ Q = \frac{1.49}{n}(A)\left(R_h{}^{2/3}\right)\left(S^{1/2}\right)$$

Where,

Q – Volumetric flow, cubic feet per second
n – roughness coefficient from the pipe. Per the City Standards, 0.013 for VCP.
A – Cross-sectional area of flow, square feet. Per the City Standards, the cross-sectional area is based on the maximum allowable depth.
R_h – Hydraulic radius of the pipe (Area/Wetted Perimeter), ft.
S – Slope of the Pipe, ft/ft.

Table 3-1 shows the given parameters for each manhole as well as the monitored depth ratios and calculated depth ratio for MH056. These results are based on the parameters and the flow data provided from US[3], shown in **Appendix B**. The existing average flow from each manhole and the corresponding flow depth was used to calculate a slope through Manning's Equation. After the diversion flow was removed from the peak monitored flow, a depth-to-diameter ratio was calculated with that flow rate and the calculated slope to analyze existing sewer capacity.

Table 3-1: Monitored Peak Depth Ratios and Calculated Depth Ratio without Diversion Flow

Manhole (MH)	Pipe Outlet Diameter (in)	Calculated Slope (ft/ft) from Average Flow	Monitored Peak Depth to Diameter Ratio (d/D)	Calculated Peak Depth to diameter ratio without diversion flow (d/D)	Allowable Depth Ratio (d/D)
037	8	0.0008	0.33	-	**0.50**
056	8	0.0018	0.50	**0.22**	**0.50**
030*	8	0.0067	1.11*	-	**0.50**
026	8	0.0011	0.57	-	**0.50**
060	10	0.0008	0.54	-	**0.50**

Note*: As mentioned previously, while MH030 has a monitored depth ratio that appears to be significantly higher than the allowable depth ratio per typical municipal standards, the conclusion on available pipe capacity at that manhole is currently nonconclusive.

According to the flow monitoring data, the d/D for the final remaining MH056 (see **Table 3-1** above) that receives dry weather diversion of 70 GPM is at the recommended threshold of 0.50. After removing this diversion flow from MH056, the corresponding calculated reduced peak flow rate went from **93 GPM (0.134 MGD)** to **23 GPM (0.033 MGD)**. The corresponding depth-to-diameter ratio went from **0.50** to **0.22**. Since the monitored d/D ratio is at the design limit, additional dry weather diversion is not recommended. Furthermore, PACE believes it is noteworthy to mention that while the peak d/D ratio recorded at MH056 was 0.50, the average ratio (which included dry weather diversion from the County) during the monitoring period was only **0.21**.

4. Sewer Capacity Study Results

4.1 Sewer Capacity Study Results

4.1.1 Existing Diversion Flow Results

In summary, the County diverts up to 70 GPM (0.101 MGD) of the San Nicholas Pump Station's dry weather runoff into the CIBCSD sewer system. However, diversion is minimal as it typically starts & stops one (1) time daily, operates continuously for around 40 minutes, and only diverts approximately 3,000 gallons each day.

The primary purpose of this study is to determine the remaining pipe capacity, if any, to accommodate any increase in the amount of daily diversion by the County. This study does not include any analysis of the available pump capacity of the existing CIBCSD Lift Station "A." While flow restrictions are dependent on the capabilities and level settings of the lift station, which would require additional information for capacity analysis, pipe capacity can be analyzed with respect to typical municipal standards. Since no information was provided on CIBCSD standards regarding pipe capacity, the City of Oxnard standards in **Appendix A** was used as a reference for typical municipal standards. These standards reference a pipe depth-to-diameter ratio for pipe capacity analysis. While this requirement depends on the individual governing agency, typical city standards require the depth-to-diameter ratio to be equal to or less than 0.5. To better understand the current dry weather diversion effect to the Lift Station "A," PACE performed a d/D calculation by removing the typical diversion flow rate of 70 gpm. The conclusion from this analysis is further described below.

4.1.2 Flow Monitoring Results and Discussion

Per US[3] flow monitoring in **Appendix B**, five (5) sewer manholes (see **Table 2-4** above) were monitored from June 28, 2022 to July 13, 2022, for a total of 17 days. MH037, MH026, and MH060 were not downstream from the dry weather diversion, so PACE did not perform any remaining pipe capacity analysis. As for one (1) of the two (2) remaining MH030 and MH056, flow monitoring data at MH030 showed the flow depth exceeded the pipe diameter during peak flow, implying pipe design capacity had been exceeded. However, monitoring data showed the velocity at the time the peak depth ratio was recorded was less than the average velocity monitored throughout the monitoring period. Typically, the average velocity would remain the same or increase with an increase in the depths of flow. However, since this is not the case with flow monitoring data at MH030, it appears occasionally inundated, similar to MH027 per a CIBCSD representative. According to the flow monitoring data, the d/D for the final remaining MH056 that receives dry weather diversion of 70 GPM is at the recommended threshold of 0.50. After removing this diversion flow from MH056, the corresponding calculated reduced peak flow rate went from **93 GPM (0.134 MGD)** to **23 GPM (0.033 MGD)**. The corresponding depth-to-diameter ratio went from **0.50** to **0.22**.

In conclusion, since the monitored d/D ratio is at the design limit, additional dry weather diversion is not recommended. Furthermore, PACE believes it is noteworthy to mention that while the peak d/D ratio recorded at MH056 was 0.50, the average ratio (which included dry weather diversion from the County) during the monitoring period was only **0.21**.

Appendix A – Standard Plan 040 – Sewer Design General Requirements Criteria

40 GENERAL SEWER SYSTEM DESIGN GOALS AND ACCEPTABLE PROCEDURES

40-1 GENERAL REQUIREMENTS

The design and construction of sanitary sewers in the City of Oxnard shall be in accordance with good engineering practice. The work shall comply with these design goals except where specific modifications have been approved by the Public Works Director in writing. The Director shall decide all questions of interpretation of " Good Engineering Practice ". All work on sewers and sewer service laterals outside of City right - of - way or sewer easements will be governed by the provisions of the Uniform Plumbing Code. Where City requirements and standards are more restrictive than U.P.C., the City requirements shall govern. Where purveyor's requirements are more restrictive than these standards, the purveyor's requirements shall govern.

40-2 SPECIFIC REQUIREMENTS

40-2.1 VELOCITY :

The velocity of flow (averaged over the wetted cross-section) for sanitary sewers flowing part-full or full should be between 2.0 f.p.s. and 10.0 f.p.s. The most commonly used formula is Manning's , which is :

$$V = \frac{1.486}{n} R^{2/3} \; S^{1/2} \quad \text{in f.p.s.}$$

Where n is roughness coefficient (see sect. 40-4)
R is hydraulic radius
S is energy gradient . For open channels, uniform flow condition it is equal to invert slope.

Discharge $Q = V A$ in c.f.s.

Where V = Velocity of flow in f.p.s.
A = Wetted cross-sectional area in sq.ft.

Also
Q g.p.m. = (Qc.f.s.) ×(448.83)

40-2.2 FLOW DEPTH

a) For pipe 10" or less in diameter :
Design pipe so that peak flow rate will be carried when pipe is flowing at one-half (1/2) depth. Discharge at one-half depth equals one-half discharge when full and velocity equals velocity when full.

b) For pipe 12" and larger in diameter :
Design pipe so that peak flow rate will be carried when pipe is flowing at two-third (2/3) depth. Discharge at 2/3 depth equals 3/4 discharge when full and velocity equals 1.16 times velocity when full.
In no case gravity sewer lines will be designed to flow full or pressurize the system.

REV.	APPR. BY	DATE

40-3 MINIMUM STREET SEWER SIZE

40-3.1 Minimum street sewer size shall be 8", except that 6" pipe may be used where all of the following conditions are met :
(a) The minimum invert slope shall be 0.008.
(b) The length shall not exceed 200' with no possibility of future extension.
(c) No more than 10 house laterals contribute to the 6" diameter reach.
(d) Minimum cover of line shall be 5.0feet.

40-4 MINIMUM INVERT SLOPE :
Slope of sewer invert shall equal or exceed those set forth in the following table. For case of checking maximum flow capacity at these minimum slope is given for V.C.P. (n= 0.013) and P.V.C. (n=0.011) in c.f.s. and g.p.m.

TABLE – 1

PIPE DIAMETER	MINIMUM SEWER INVERT SLOPE	MAXIMUM FLOW CAPACITY IN c.f.s. (g.p.m.)	
		V.C.P.	P.V.C.
6"	0.0060	0.218 (97.7)	0.257 (115.5)
8"	0.0040	0.383 (171.8)	0.452 (203.0)
10"	0.0028	0.581 (260.6)	0.686 (308.0)
12"	0.0020	1.250 (561.0)	1.477 (663.0)
14"	0.0020	1.885 (846.2)	2.228 (1000.0)
15"	0.0016	2.027(909.8)	2.396 (1075.2)
16"	0.0016	2.408 (1080.6)	2.845 (1277.0)
18"	0.0016	3.296 (1479.4)	3.895 (1748.4)
20"	0.0012	3.781 (1696.8)	4.468 (2005.3)
21"	0.0012	4.306 (1932.6)	5.089 (2284.0)
24"	0.0012	6.148 (2759.2)	7.265 (3260.9)
27"	0.0012	8.416 (3777.4)	9.946 (4464.1)
30"	0.0012	11.146(5002.7)	13.173 (5912.3)
33"	0.0012	14.372(6450.4)	16.985 (7623.2)
36"	0.0012	18.125 (8135.0)	21.420 (9614.0)

REV. APPR. BY DATE

CITY OF Oxnard

GENERAL REQUIREMENTS – SEWER

DRAWN: SOHER CKD. Jay Patel APPR. BY Benjamen Y. Wong

Public Works Department

STANDARD PLAN

PLATE 41

SHEET OF

Substandard slopes below the minimum slopes listed in table -I may be used in order to avoid pumping only upon specific approval of the City Engineer . Such approval should be solicited well in advance of completion of design.

41 DESIGN CRITERIA

41-1 AVERAGE SEWAGE FLOW RATES

The average flow rate shall be determined by the developer's Engineer based on good engineering practice . Sewage flows shall be determined from the potential land use of the tributary area. Average sewage flow rates were developed for various land use and anticipated population density and given in term of G.P.M./Acre The currently accepted values are given in Table on Plate 44 These flow rates should be used for new development and determining effects of future land use per general plan. Acreage in table is gross acreage including roads , yards, parking , etc. For estimating the sewage flows for specific land use the flow rate value given in Table on Plate 43.

41-2 PEAK SEWAGE FLOW RATES

The rates between peak flow to average flow shall be determined by using following information

41-2.1 For average flow up to 1C.F.S.

$(\text{Peak flow , c.f.s.}) = 2.0 \times (\text{Average flow ,c.f.s.})^{0.822}$

41-2.2 For average flow greater than 1 C.F.S.

$\text{Peaking factor} = 2.0 \times (\text{Average flow, c.f.s.})^{-0.1}$

The graphical representation of above equations is given on plate 45 . This should be used in designing sewer system in the City of Oxnard.

REV.	APPR. BY	DATE

CITY OF Oxnard	GENERAL REQUIREMENTS — SEWER		STANDARD PLAN
	DRAWN:	CKD. Jay Patel	APPR. BY Benjamen Y. Wong	PLATE 42
	Public Works Department		SHEET OF

Appendix B – US³ Temporary Wastewater Flow Monitoring

Methods & Procedures & Equipment

Methods and Procedures

Utility Systems Science & Software provided PACE with an off the shelf, non-proprietary flow monitoring solution that included five state of the art Hach Flo-Dar® AV Sensor systems. The project course of action is listed below. The US^3 team:

- Assessed permitting and traffic control at the sites on Victoria Av and Roosevelt Blvd in Oxnard, CA.
- Installed and removed traffic control in accord with site-specific California Temporary Traffic Control Handbook (CATTCH) requirements for both the installation and removal of equipment.
- Validated the sites for suitability for sewer flow monitoring.
  - Manhole (MH) 37 had two inlets from the east and north with a third drop inlet entering from the east. The site had slow to moderate open channel hydraulics and some turbulence due to inflow from the lateral.
  - MH 56 had three inlets from the east, north and NW with a fourth drop inlet entering from the east. The site had slow to moderate open channel hydraulics with turbulence due to inflow from the laterals.
  - MH 30 had two inlets from the east and the NW with moderate open channel hydraulics and turbulence due to inflow from the lateral and calcium deposits.
  - MH 26 had no laterals with slow to moderate open channel hydraulics.
  - MH 60 had two inlets from the south and the west with slow to moderate open channel hydraulics and some turbulence due to inflow from the lateral.
- Installed and calibrated the flow monitoring equipment at the sites per manufacturer recommendations on 6/28/2022.
  - Follow-up on the installations confirmed equipment was reading properly.
  - Collected 15-minute interval depth and velocity data points over the entire monitoring period.
- Removed the equipment on 7/13/2022 and validated the data.
  - All of the equipment went through diagnostic testing before and after the study with less than a 1% deviation between manual and meter level readings and less than a 5% deviation between manual and meter velocity readings.
  - Equipment calibration was verified in accordance with manufacturer specifications.
- Prepared the data reports.
  - The table below contains a summary of the average (Avg) and maximum (Max) velocities (Vel) and levels (Lev) collected during this study as well as the calculated flow rates (Flow) and depth versus diameter ratios (d/D).

MH	Pipe Size (in)	Avg Vel (fps)	Max Vel (fps)	Avg Lev (in)	Max Lev (in)	Avg Flow (gpm)	Max Flow (gpm)	Avg d/D	Max d/D
37	8	0.65	1.09	1.84	2.63	17.70	42.48	0.23	0.33
56	8	0.85	1.76	1.65	4.02	21.31	93.01	0.21	0.50
30	8	2.03	3.31	2.10	8.91	67.08	179.88	0.26	1.11
26	8	0.53	1.49	1.13	4.52	7.89	86.05	0.14	0.57
60	10	0.87	1.72	3.20	5.43	63.20	228.78	0.32	0.54

Equipment

Figure above: Web-Enabled Flo-Dar® AV Sensor, Radar-Based Velocity/Area Flow Meter

FloDar® AV Sensor Specifications:

- **Enclosure**
    - IP68 Waterproof rating, Polystyrene
- **Dimensions**
    - 160.5 W x 432.2 L x 297 D mm (6.32 x 16.66 x 11.7 in.),
    - With SVS, D = 387 mm (15.2 in.)
- **Weight**
    - 4.8 kg (10.5 lbs.)
- **Operating Temperature**
    - -10 to 50°C (14 to 122°F)
- **Storage Temperature**
    - -40 to 60°C (-40 to 140°F)
- **Power Requirements**
    - Supplied by FL900 Flow Logger, Flo-Logger, or Flo-Station
- **Interconnecting Cable**
    - Disconnect available at both sensor and logger or Flo-Station
    - Polyurethane, 0.400 (±0.015) in. diameter; IP68
    - Standard length 9 m (30 ft), maximum 305 m (1000 ft)
- **Cables – available in two styles:**
    - connectors at both ends
    - connector from sensor with open leads to desiccant hub, desiccant hub with connector to logger. A potting/sealant kit will be included. This can be used to run the cable through conduit.
- **Certification**
    - Certified to: FCC Part 15.245: FCC ID: VIC-FLODAR24
    - Industry Canada Spec. RSS210. v7: IC No.: 6149A-FLODAR24

SURCHARGE DEPTH MEASUREMENT

- Auto zero function maintains zero error below 0.5 cm (0.2 in.)
- **Method**
    - Piezo-resistive pressure transducer with stainless steel diaphragm
- **Range**
    - 3.5 m (138 in.), overpressure rating 2.5 x full scale

VELOCITY MEASUREMENT

- **Method**
    - Radar
- **Range**
    - 0.23 to 6.10 m/s (0.75 to 20 ft/s)

- **Frequency Range**
    - 24.075 to 24.175 GHz, 15.2 mW (max.)
- **Accuracy**
    - ±0.5%; ±0.03 m/s (±0.1 ft/s)

DEPTH MEASUREMENT

- **Method**
    - Ultrasonic
- **Standard Operating Range from Flo-Dar® Housing to Liquid**
    - 0 to 152.4 cm (0 to 60 in.)
- **Optional Extended Level Operating Range from Transducer Face to Liquid**
    - 0 to 6.1 m (0 to 20 ft.) with 43.18 cm (17 in.) dead band, temperature compensated.
- **Accuracy**
    - ±1%; ±0.25 cm (±0.1 in.)

FLOW MEASUREMENT

- **Method**
    - Based on Continuity Equation
- **Accuracy**
    - ±5% of reading typical where flow is in a channel with uniform flow conditions and is not surcharged, ±1% full scale max.

SURCHARGE CONDITIONS DEPTH/VELOCITY DEPTH (Std with Flo-Dar® Sensor)

- **Surcharge depth supplied by Flo-Dar® sensor.**

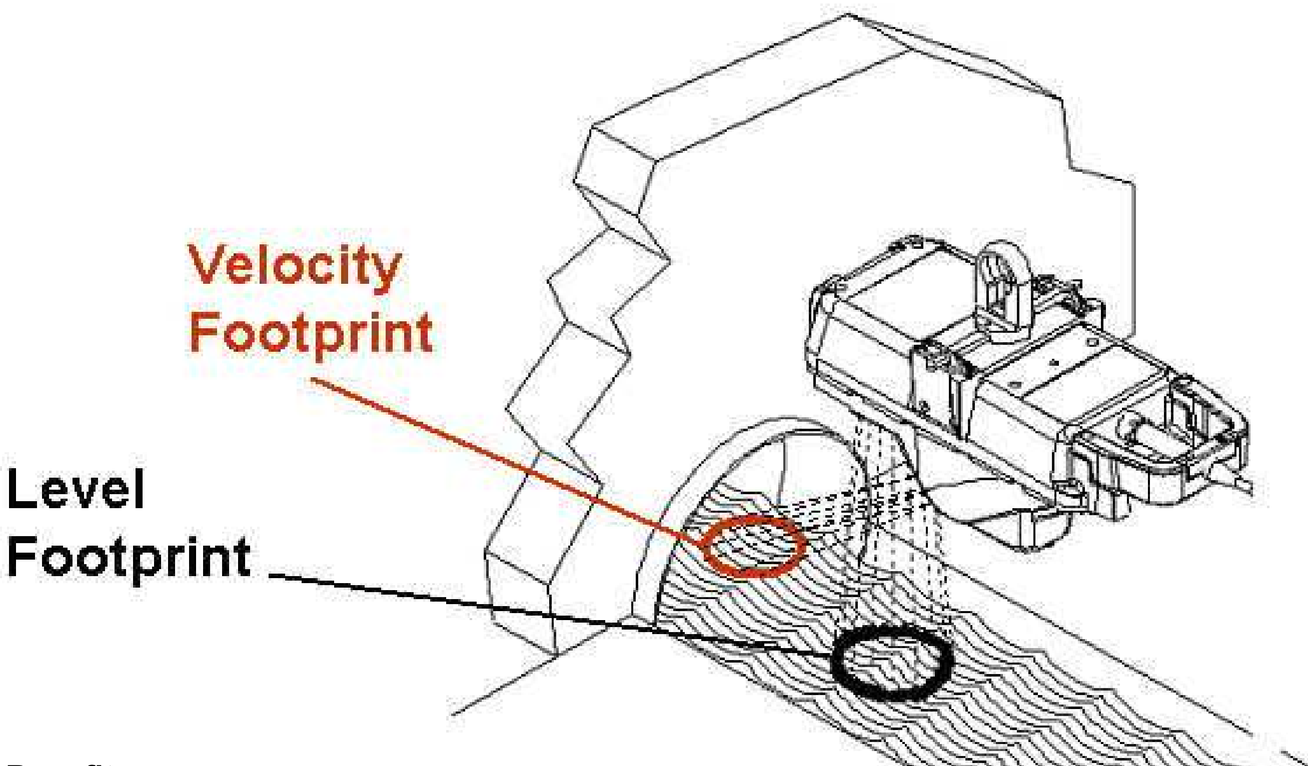
VELOCITY (Optional Surcharge Velocity Sensor)

- **Method**
    - Electromagnetic
- **Range**
    - ±4.8 m/s (±16 ft/s)
- **Accuracy**
    - ±0.15 ft/s or 4% of reading, whichever is greater.
- **Zero Stability**
    - ±0.05 ft/s

The Flo-Dar® Open Channel Flow Meters provide an innovative approach to open channel flow monitoring. Combining digital Doppler radar velocity sensing with ultrasonic pulse echo level sensing Flo-Dar® provides accurate open channel flow monitoring without the fouling problems associated with submerged sensors.

Perfect Solution for Difficult Flow Conditions:

- Flows with High Solids Content
- High Temperature Flows
- Caustic Flows
- Large Man-Made Channel
- High Velocities
- Shallow Flows



Benefits:

1. Personnel have no contact with the flow during installation.
2. Maintenance caused by sensor fouling is eliminated
3. Field Replaceable/Interchangeable Sensors and Monitors

How It Works

Flo-Dar® transmits a digital Doppler radar beam that interacts with the fluid and reflects back signals at a different frequency than that which was transmitted. These reflected signals are compared with the transmitted frequency. The resulting frequency shift provides an accurate measure of the velocity and the direction of the flow. Level is detected by ultrasonic pulse echo. Flow is then calculated based on the Continuity Equation:

$$Q = V \times A, \text{ Where } Q = \text{Flow}, V = \text{Average Velocity and } A = \text{Area}$$

Accurate Flow Measurements

Flo-Dar® provides the user with highly accurate flow measurements under a wide range of flows and site conditions. By measuring the velocity of the fluid from above, Flo-Dar® eliminates accuracy problems inherent with submerged sensors including sensor disturbances, high solids content and distribution of reflectors.

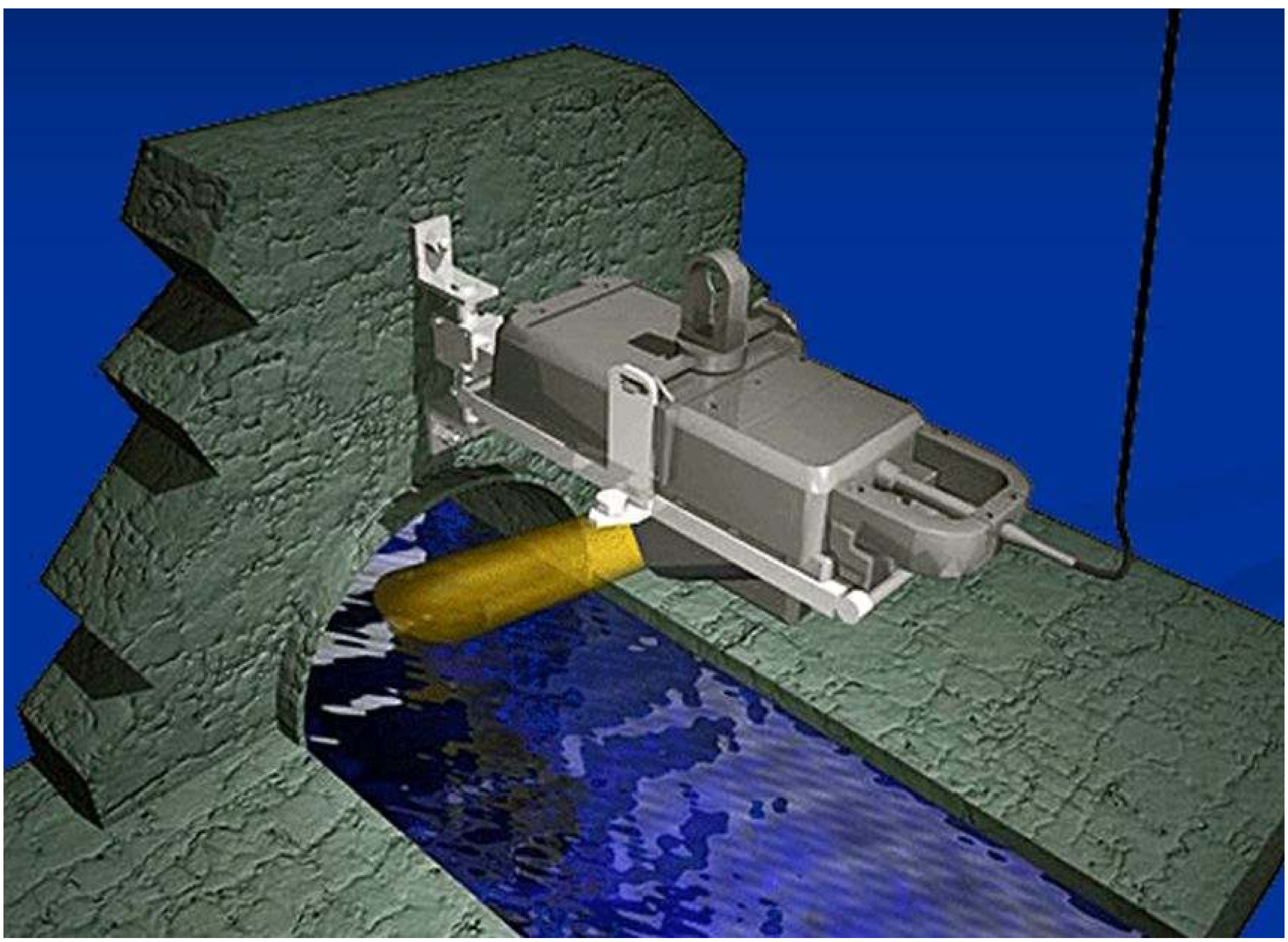


Figure above: US³ utilizes exclusively Hach March-McBirney Flo-Dar® Meters

US^3 Company Information

US^3 is a California Corporation **Federal ID No. 33-0729605** and qualifies as a Minority Business Enterprise. US^3 has certified as an MBE with the California Public Utility Commission's authorized clearinghouse, **Verification Number: 97ES0008.**

US^3 is a specialty service company for the Water & Waste Water industry, providing monitoring and control for Utilities since 1996. US^3 is in the forefront of this industry by taking the proven technological approaches developed in other high-tech industries and applying them to protect one of our most precious natural resources - our water.

US^3 engineers and technical personnel have applied advanced instrumentation system technology to water/wastewater open channel flow monitoring, pipeline evaluation, engineering, and data analysis, all coupled to the power of the Internet. This unique integrated systems approach allows the company to bring greater insight and intelligence to gathering information about water/wastewater system performance of our clients, and in turn, to support the fulfillment of their commitments to manage and cost effectively design, operate, and maintain these systems.

Moreover, ***US^3*** supports Municipalities, Consulting Engineering firms and other water/waste water systems integrators by providing temporary technical services for engineering, software programming and technical site maintenance and calibration site support work, primarily in the Water and Waste Water industries.

Figure at right: All US^3 technicians are certified for Confined Space Entry.

Key Personnel Assigned

US³ provided the necessary resources to fully implement this project. Primary in support of this effort were the following personnel:

Mr. Mark Serres: Mr. Serres is a degreed electrical engineer with over 25 years of experience with fresh/wastewater systems, project management, and systems integration in relation to complex industrial systems. This includes experience in industrial automation and water/wastewater industries. Mr. Serres is responsible for assuring client satisfaction and marshalling the required resources to meet the project requirements.

Mr. Thomas Williams: Mr. Williams is an Engineering Manager with over 20 years of experience in complex systems development for wastewater monitoring. This experience includes hydraulic compatibility, instrumentation, communications and analysis. Mr. Williams is responsible for assuring that the required equipment is designed and calibrated to meet the project requirements.

Darlene Szczublewski, PE: Mrs. Szczublewski is a licensed Civil Engineer in multiple states. She has over 15 years of engineering experience with stormwater/wastewater related projects. She assisted in the completion of several Sanitary Sewer Evaluation Surveys and Capacity Analysis projects to meet Consent Decrees as well as completing numerous Infiltration and Inflow (I&I) studies for other clients. Mrs. Szczublewski has developed numerous flow data analysis techniques to present a clear informative picture of flow in a monitored system. Her work also includes the development of training programs for clients describing I&I and capacity analysis methodologies. Mrs. Szczublewski is responsible for analyzing the data as well as the data collection process and assuring that the reports meet the project requirements.

Name, title, address and telephone number of persons to contact regarding this US³ project.

Darlene Szczublewski, PE
Senior Civil Engineer
darlene.szczublewski@uscubed.com

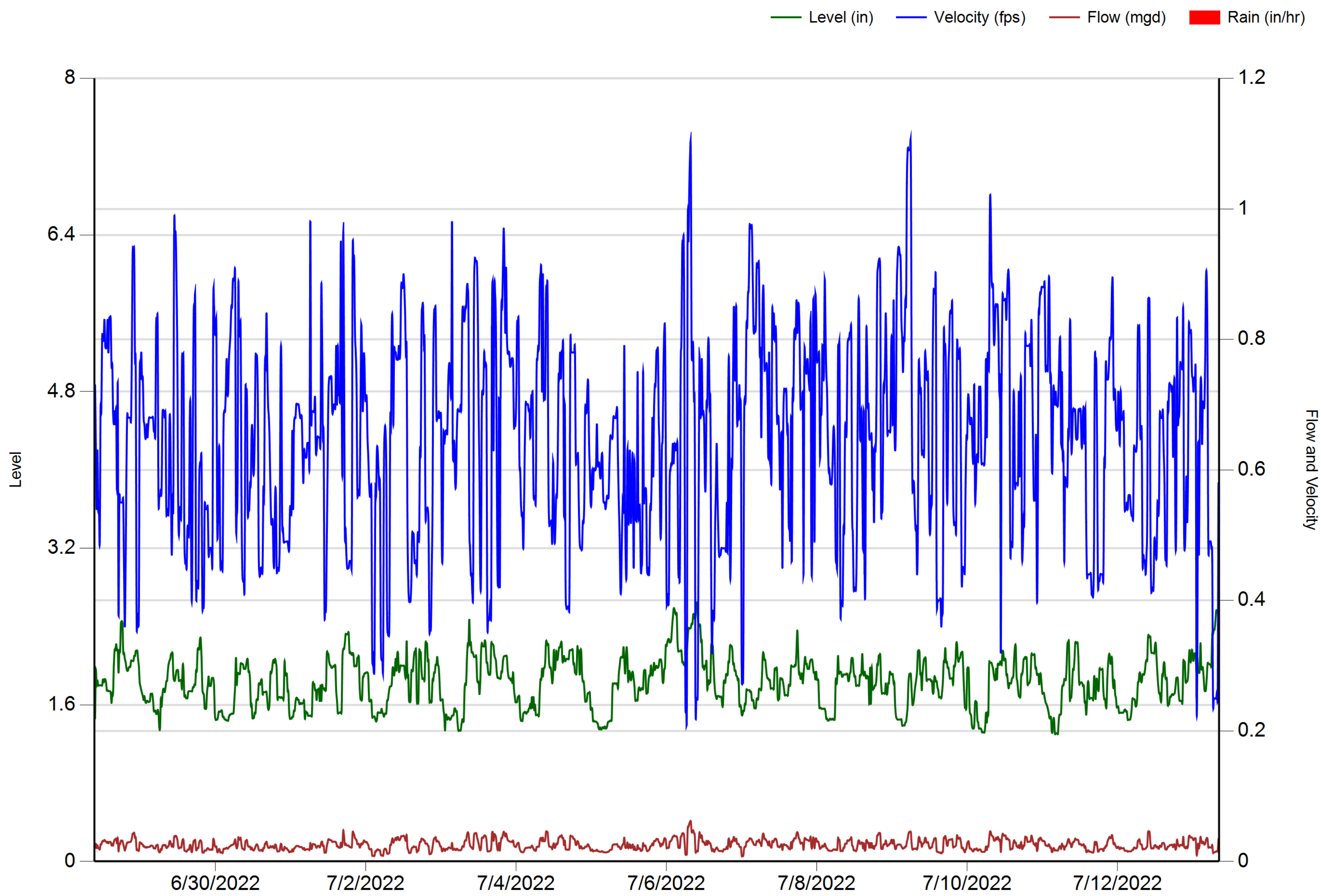
9314 Bond Av, Suite A
El Cajon, CA 92021
619-546-4281 (work)
619-246-5304 (cell)

Tom Williams
Engineering Manager
tom.williams@uscubed.com

9314 Bond Av, Suite A
El Cajon, CA 92021
619-546-4281 (work)
619-398-7799 (cell)

2022.06 Site 1 Victoria MH:
06/28/2022 thru 07/13/2022

Report Date: 07/27/2022
Customer: PACE
Group: CIBCSD Project
Site: 2022.06 Site 1 Victoria MH



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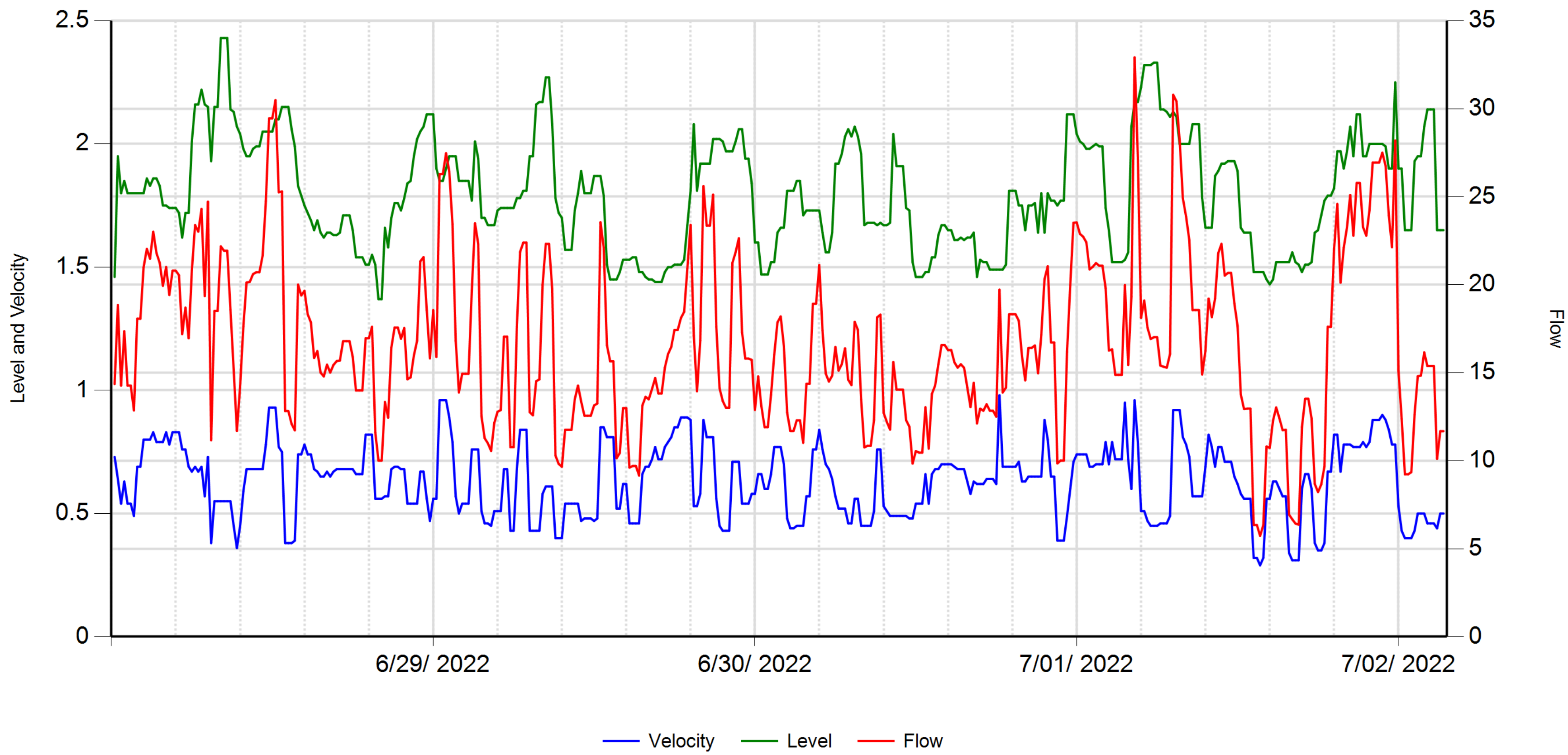
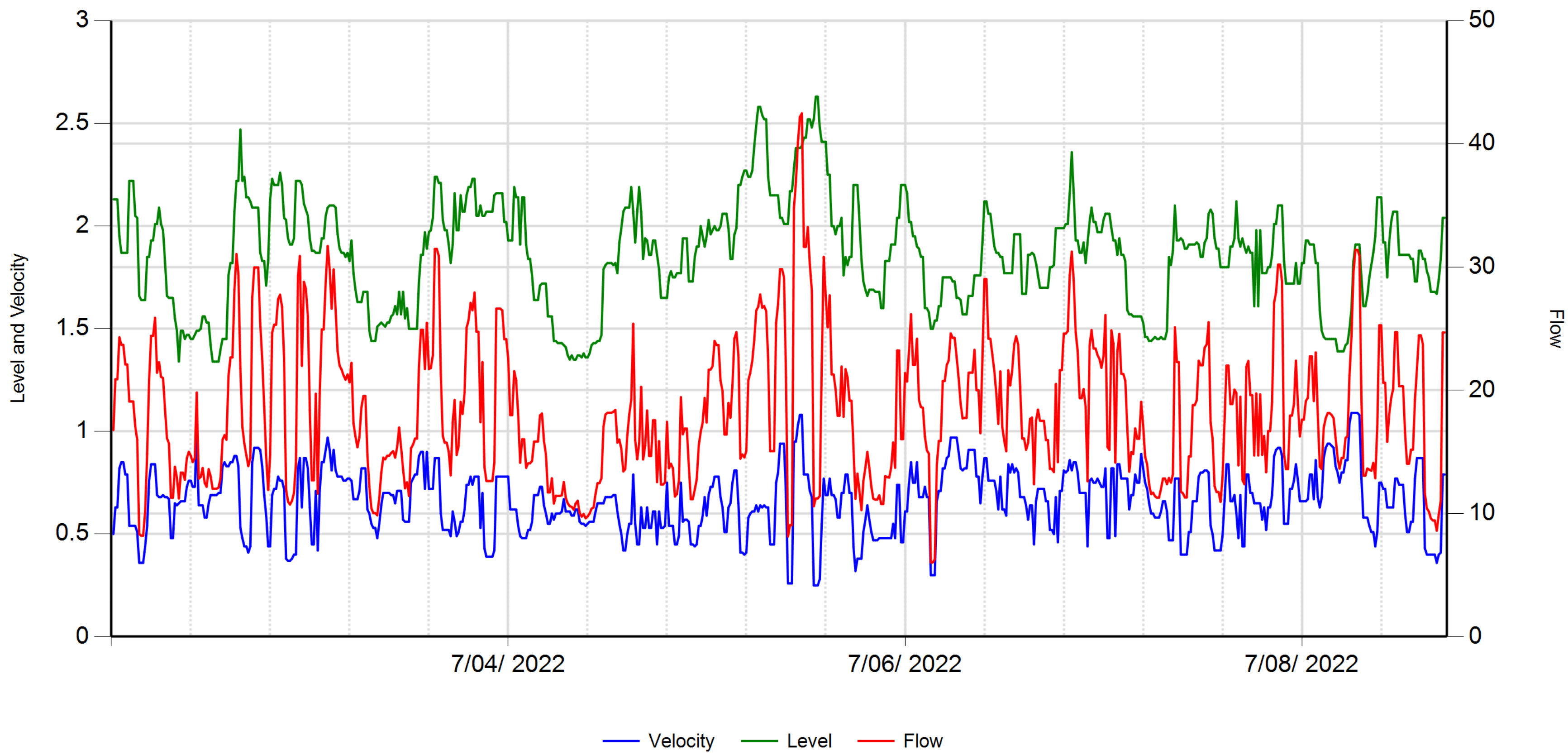


	Velocity (fps)	Level (in)	Flow (gpm)	RainFall	Inches
Average	0.631	1.800	16.673		
Maximum	0.980	2.430	32.920		
Minimum	0.290	1.370	5.730		

7/27/2022

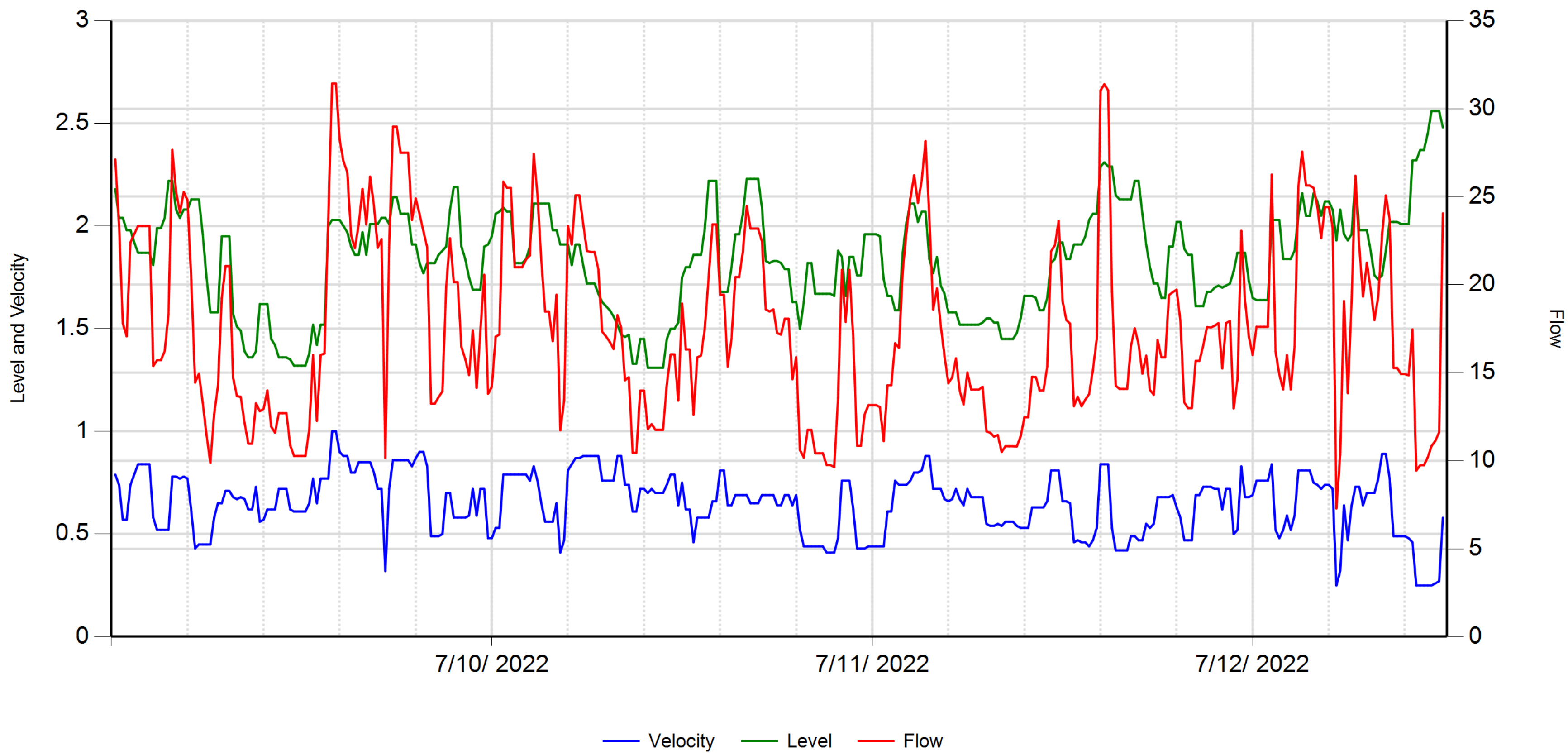
2022.06 Site 1 Victoria MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.662	1.860	18.349	**RainFall**	Inches
Maximum	1.090	2.630	42.480		
Minimum	0.250	1.340	6.040		

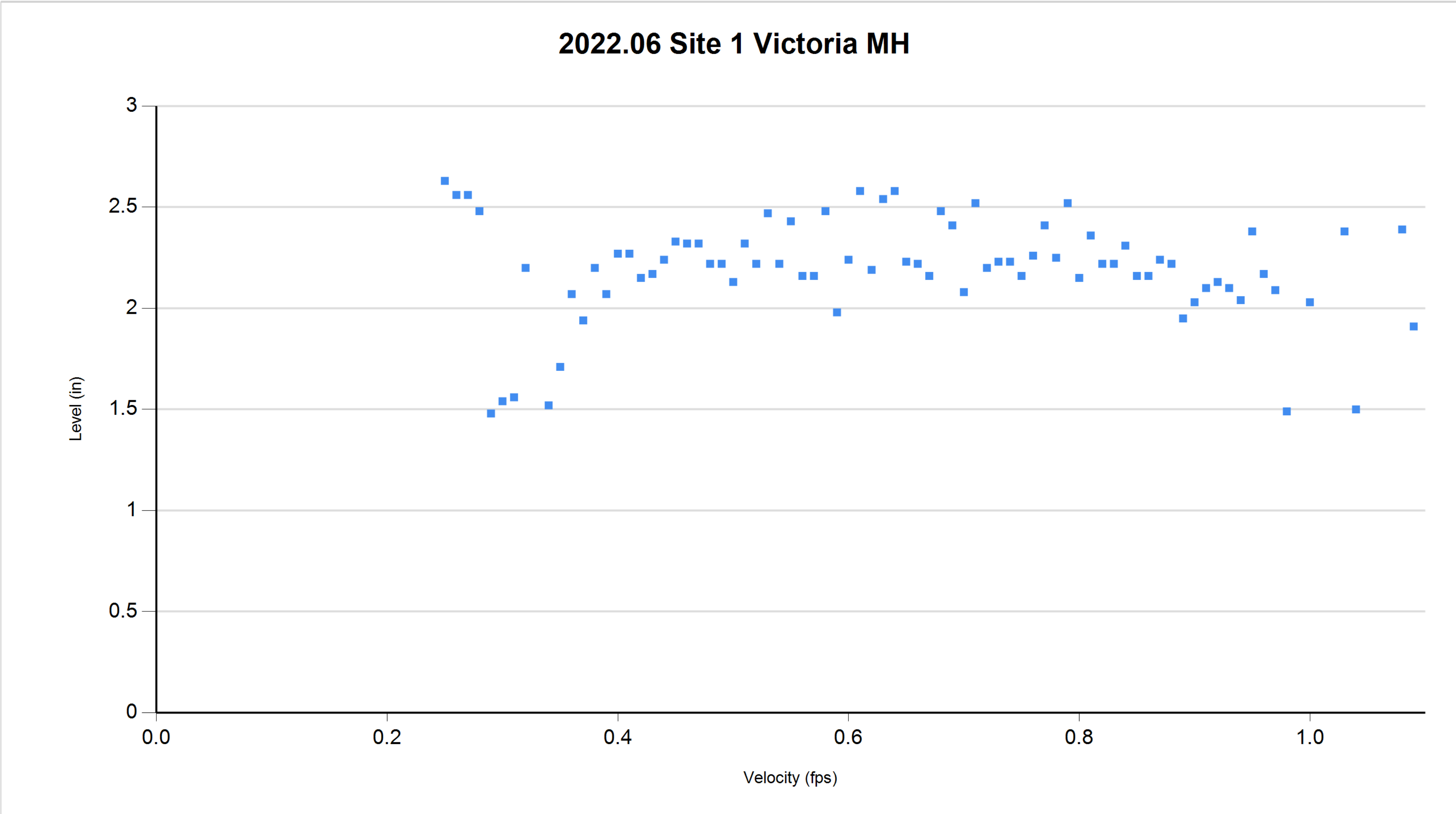
7/27/2022

2022.06 Site 1 Victoria MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.652	1.838	17.670	**RainFall**	Inches
Maximum	1.000	2.560	31.430		
Minimum	0.250	1.310	7.290		

7/27/2022



6/28/2022 thru 7/13/2022

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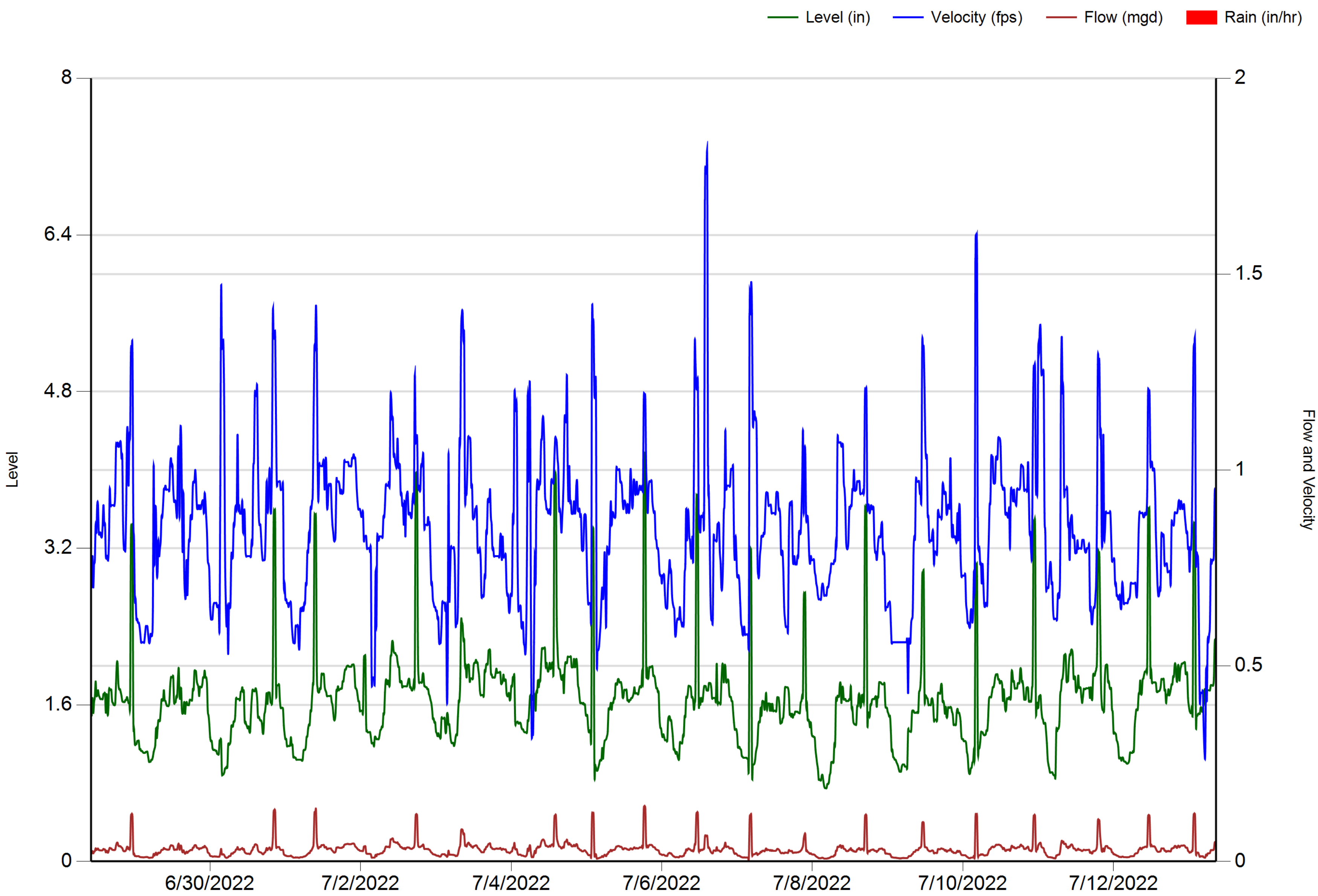
Report Date: 07/27/2022
Customer: PACE
Group: CIBCSD Project
Site: 2022.06 Site 2 Roosevelt MH

2022.06 Site 2 Roosevelt MH:
06/28/2022 thru 07/13/2022



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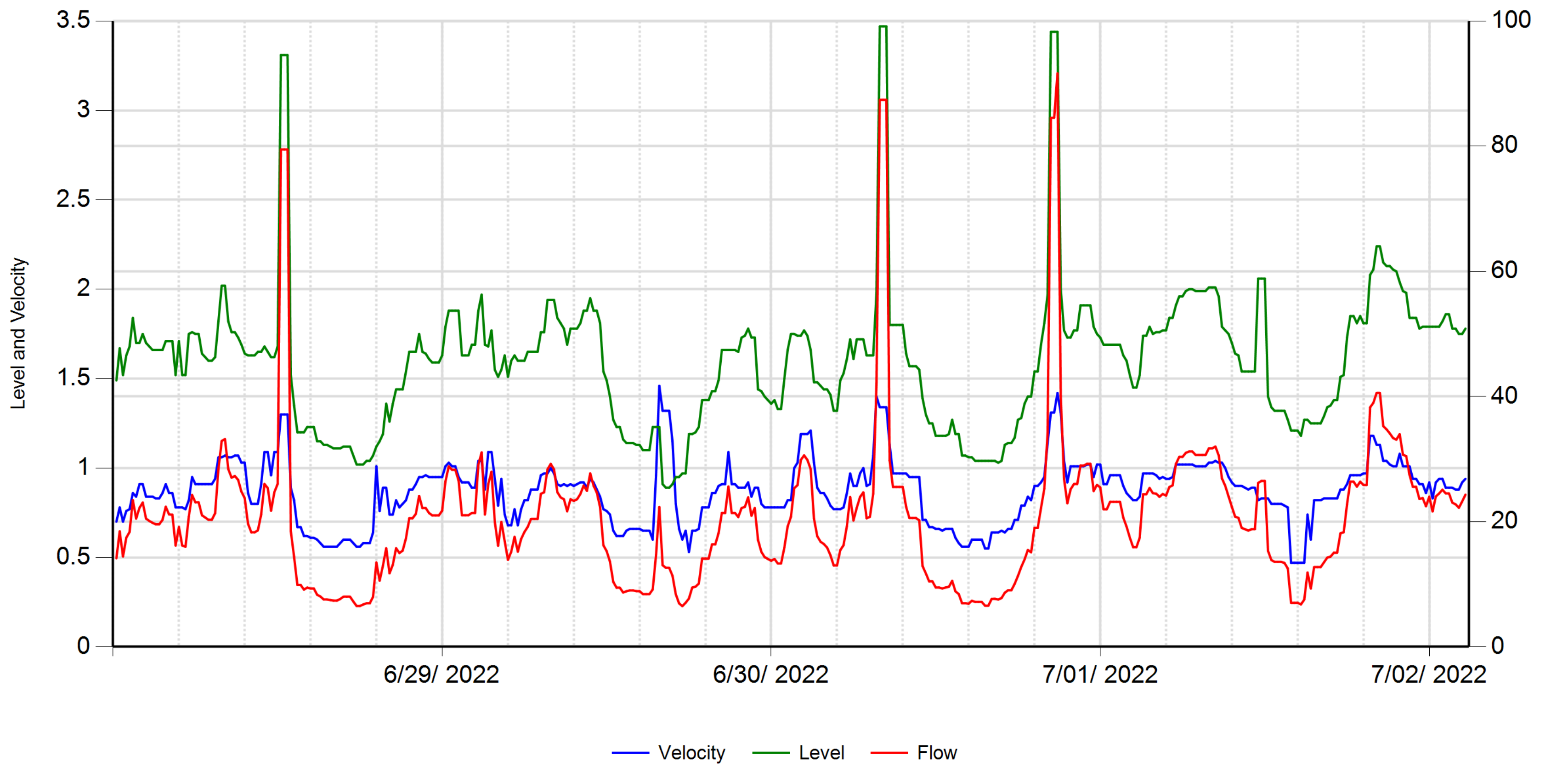
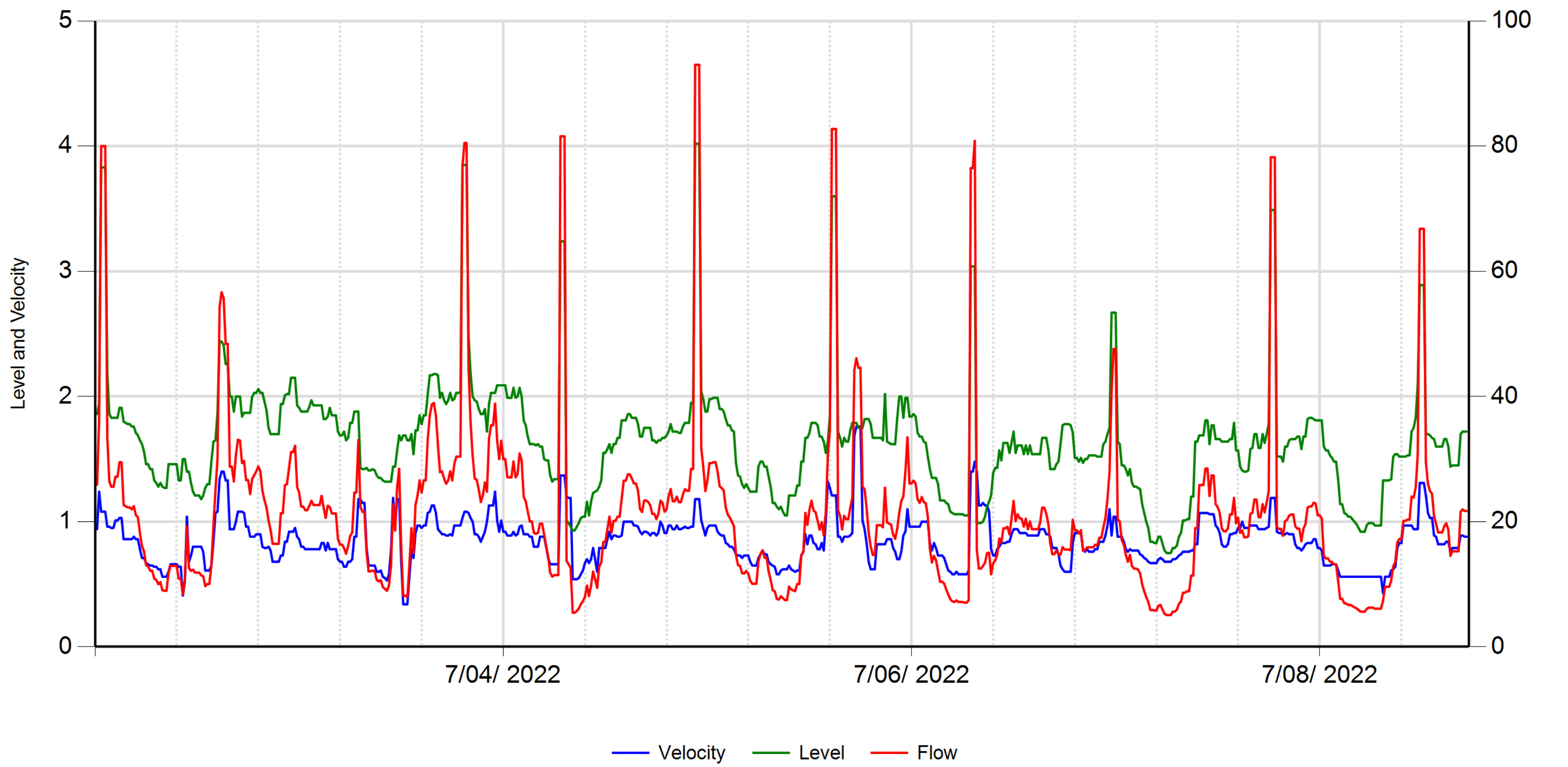


	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.872	1.602	20.836	**RainFall**	Inches
Maximum	1.460	3.470	91.640		
Minimum	0.470	0.890	6.510		

7/27/2022

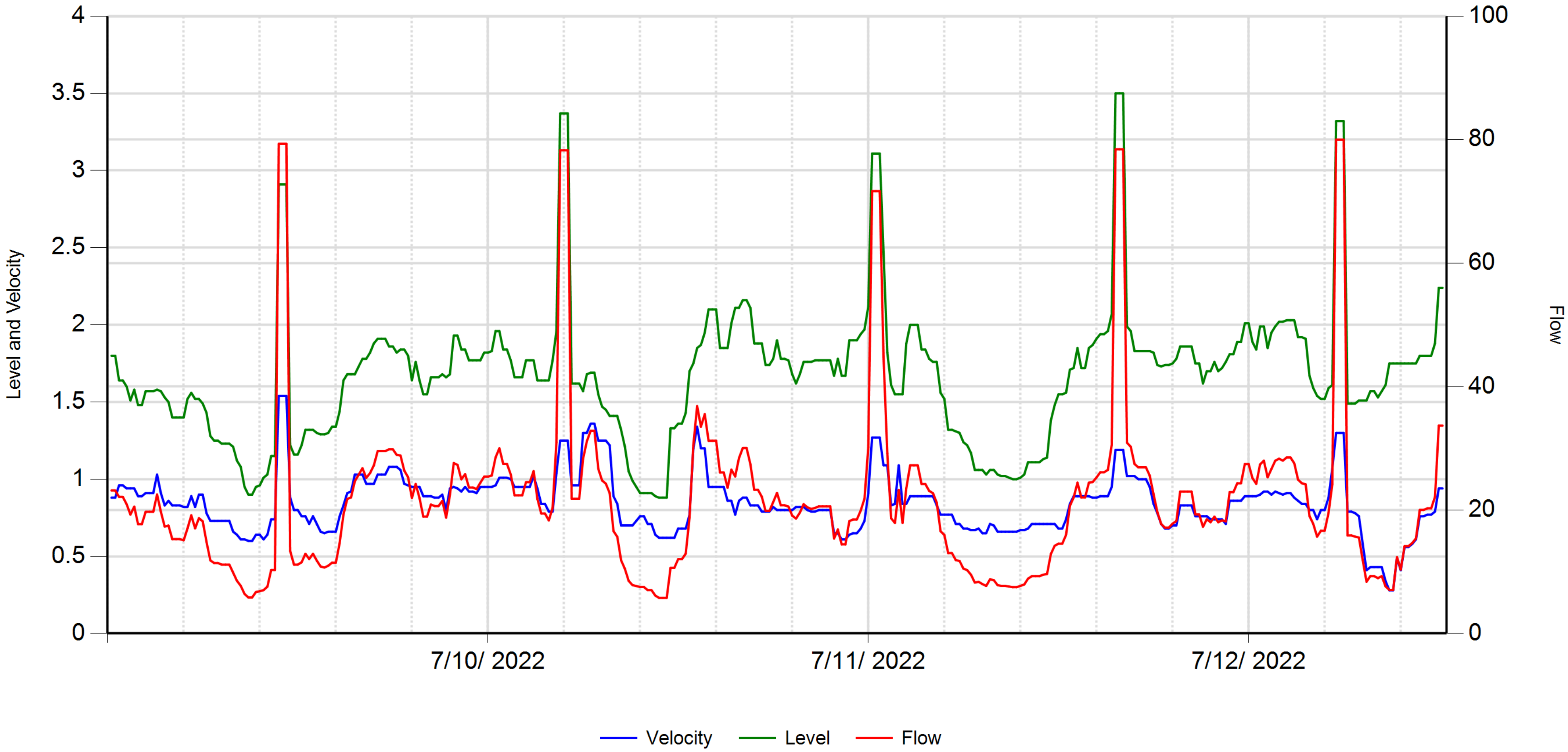
2022.06 Site 2 Roosevelt MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.848	1.653	21.361	**RainFall**	Inches
Maximum	1.760	4.020	93.010		
Minimum	0.340	0.750	5.080		

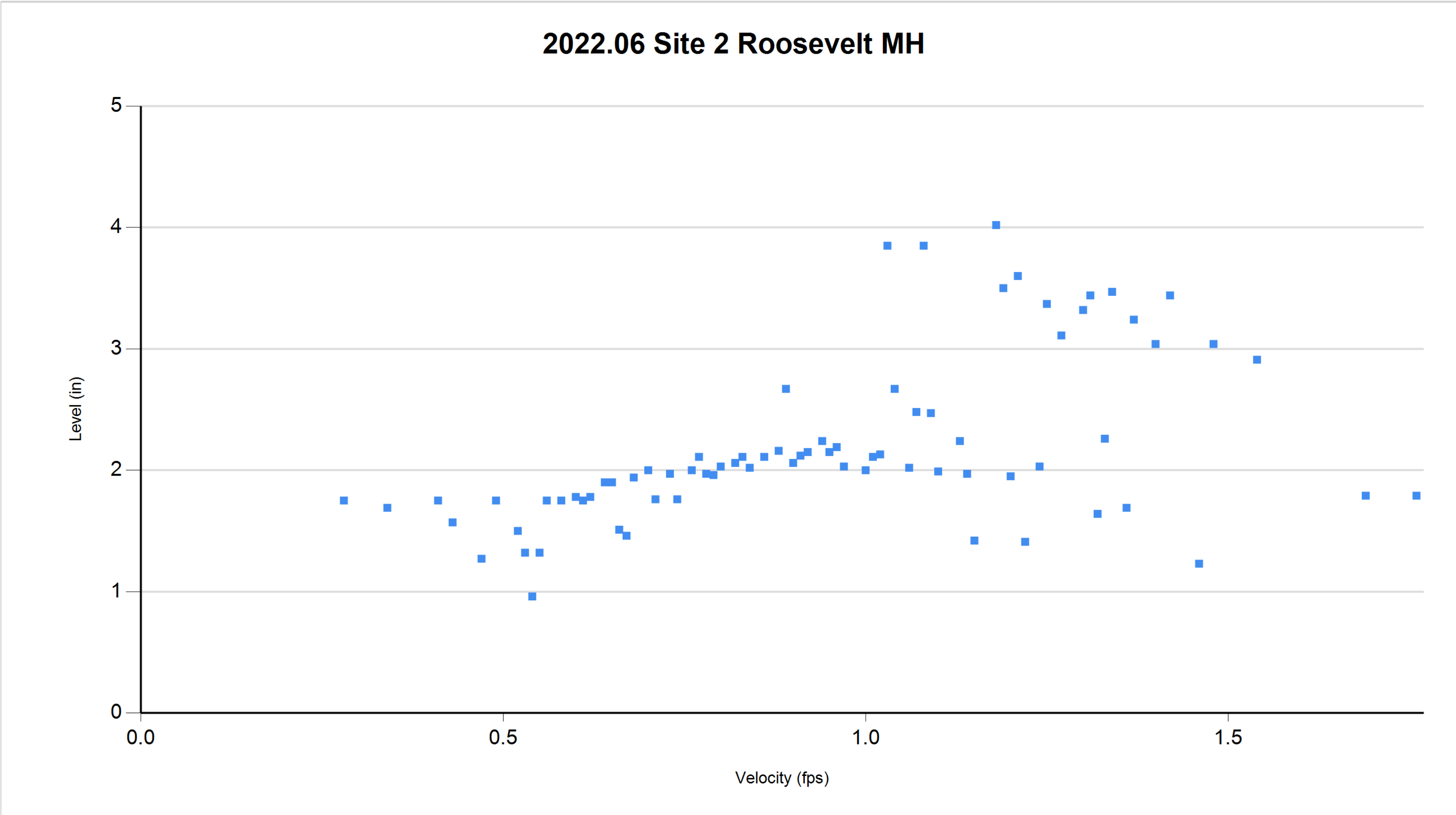
7/27/2022

2022.06 Site 2 Roosevelt MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.845	1.684	21.785	**RainFall**	Inches
Maximum	1.540	3.500	80.000		
Minimum	0.280	0.880	5.770		

7/27/2022



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06/28/2022 thru 07/13/2022

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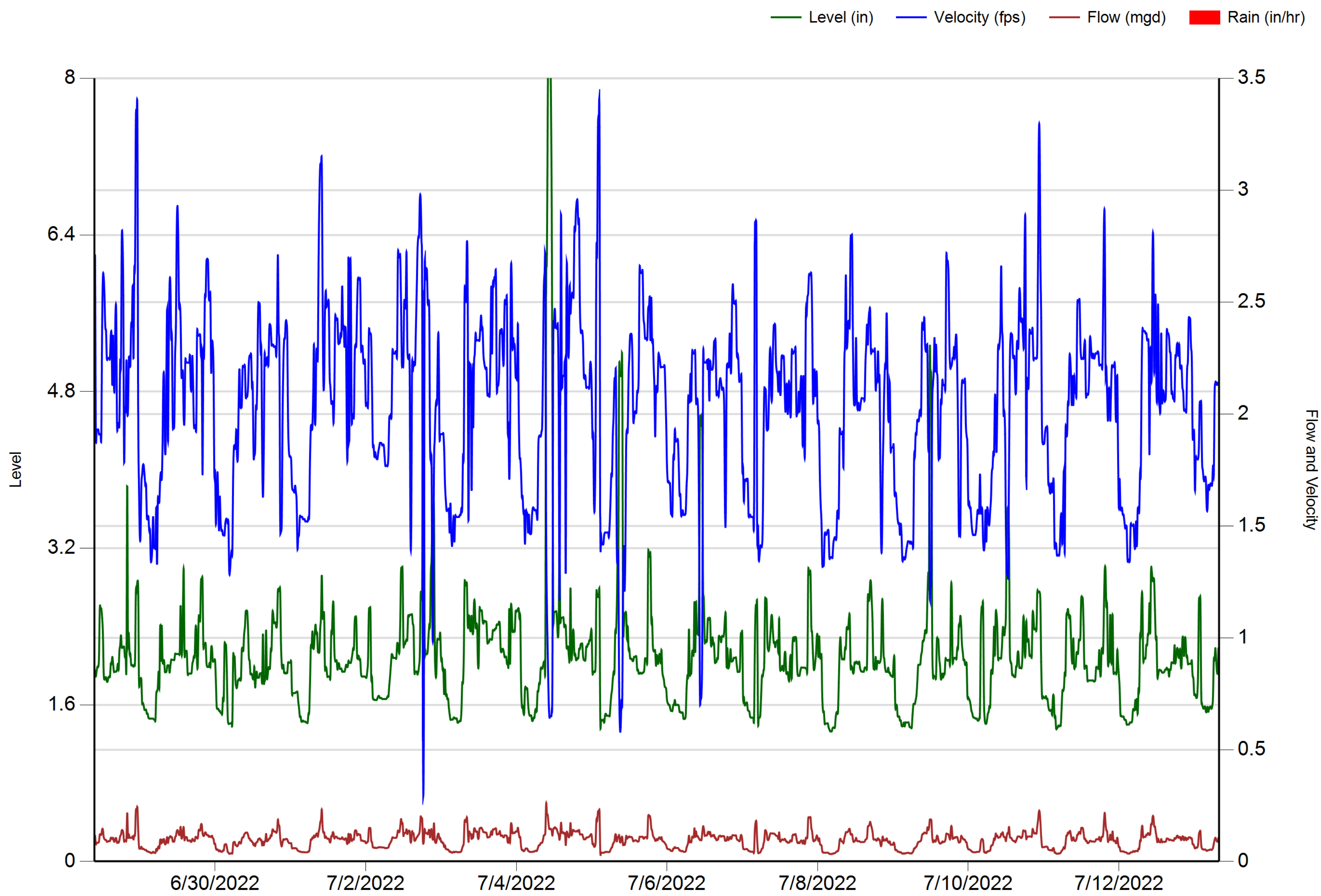
Group: CIBCSD Project
Site: 2022.06 Site 3 Cahuenga MH



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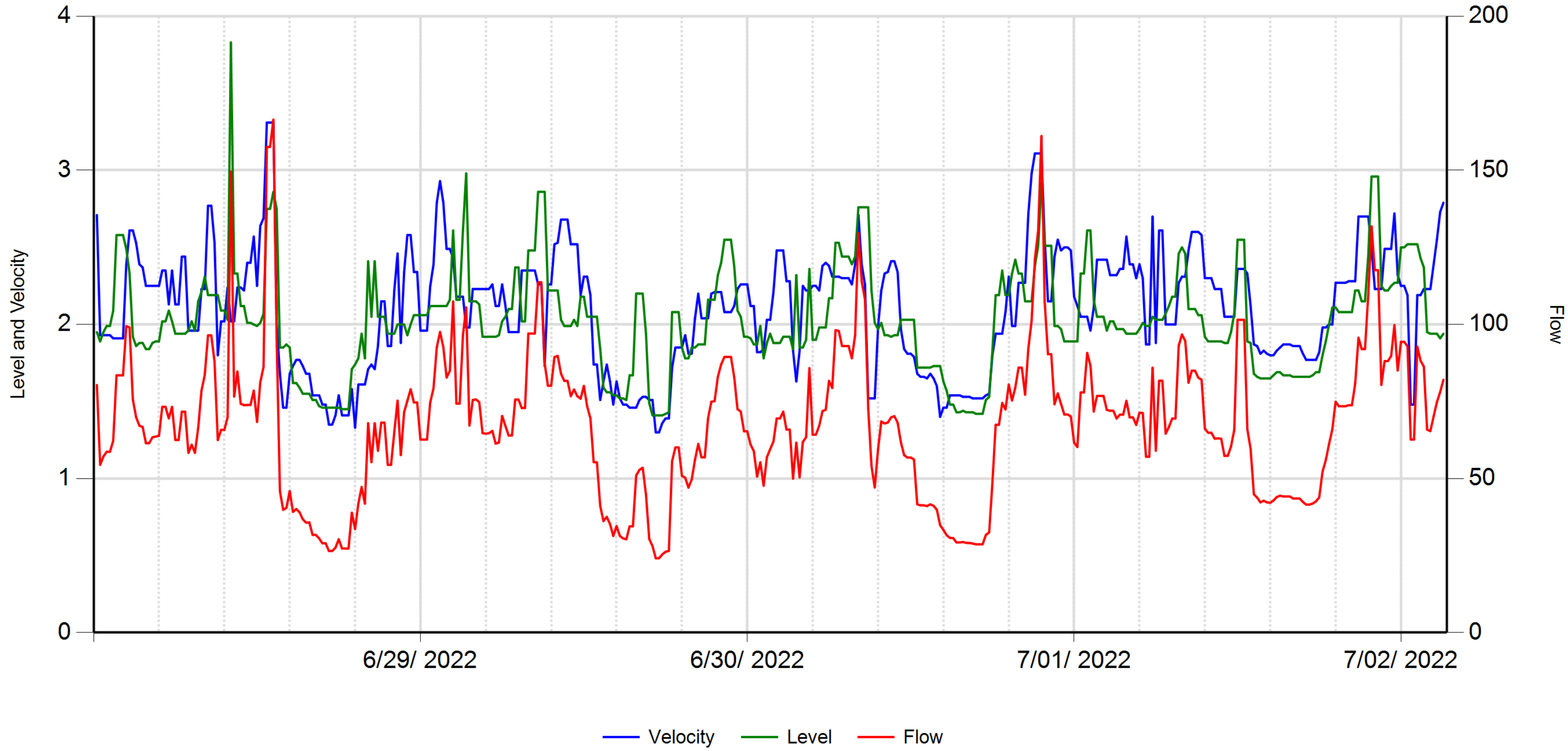
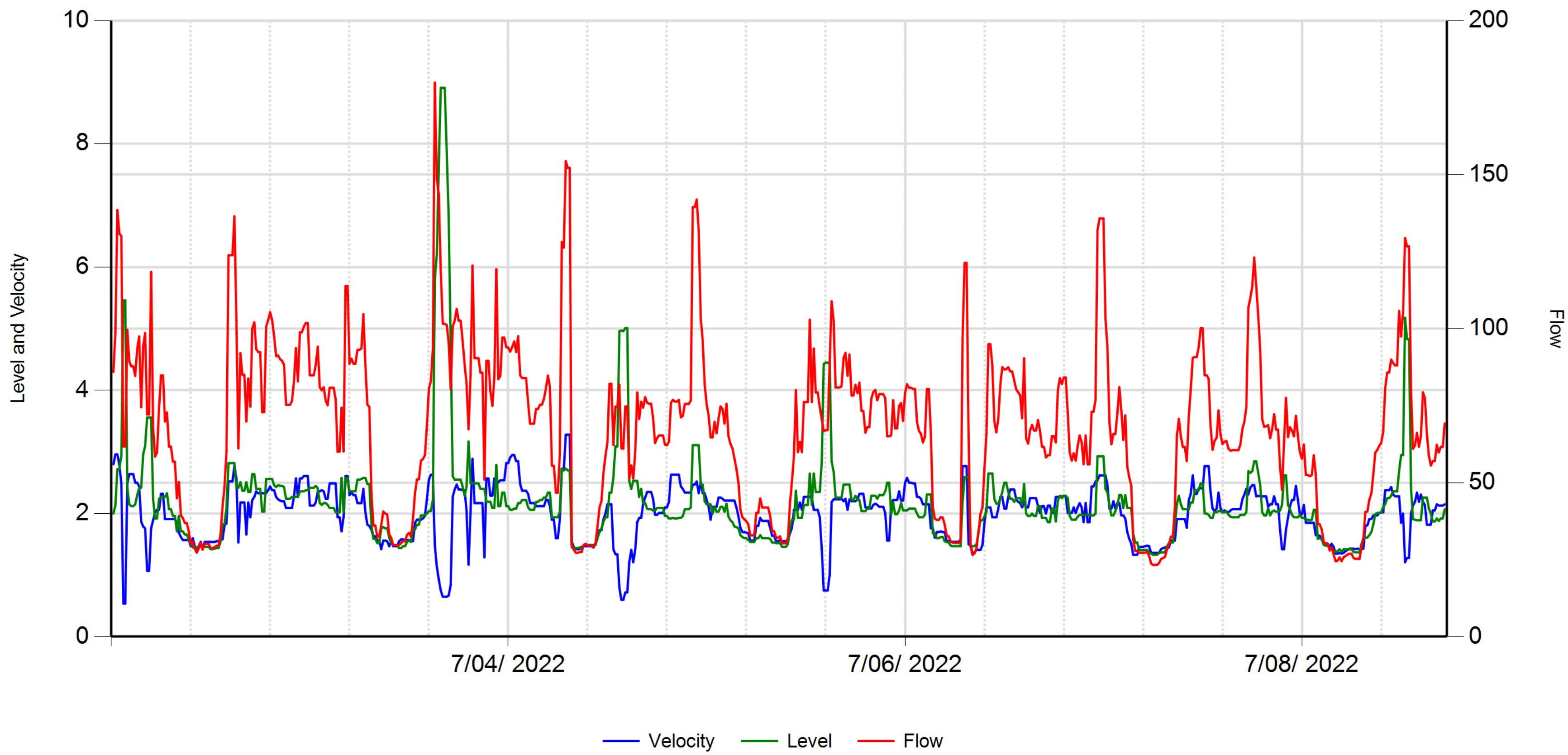


	Velocity (fps)	Level (in)	Flow (gpm)		
Average	2.099	2.027	67.325	**RainFall**	Inches
Maximum	3.310	3.830	166.430		
Minimum	1.300	1.410	24.150		

7/27/2022

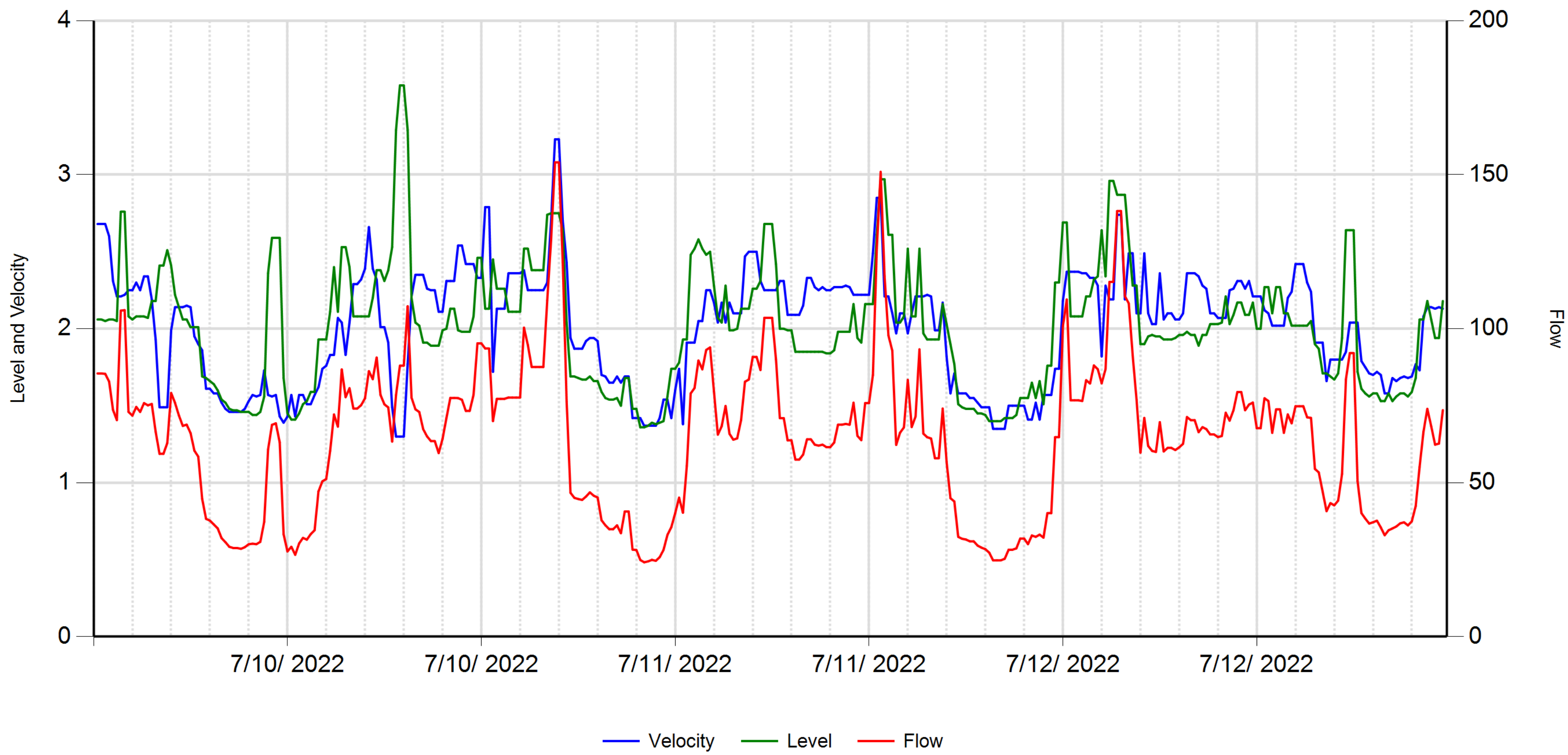
2022.06 Site 3 Cahuenga MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.992	2.190	68.352	**RainFall**	Inches
Maximum	3.280	8.910	179.880		
Minimum	0.540	1.330	23.310		

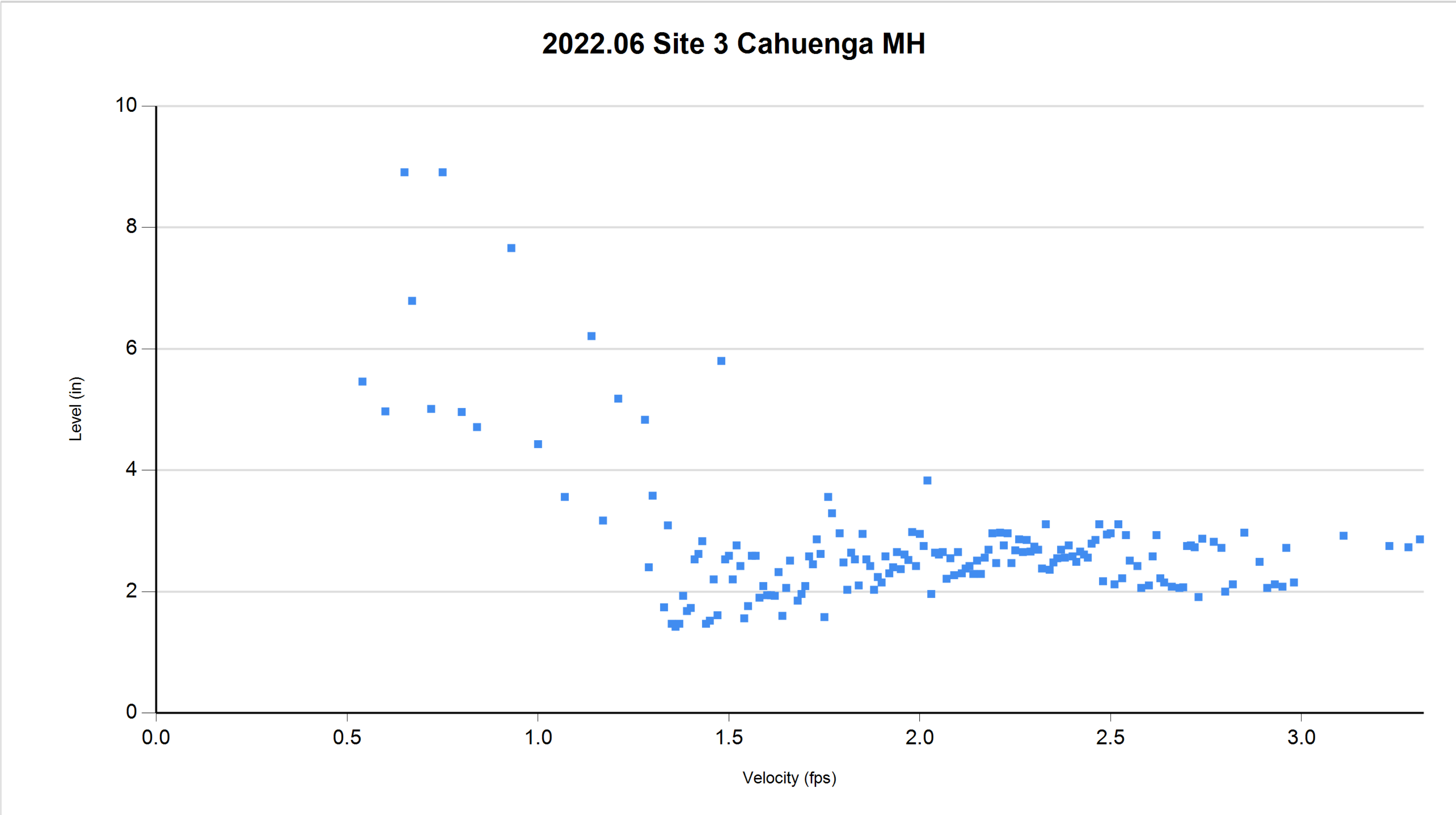
7/27/2022

2022.06 Site 3 Cahuenga MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	2.010	2.015	64.333	**RainFall**	Inches
Maximum	3.230	3.580	154.000		
Minimum	1.300	1.360	24.180		

7/27/2022



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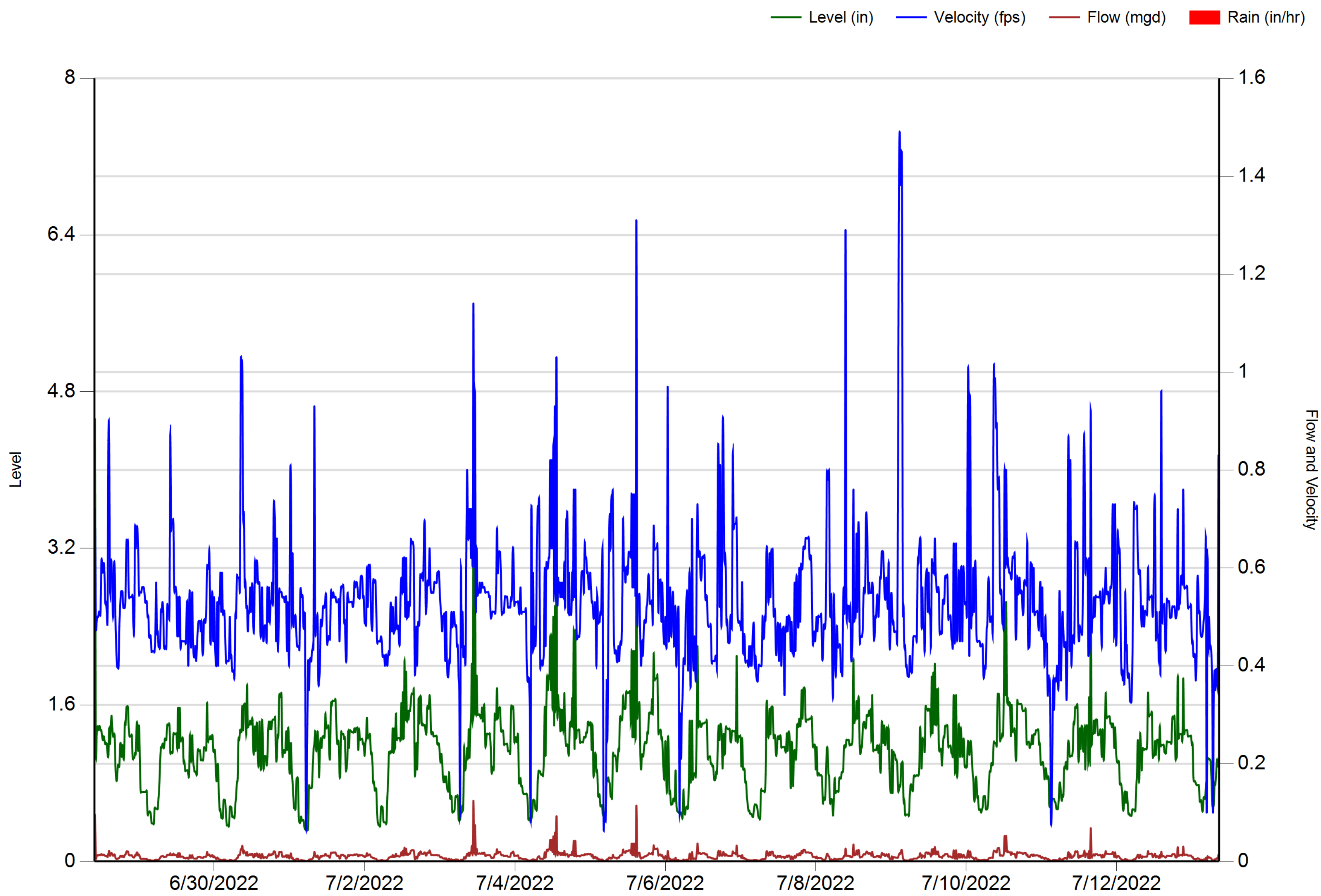
2022.06 Site 4 Highland MH:
06/28/2022 thru 07/13/2022

Report Date: 07/27/2022
Customer: PACE
Group: CIBCSD Project
Site: 2022.06 Site 4 Highland MH



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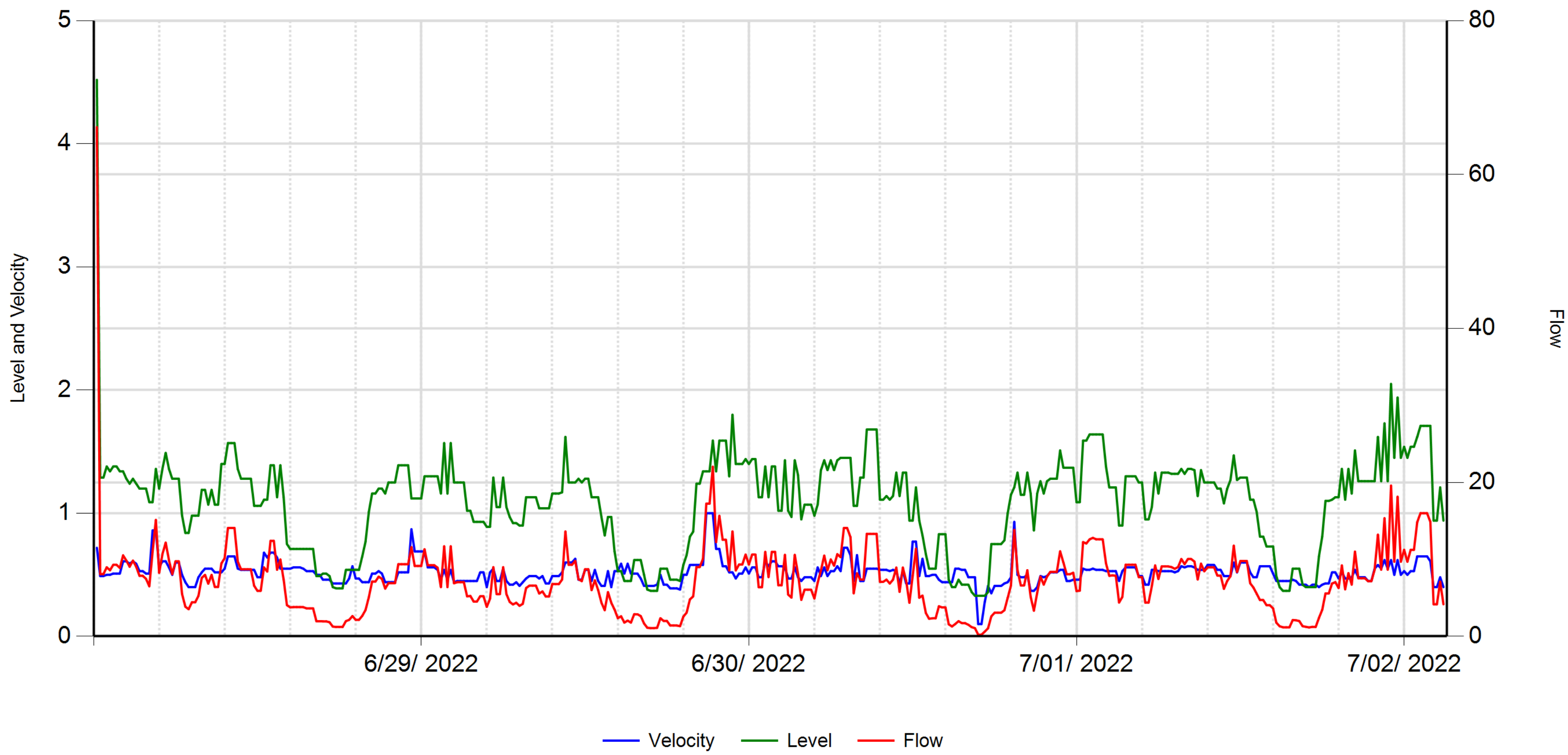
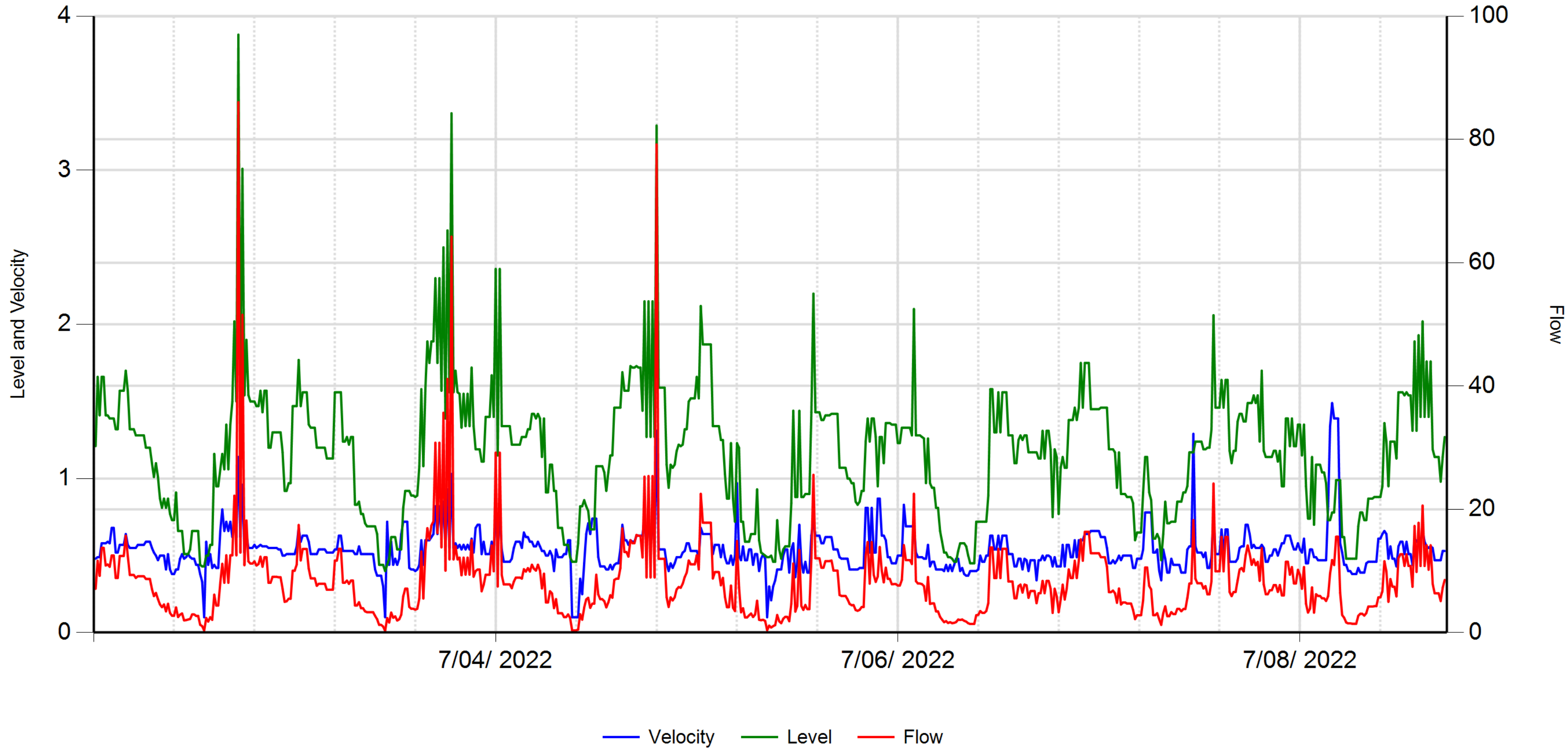


	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.518	1.098	7.255	**RainFall**	Inches
Maximum	1.000	4.520	66.180		
Minimum	0.100	0.330	0.220		

7/27/2022

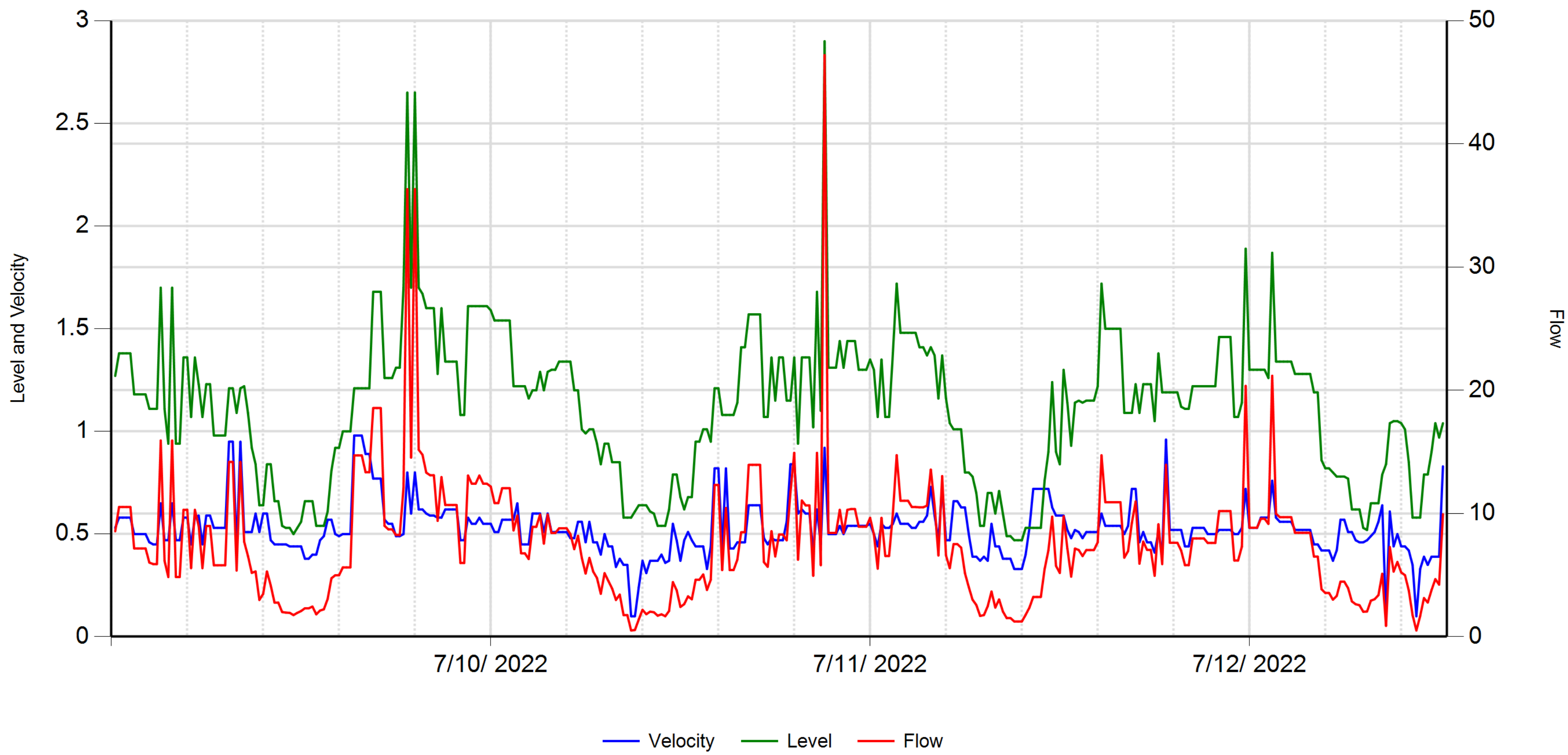
2022.06 Site 4 Highland MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.537	1.161	8.416	**RainFall**	Inches
Maximum	1.490	3.880	86.050		
Minimum	0.100	0.400	0.290		

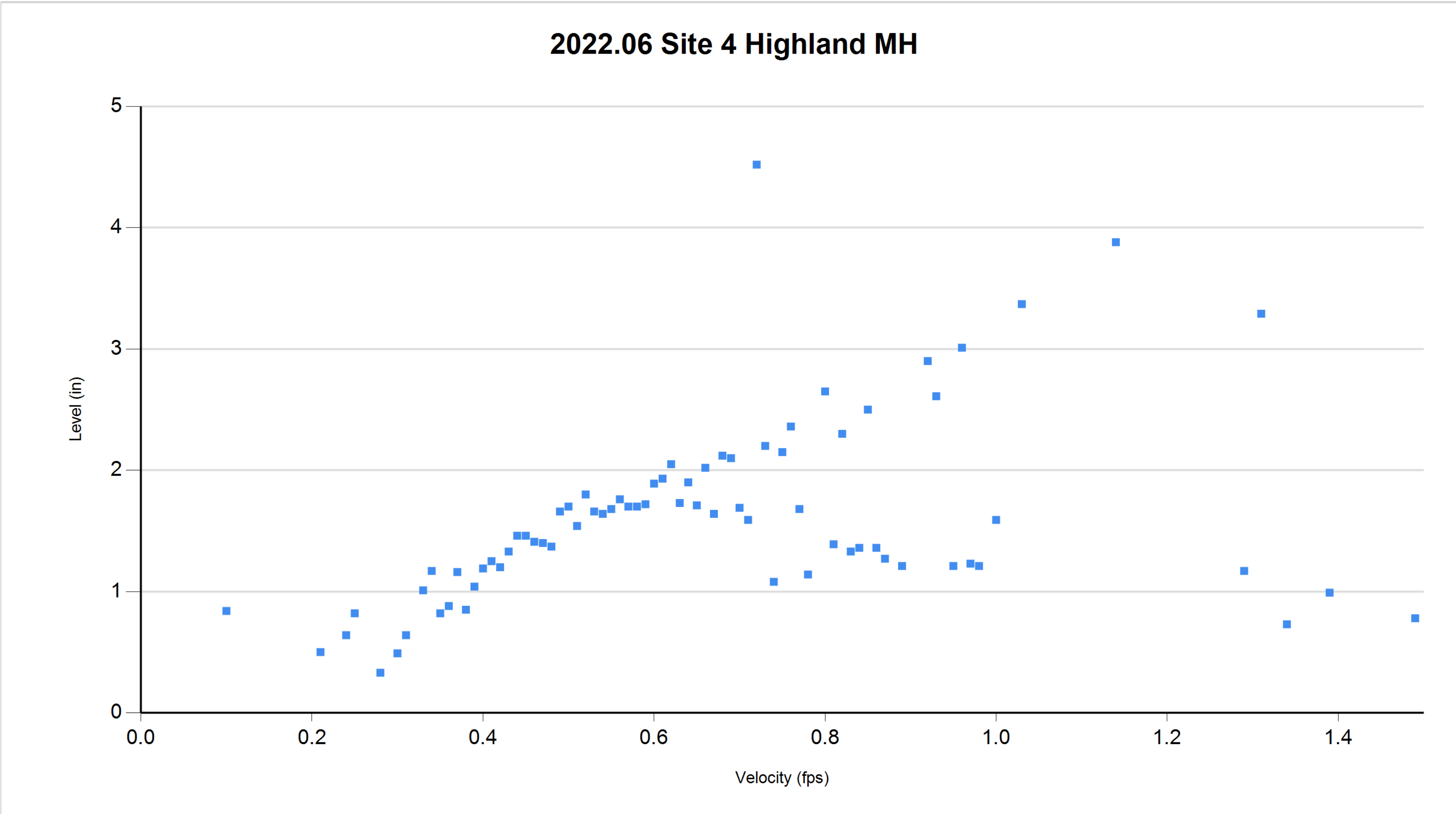
7/27/2022

2022.06 Site 4 Highland MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.527	1.118	7.638	**RainFall**	Inches
Maximum	0.980	2.900	47.210		
Minimum	0.100	0.470	0.510		

7/27/2022



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7/27/2022 1:47:23 PM

2022.06 Site 5 Malibu MH:
06/28/2022 thru 07/13/2022

Report Date: 07/27/2022
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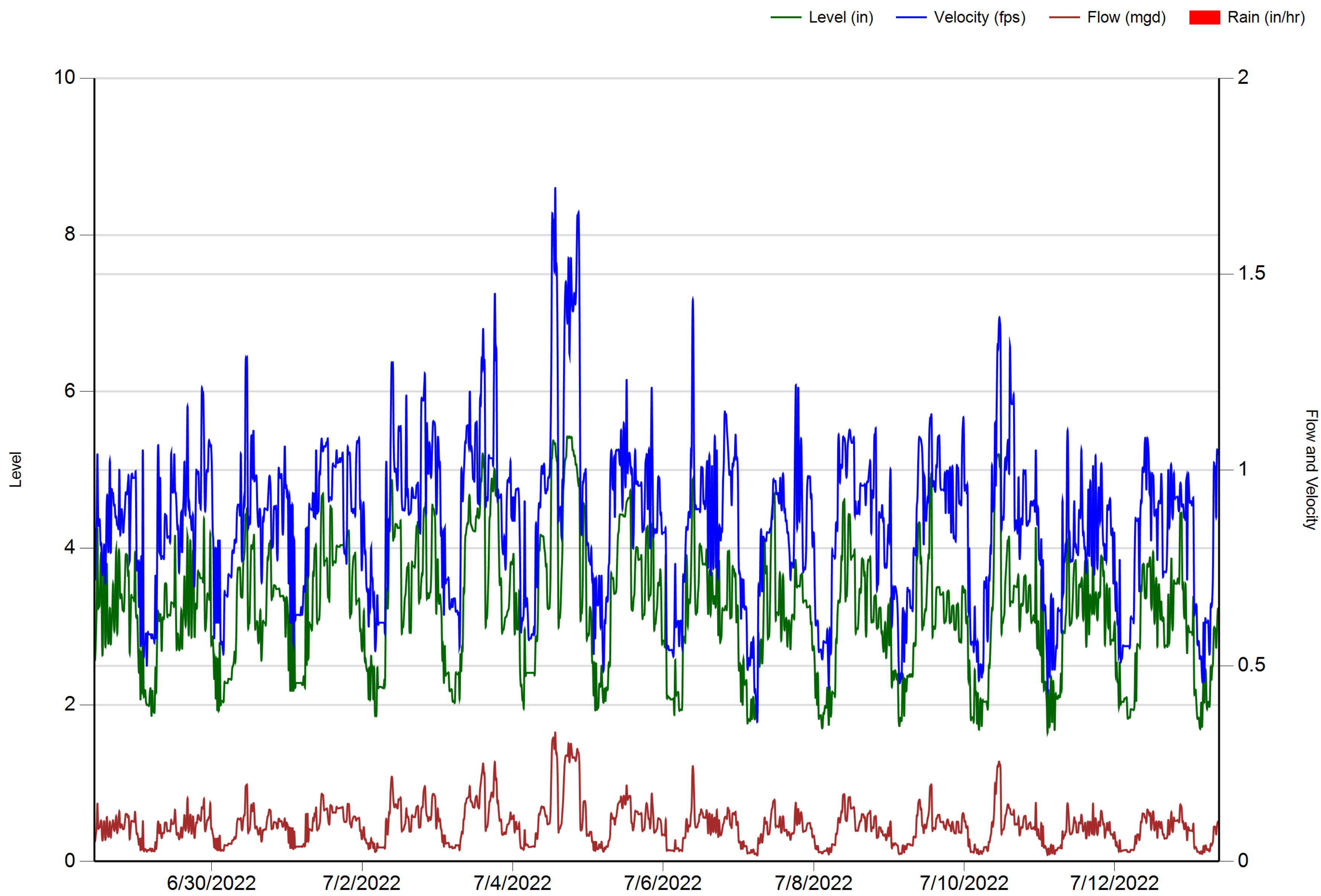
Group: CIBCSD Project
Site: 2022.06 Site 5 Malibu MH



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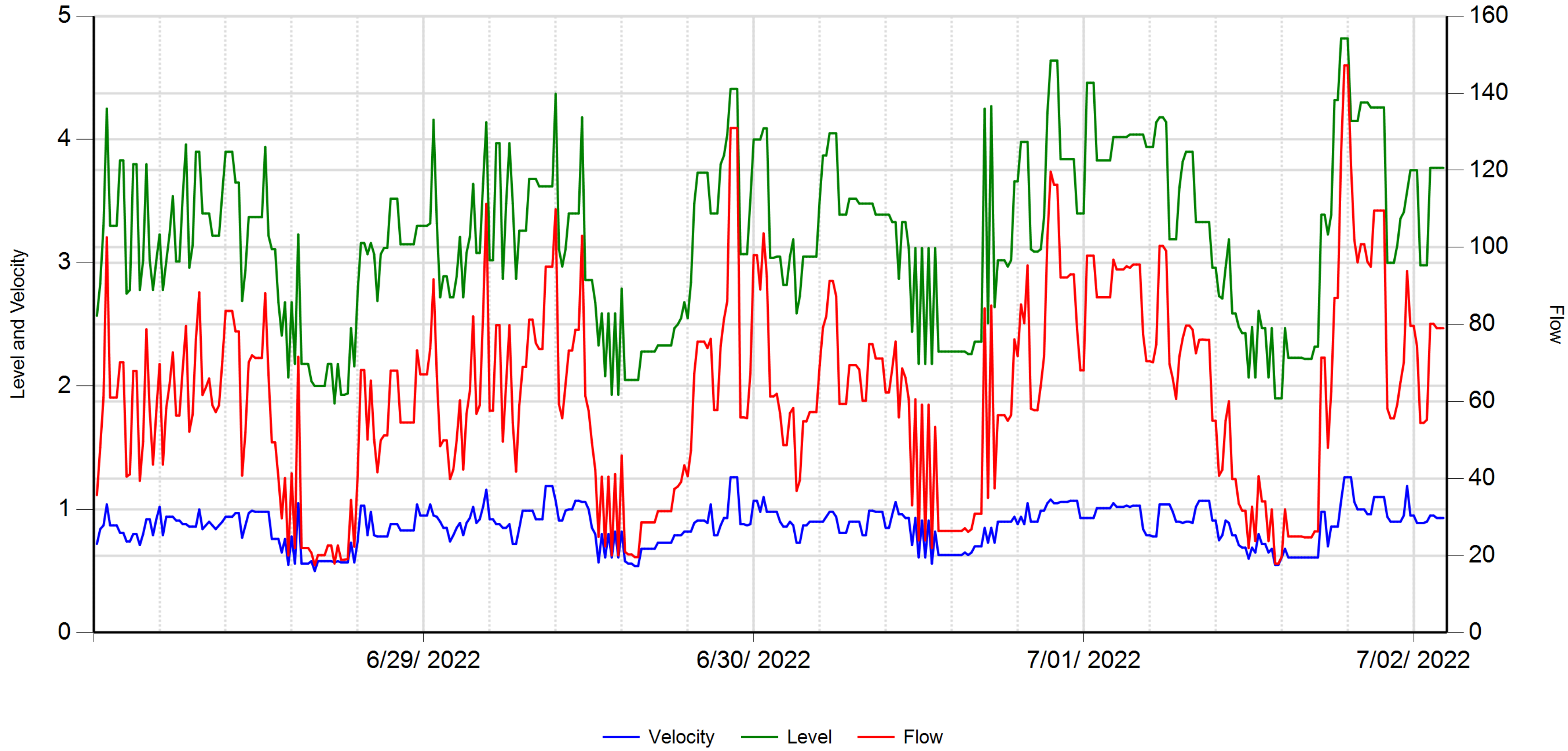
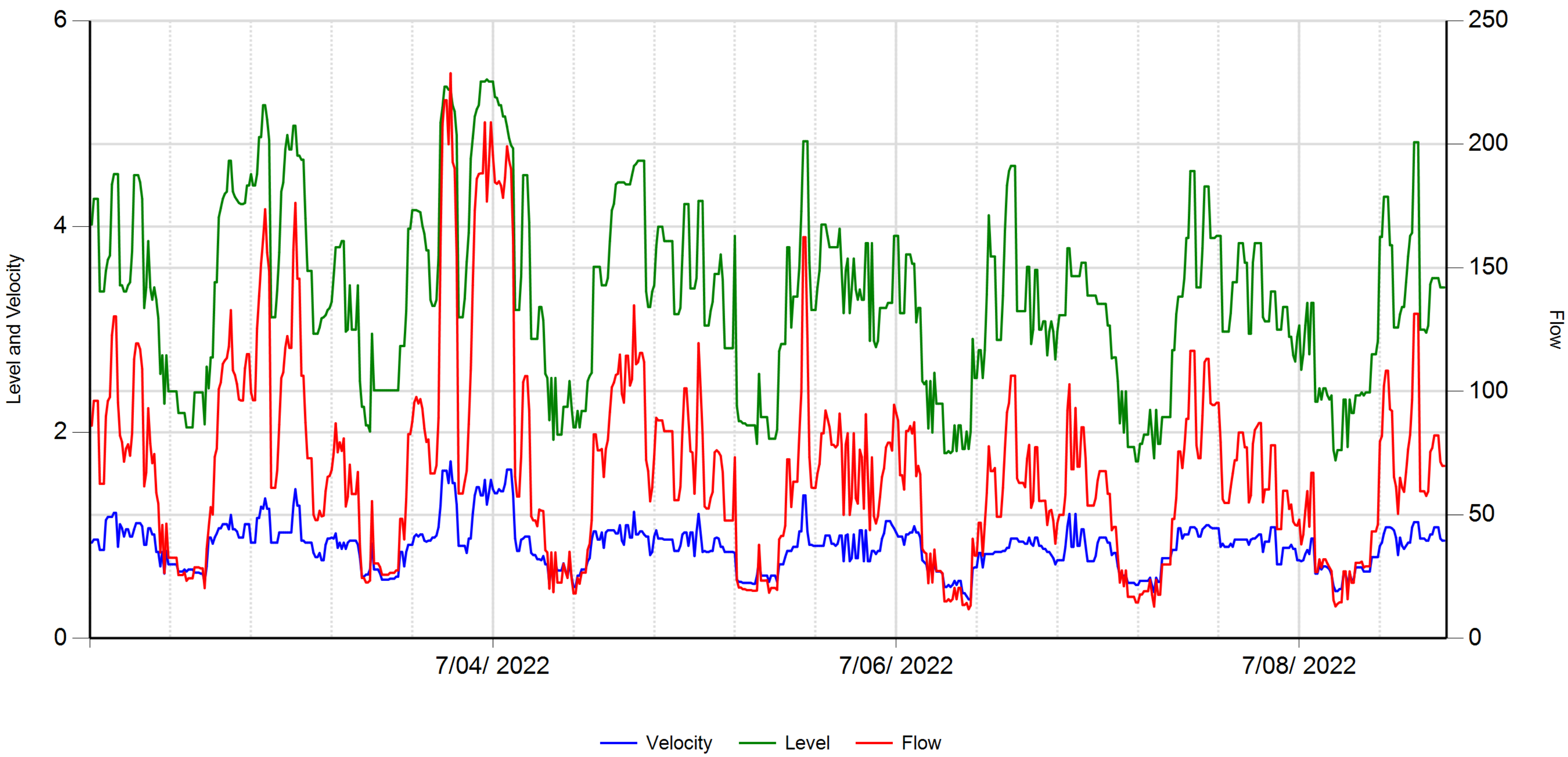


	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.865	3.199	61.332	**RainFall**	Inches
Maximum	1.260	4.820	147.235		
Minimum	0.500	1.860	17.519		

7/27/2022

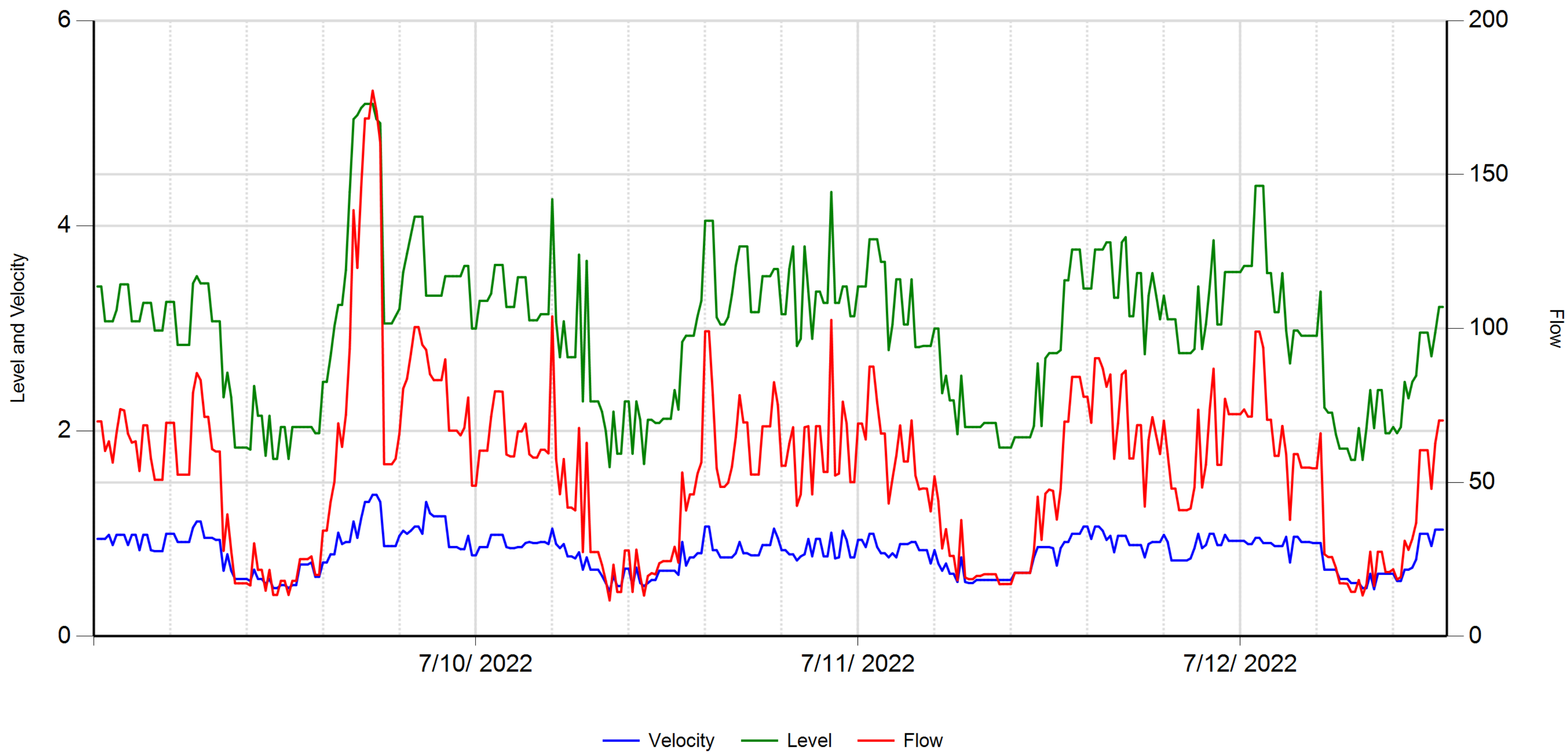
2022.06 Site 5 Malibu MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.894	3.313	69.123	**RainFall**	Inches
Maximum	1.720	5.430	228.777		
Minimum	0.380	1.720	11.841		

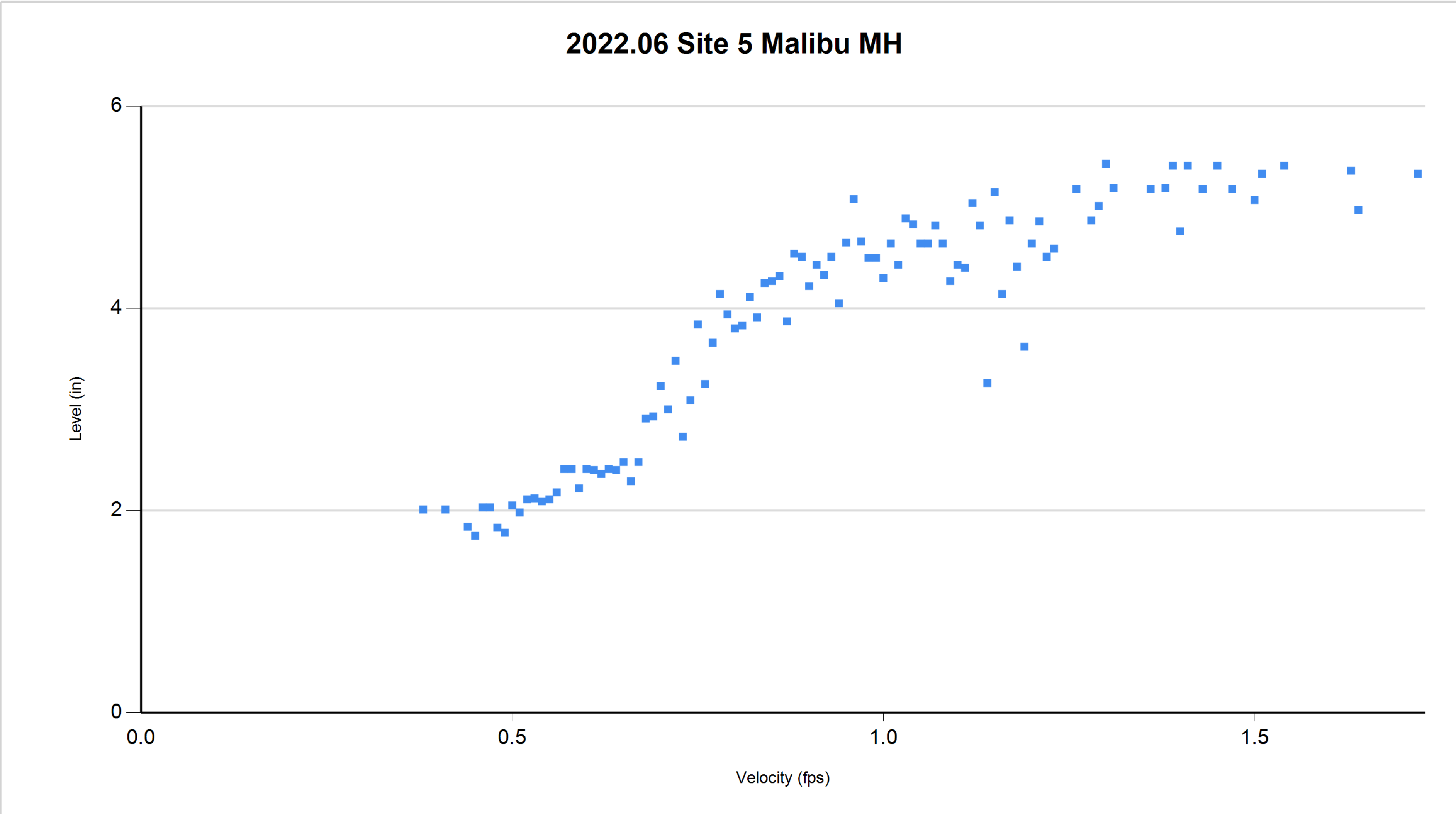
7/27/2022

2022.06 Site 5 Malibu MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	0.822	2.980	54.083	RainFall	Inches
Maximum	1.380	5.190	177.256		
Minimum	0.440	1.650	11.659		

7/27/2022



6/28/2022 thru 7/13/2022

7/27/2022 3:20:07 PM

Appendix C: Data and Documentation Technical Memorandum

REGISTERED PROFESSIONAL ENGINEER
DUNCAN S. LEE
No. C44825
Exp. 03-31-24
CIVIL
STATE OF CALIFORNIA

DRAFT - Technical Memorandum

Date: October 26, 2022

To: Ewelina Mutkowska, M.Sc, Senior Storm Water Manager
Ventura County Public Works Watershed Protection
800 S. Victoria Avenue, #1610
Ventura, CA 93009-1610

From: Duncan Lee, P.E., Principal – Utilities Division
Cherise Thompson, EIT, Project Engineer
Matthew Mills, EIT, Design Engineer

Re: Kiddie Beach Bacteria TMDL Reduction – Summary of Plans, Databases, and Documentation

PACE JN: B804

Introduction / Purpose

This technical memorandum has been prepared in order to summarize the data and documentation that PACE has gathered, in order to perform several analyses for the Ventura County Public Works Watershed Protection Division (County). The data and documentation compiled was used to generate alternatives for compliance with the bacteria TMDL set for the storm drain outfall at Kiddie Beach, as well as complete a sewer capacity study.

Many sources were utilized for the project, including (1) the 2014 Silver Strand Pump Station Deficiency Study (prepared by PACE), (2) San Nicolas Pump Station (SNPS) as-built plans, (3) SNPS diversion as-built plans, (4) SNPS diversion drainage area delineation, (5) SNPS diversion data, (6) storm drain / sanitary sewer shapefiles, (7) 2021 MS4 Permit, (8) stormwater quality data, (9) monitoring well data, (10) PACE Kiddie Beach Sewer Capacity Study, (11) County assessor / parcel information, (12) Oxnard Airport rain gauge data, and (13) Navy Drainage & Sewer Map Atlases – Visited Navy Site on 5/12/22. It should be noted that although the SNPS is located adjacent to the Santa Paula and Santa Monica Pump Stations, the plans, data, and documentation discussed in this memo will center primarily around the SNPS, as this is the location of the proposed improvements. Each of the sources will be discussed in detail in the following sections, as well as how they were utilized for the project.

Project Background – General

PACE is in the process of compiling viable options to present to the County for reducing or eliminating bacteria from dry- and wet-weather flows reaching Kiddie Beach. Kiddie Beach is part of the larger Silver Strand Community Watershed (SSCW), consisting of approximately 108 acres (~0.17 square miles) under the County's jurisdiction, in the City of Oxnard, California. The SNPS watershed is one of three that make up with SSCW, each draining to a separate pump station. In current conditions, dry-weather flow is pumped to the sewer system through a diversion at the SNPS. The existing sewer system is owned and operated by Channel Islands Beach Community Services District (CIBCSD). The sewage is directed to the City of Port Hueneme and the City of Oxnard's Sanitary Sewer Collection System, eventually reaching the Oxnard Wastewater Treatment Plant (WWTP). The pollutants associated with this diverted dry-weather flow are removed from the flow path to Kiddie Beach; however, the MS4 Permit calls for a greater reduction in bacteria pollutant loads than is provided by the existing dry-weather flow diversion. Therefore, the County needs to identify additional options for reducing or eliminating bacteria from dry- and wet-weather flows entering Kiddie Beach, in order to meet the bacteria TMDL of the MS4 Permit.

1 – Silver Strand Pump Station Deficiency Study (PACE, 2014)

The 2014 study performed by PACE evaluated the three pump stations located in the Silver Strand Beach Area: (1) San Nicolas, (2) Santa Paula, (3) and Santa Monica. The study concluded that all three were in good working condition, mechanically operating as per the original design and stated capacities in record documents.

Some recommendations were made as to improvements that could take place at the pump stations, including installation of a protective shroud over the face of existing stormdrain outfalls to reduce impacts from sanding; repairs to the above grade building structures at the Santa Paula and Santa Monica stations; repair/replacement of fixed manual trash screens at all three stations; installation of a drain/pumping system in the valve vault at the Santa Paula station to remove standing water; installation of a solid floor covering over the wet wells; adding variable frequency drives (VFDs) to the main pump motor starters; replacement of low flow sump pumps with specialized ceramic lined pumps; apply epoxy coating to the interior of the wet wells; installation of new control systems including programmable logic controls (PLCs); and installation of remote centralized supervisory control and data acquisition (SCADA) systems.

Hydrologic and hydraulic modeling was also performed to evaluate the effectiveness of the existing stormwater drainage infrastructure for Silver Strand. The models indicated that the existing pump and storm drain infrastructure are operating suitably for the 10-, 50-, and 100-yr storm events. The capacities for each pump station are summarized in **Table 1**. The size of watershed draining to each pump station is summarized in **Table 2**. The watershed delineations are shown in **Figure 1**.

Table 1: Silver Strand Pump Station Capacities

Pump Station	Single Pump Capacity (gpm)	Dual Pump Capacity (gpm)
San Nicolas	**7,600**	**15,000**
20hp Main Pumps (2):	*7,600 @ 7.5' tdh*	*15,000 @ 7.5' tdh*
5hp Sump Pump:	*450 @ 13' tdh*	
5hp Sewer Pump:	*120 @ 22' tdh*	
Santa Paula	**11,700**	**23,000**
50hp Main Pumps (2):	*11,700 @ 10.5' tdh*	*23,000 @ 10.5' tdh*
5hp Sump Pump:	*450 @ 12' tdh*	
Santa Monica	**8,300**	**16,000**
40hp Main Pumps (2):	*8,300 @ 10.5' tdh*	*16,000 @ 10.5' tdh*
5hp Sump Pump:	*450 @ 13' tdh*	

Table 2: Silver Strand Watershed Areas

Pump Station	Area (ac)	No. of Catch Basins
Santa Paula	36.4	15
Santa Monica	26.0	21
San Nicolas	31.1	19

2 – San Nicolas Pump Station As-Built Plans

Two sets of as-builts were obtained for the SNPS. One, titled *Silver Strand Storm Drain – Unit V,* was stamped with a project completed date of 5/26/87. This includes the storm drain and catch basin inlets leading to the SNPS, running along Roosevelt Blvd. from Highland Dr. to San Nicolas Ave., and includes the pump station itself. The maximum size storm drain shown on the plans for Roosevelt Blvd. is 36-inch diameter reinforced concrete pipe (RCP), while the minimum is 24-inch RCP. The footprint of the SNPS enclosure is 24-ft. long by 16-ft. wide, and included two main pumps (15hp) and a sump pump (3hp) at the time of construction. A review of the O&M Manual for this pump station revealed that two 20hp main pumps were installed at the San Nicolas Station, in place of the 15hp pumps called out on the plans.

The second set of plans, titled *Silver Strand Storm Drain – Unit VI,* was stamped with a project completed date of 3/8/89. This includes the storm drains and inlets leading to the Roosevelt Blvd. storm drain main line, from Santa Monica Dr., Melrose Dr., Rossmore Dr., Cahuenga Dr., and Highland Dr. The plan set also includes the storm drain laterals leading to the main lines of the Santa Paula and Santa Monica Pump Stations. These laterals are located on Simi Ave., Ocean Dr., Hueneme Ave., Glendale Ave., Ventura Ave., Pasadena Ave., Burbank Ave., San Fernando Ave., Hollywood Ave., Van Nuys Ave., and Sawtelle Ave. The maximum size storm drain lateral shown on the plans is 24-inch RCP, while the minimum is 12-inch RCP. The plans also show a new 15-inch diameter RCP storm drain line connected directly to the SNPS wet well.

Although not shown on an as-built plan, invoicing from Flo-Systems (#F12832 and #F12897, dated 11/29/11 and 12/21/11 respectively) shows that the 3hp sump pump was replaced by a 5hp pump.

A third set of as-built plans was identified for the retrofit including the diversion to the Channel Islands sewer collection system and several other pump station upgrades, which are discussed in **Section 3**.

3 – San Nicolas Diversion As-Built Plans

The as-built drawing containing the diversion retrofit for the SNPS is titled *Silver Strand Beach Diversion Facilities – San Nicolas Pump Station Upgrades in the City of Port Hueneme,* with a project completed date of 3/1/06. The plan set shows several improvements to the pump station, including the addition of a 5hp pump in the wet well to pump stormwater through a 3-inch line to the existing sewer manhole located near the intersection of San Nicolas Ave., Melrose Dr., and Roosevelt Blvd.

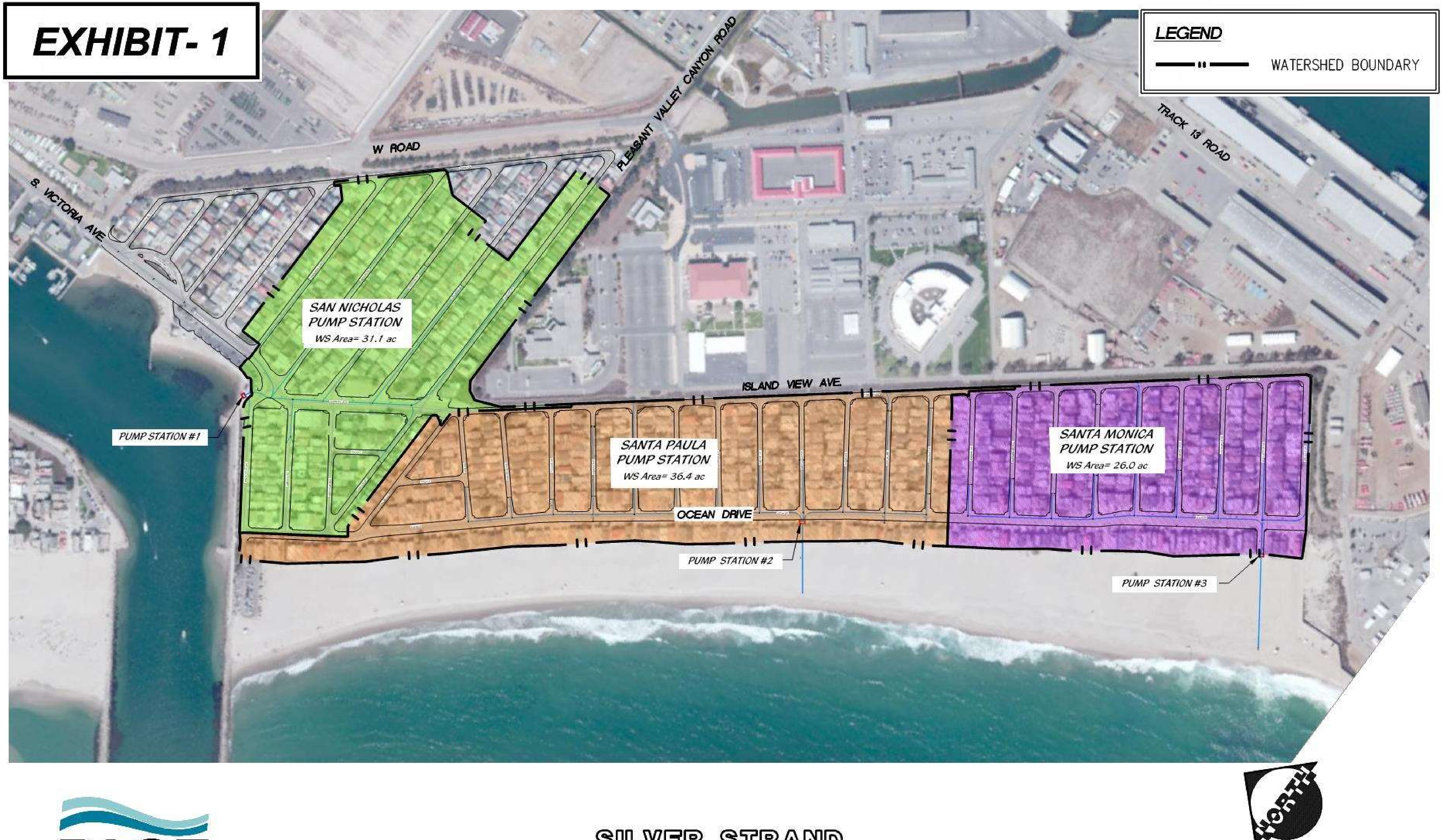
The average operating flow rate of the sewer pump (70 gpm) was used as an input to the 20-yr rainfall analysis performed by PACE, which determined the peak diverted flow rate and volume associated with various percentile storm events. Although the sewer pump is capable of a diversion flow rate up to 120 gpm, a flow rate of 70 gpm was used in the 20-yr rainfall analysis, which aligns with the average diversion rate observed in the pump station operational data obtained from the County (discussed in **Section 5**). The percentile storm results will be utilized to score and size various alternatives that will be presented to the County for compliance with the bacteria TMDL.

4 – San Nicolas Diversion Drainage Area Delineation

The hydrology analysis performed for the SNPS is described in PACE's *Silver Strand Pump Stations Deficiency Study.* This report identified the watershed boundaries shown in **Figure 1** and summarized in **Table 2**. The boundaries were delineated using a combination of 2005 topographic data and a local GPS survey performed by PACE. The overall slopes within the neighborhood are relatively flat, with elevations ranging from 3-ft. to 17-ft. above mean sea level (MSL), based on the 1988 North American Vertical Datum (NAVD88). The 2014 hydrology analysis was considered appropriate for use in the present analysis.

The San Nicolas watershed boundary was utilized in the 20-yr rainfall analysis performed by PACE, which determined peak runoff flow rates and volumes for various percentile storm events draining to the pump station. The percentile storm results will be utilized to score and size the various alternatives that will be presented to the County for compliance with the bacteria TMDL.

EXHIBIT- 1



SILVER STRAND
PUMP STATION - OVERALL WATERSHED MAP

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Xrefs: a201_drains.dwg; a201_pump station.dwg; a201_sd.dwg; a201_street cl.dwg; a201_street fl.dwg; a201_street names.dwg; a201_street tc.dwg; a201_cl-tc-fl (pnts 5-13-13).dwg; A201_Google A
Dimscale = 1; LTscale = 0.5; PSltscale = 1; Acad Ver. = 18.1s (LMS Tech); Visretain = 1

5 – San Nicolas Pump Station Diversion Data

The County provided recent SCADA data for the diversion at the SNPS, spanning from June 27th, 2022 to July 14th, 2022 (17 days). This data showed that, in total, the SNPS diverted 71,611 gallons to the Channel Islands sewer system, with an average pump cycle of 16 hours. Each time the pump turned on, it pumped approximately 3,000 gallons at an average rate of 70 gallons per minute.

The SCADA data was used to determine how much dry weather flow is diverted from SNPS to the sewer system, on average. The diversion rate was utilized in the 20-yr rainfall analysis performed by PACE, which is described in PACE's *Kiddie Beach Bacteria TMDL Reduction: Summary of Alternatives Ranking Matrix & 20-Year Rainfall Analysis* technical memorandum (October 2022). The total volume diverted to the sewer was calculated for each percentile storm event analyzed.

6 – Storm Drain / Sanitary Sewer Shapefiles

The Channel Islands Beach Community Services District (CIBCSD) owns and maintains the sanitary sewer collection system that the SNPS diverts stormwater into. The sewer atlas was obtained from the CIBCSD in order to identify the location and sizes of the sewer pipes within this network. The files showed numerous vitrified clay pipe (VCP) sizes, ranging from 4- to 10-inches in diameter, with a majority being 8-inches in diameter. A majority of the network is made up of gravity lines, although there are some force mains. The sewer data also included the locations of all manholes, cleanouts, and lift stations associated with the sewer lines.

The sewer information provided by the CIBCSD was utilized in the sewer capacity study, which was recently completed by PACE (see **Section 10**). The evaluation of the sewer system's capacity will also be used to determine the viability, cost, and sizing of some of the alternatives that will be presented to the County for compliance with bacteria TMDLs.

7 – 2021 MS4 Permit

The MS4 Permit applicable to discharges at Kiddie Beach is Order R4-2021-0105, adopted July 23, 2021, effective beginning September 11, 2021, and expiring on September 11, 2026. The permit includes discharge prohibitions and total maximum daily load provisions (TMDLs). The City of Oxnard is prohibited from non-stormwater discharges into receiving waters, and is prohibited from discharging trash to surface waters of the State. The TMDLs applicable to the project are listed in Attachment L of the MS4 permit, also summarized in **Table 3** below.

Table 3: Kiddie Beach TMDLs

Pollutant	Applicable TMDL Effluent Limitations (MPN or cfu)	
	Daily Maximum	Geometric Mean
Bacteria (Total Coliform)	10,000/100 mL	1,000/100 mL
Bacteria (Fecal Coliform)	400/100 mL	200/100 mL
Bacteria (Enterococcus)	104/100 mL	35/10 mL

The bacteria TMDLs will be utilized to determine the effectiveness of various alternatives that will be proposed to the County in order to meet the requirements of the MS4 Permit.

In addition, the Order states that permittees can achieve compliance with the stated WQBELs and receiving water limitations through retention of all stormwater runoff volume up to and including the 85th percentile, 24-hour event for the drainage area tributary to the applicable receiving water. As such, the County will focus on the 85th percentile storm event as the preferred design condition when analyzing alternative methods for achieving bacteria TMDL compliance in the Kiddie Beach watershed.

8 – Stormwater Quality Data

Monitoring data from the Ventura County Environmental Health Division was provided for Sampling Point No. 37000, located at Kiddie Beach, and Sampling Point No. 3800, located at San Nicolas. It should be noted that it was outside of PACE's scope of work to analyze this data, so only a summary is provided herein.

The sampling for Kiddie Beach took place on 12 different days between June 22nd, 2021 and August 31st, 2021, resulting in the measurements presented in **Table 4**. Rainfall did not occur on any of the days that sampling took place. Two days resulted in an exceedance of TMDLs, namely July 27th, where 137/100 mL of enterococcus was encountered and October 5th, where exceedances of total coliform, fecal coliform, and enterococcus were all encountered.

Table 4: Kiddie Beach Water Quality Sampling Results (Point 37000)

Date	Time	Total Coliform (MPN/100 mL)	Fecal Coliform (MPN/100 mL)	Enterococcus (MPN/100 mL)
06/22/21	925	86	31	20
06/29/21	931	120	31	75
07/06/21	1044	52	41	20
07/14/21	1033	52	10	10
07/27/21	928	857	364	137
07/28/21	1503	341	98	10
08/03/21	937	328	10	31
08/10/21	936	295	148	64
08/17/21	931	120	51	10
08/24/21	939	41	10	31
08/31/21	954	135	10	20
10/05/21	937	2,489	1,664	24,196

Sampling at San Nicolas took place on 26 different days between March 1st, 2021 and August 31st, 2021, resulting in the measurements presented in **Table 5**. Rainfall did not occur on any of the days that sampling took place. No exceedances of TMDLs were detected for any of the samples taken.

Table 5: S.S. San Nicolas Water Quality Sampling Results (Point 38000)

Date	Time	Total Coliform (MPN/100 mL)	Fecal Coliform (MPN/100 mL)	Enterococcus (MPN/100 mL)
3/1/21	1045	10	10	10
3/8/21	1044	10	10	10
3/15/21	1045	10	10	10
3/22/21	1059	10	10	10
3/29/21	1050	10	10	10
4/6/21	937	10	10	10
4/13/21	938	10	10	10
4/20/21	940	10	10	10
4/27/21	945	10	10	10
5/4/21	943	10	10	10
5/11/21	947	10	10	10
5/18/21	951	10	10	10
5/25/21	No sample – lab supply shortages			
6/1/21	1113	10	10	10
6/8/21	936	10	10	10
6/15/21	942	10	10	10
6/22/21	933	31	10	10
6/29/21	942	10	10	10
7/6/21	1051	10	10	10
7/14/21	1040	10	10	10
7/27/21	937	40	10	10
8/3/21	951	10	10	10
8/10/21	941	10	10	42
8/17/21	941	10	10	10
8/24/21	949	10	10	10

8/31/21	959	10	10	10
10/05/21	946	132	31	20

9 – Monitoring Well Data

In August 2021, Padre Associates, Inc. completed a technical report for the County of Ventura Public Works Agency titled *Installation of Six Temporary Groundwater Monitoring Wells – Groundwater Quality Assessment Project, Hobie Beach and Kiddie Beach Park Area Oxnard, Ventura County, California.* The report details the construction of six temporary groundwater monitoring wells in the Silverstrand Beach area. The drill holes reached depths of approximately 25-ft. below ground surface (bgs). Static groundwater was measured to be on average approximately 5-ft. bgs. The groundwater monitoring well locations are shown on **Figure 2**. It should be noted that the monitoring well data was not within PACE's scope of work to analyze; therefore, only a summary of the data is contained herein.

Figure 2: Groundwater Monitoring Wells Locations

10 – PACE Kiddie Beach Sewer Capacity Study

PACE separately completed a report titled *Kiddie Beach Sewer Capacity Study,* which analyzes the capacity of the existing CIBCSD system, as well as how much of that capacity is currently utilized and how much room it has for additional demand. The analysis was performed in order to determine how much, if any, additional flow could be diverted from the SNPS to the CIBCSD sewer system. The report has been finalized, conclusions have been made, which are summarized herein.

Utilizing the SCADA data described in **Section 5**, it was determined that the SNPS diverts, on average, 0.101 MGD, at an average rate of 70 gallons per minute each pump cycle (It is PACE's understanding that the CIBCSD directed County's staff to reduce diversion pump rate to 70 gallons per minute). Peak flow rates were also observed from this data, which showed up to 180 GPM being diverted from the SNPS to the CIBCSD sewer system. The size of the existing sewer system was estimated using representative locations within the system. Site visits were performed at two existing sewer manholes (MH030 and MH056) downstream of the SNPS, which revealed size, configuration, and flow conditions. Using the maximum recorded diversion rate from the SNPS diversion SCADA data in conjunction with the recorded pipe sizes, it was determined that MH030 would experience inundation / exceedance of design capacity during the peak outflow rate. Additionally, a CIBCSD representative was able to recall past events where MH056 became temporarily inundated.

Figure 3: Locations of MH030 and MH056 from Sewer Atlas

One parameter commonly referenced for determining available pipe capacity is the depth-to-diameter ratio (d/D). While the stringency on the restrictions of this ratio varies per governing faction, it is commonly utilized as a method for analyzing available pipe capacity. Per typical industry practice, the design peak flow rate in smaller diameter collection pipes is limited by a depth-to-diameter ratio of 0.5. MH030 and MH056 are both connected to 8-inch diameter pipes. Calculation of the (d/D) ratio for MH056 revealed that these pipes are at the recommended threshold of 0.50 for the condition where SNPS diverts at the average recorded pumping rate of 70 GPM. Any addition of flows from the SNPS to the CIBCSD sewer system would result in an exceedance of recommended design parameters set forth by the CIBCSD; therefore, additional dry weather diversion from SNPS is not recommended.

Diversion of more dry weather flow to the CIBCSD could potentially be achieved, through upsizing of all downstream pipelines and lift stations; however, the existing sewer system includes 6 miles of pipe and two lift stations, so an upgrade of these facilities would come at a significant cost / effort.

11 – County Assessor / Parcel Information

Assessor maps and other parcel data were gathered in order to identify open spaces that might be utilized for one or more of the alternatives that will be presented in the bacteria TMDL compliance plan. In particular, Lots 6 and 13, identified in the assessor's map for 'Portion Patterson Ranch Subdivision, M.R. Bk.8, Pg.1 were reviewed and found to be owned by the Navy. This area is currently used as a golf course by the Navy. Both a parcel report and a special districts report that were generated for this area revealed no other information with regards to ownership, land use, zoning, etc.

12 – Oxnard Airport Rain Gauge Data

Rainfall precipitation data was obtained from the Oxnard Airport Rain Gauge, for use in the percentile storm analysis performed by PACE. The percentile storm analysis was utilized for sizing the various alternatives that will be presented to the County for compliance with the bacteria TMDL. The rainfall data spans 21 years, from October 1st, 1999 to October 1st, 2020. Analyzing the rainfall data resulted in the different percentile precipitation depths shown in **Table 6**.

Table 6: Percentile Precipitation Depths

Percentile Rain Event	Precipitation Depth (in)
20th	0.01
40th	0.09
50th	0.15
70th	0.45
85th	0.96
95th	1.84

13 – Navy Site Visit / Drainage & Sewer Map Atlas

The Navy site, located adjacent to Kiddie Beach, was visited by the project team on May 12th, 2022. During this visit, the condition of the Navy golf course was observed. The golf course represents the most viable location to place a temporary storage basin for the alternate bacteria compliance options developed by PACE and the City. The golf course contains 18 holes, with the 'front nine' being situated east of Patterson Road and the 'back nine' being situated west of Patterson Road. The 'back nine' is currently not used for golfing by the Navy, but serves as a training area for military exercises, with graded berms formed on its southern end. The field visit showed that the 'back nine' would be an ideal location for a detention basin and would require minimal re-grading to achieve that function.

Additionally, the Navy drainage and sewer map atlas was provided to the project team, which revealed several locations where existing sewer lines pass near the 'back nine'. These existing sewer lines would present an ideal location for water from the 'back nine' detention area to slowly be released back into the sewer system for treatment.

The Navy also expressed a willingness to collaborate on a plan to detain water in the 'back nine' in order to help them satisfy their own BMP requirements.

Figure 4: Golf Course on Navy Base

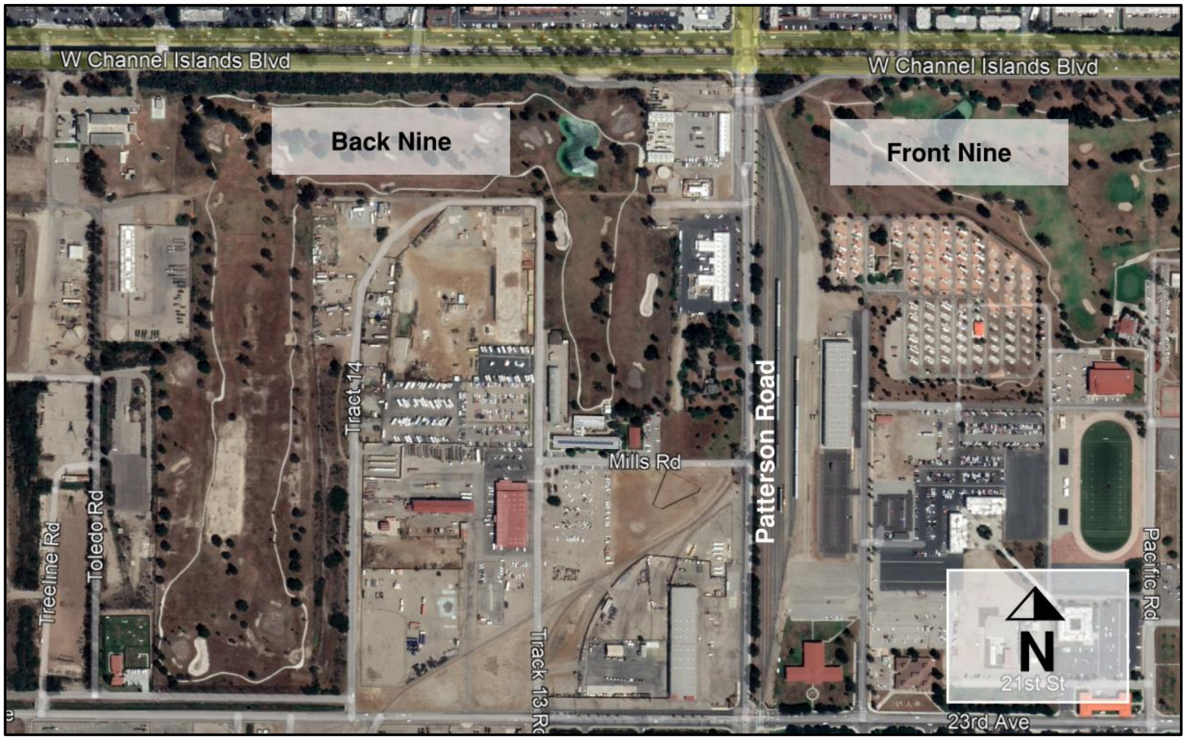
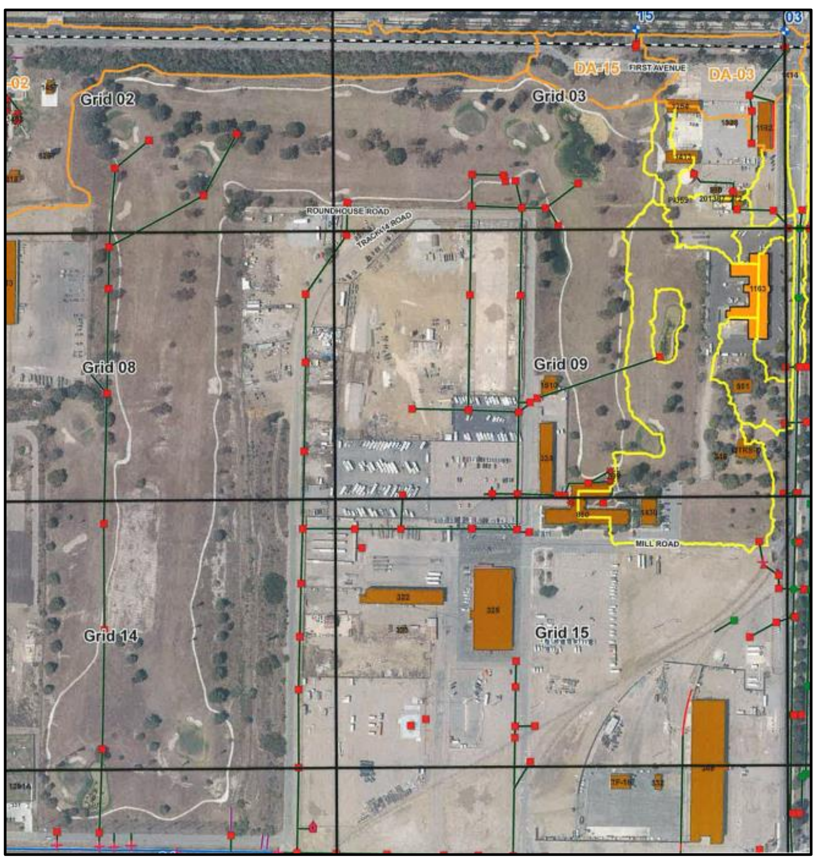


Figure 5: Navy Sewer Atlas



Appendix D: Ranking and 20-Year Hydrology Technical Memorandum

REGISTERED PROFESSIONAL ENGINEER
DUNCAN S. LEE
No. C44825
Exp. 03-31-24
CIVIL
STATE OF CALIFORNIA

DRAFT - Technical Memorandum

Date: October 5, 2022

To: Ewelina Mutkowska, M.Sc, Senior Storm Water Manager
Ventura County Public Works Watershed Protection
800 S. Victoria Avenue, #1610
Ventura, CA 93009-1610

From: Duncan Lee, P.E., Principal – Utilities Division
Cherise Thompson, EIT, Project Engineer
Matthew Mills, EIT, Design Engineer

Re: Kiddie Beach Bacteria TMDL Reduction: Summary of Alternatives Ranking Matrix & 20-Year Rainfall Analysis

PACE JN: B804

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1 - Introduction / Purpose

This technical memorandum has been prepared to document the process and results of an alternative ranking analysis performed by PACE for the Ventura County Public Works Watershed Protection Division (County). Various alternatives were evaluated for their viability in reducing or eliminating bacteria in dry- and wet-weather flow reaching Kiddie Beach. This Technical Memorandum (TM) will summarize the project's background, evaluated alternatives, and the criteria used to rank the alternatives.

This TM also presents the process and data sources utilized to perform a hydrology analysis of the watershed draining to the San Nicolas Pump Station (SNPS), located at Kiddie Beach. The 20-year hydrology analysis was instrumental in determining design flow rates/volumes for each of the alternatives and can be used to estimate the percent reduction of pollutant load by the corresponding decrease in flow to Kiddie Beach. PACE was not tasked with calculating the amount of pollutant load reduction as such water quality data is not provided by the County. Furthermore, such data typically will vary greatly, from which it would be challenging to draw any firm quantitative conclusions.

2 - Project Background – General

The alternative analysis completed by PACE was performed to present viable options to the County for reducing or eliminating bacteria from dry- and wet-weather flows reaching Kiddie Beach. The determination of the most viable and feasible alternative will be in the final feasibility study. Kiddie Beach is part of the larger Silver Strand Community Watershed (SSCW), consisting of approximately 108 acres (~0.17 square miles), under the County's jurisdiction, in the City of Oxnard, California. The SNPS watershed is one of three watersheds that make up SSCW, each draining to a separate pump station. In current conditions, dry-weather flow is pumped to the sewer system through a diversion at the SNPS. The existing sewer system is owned and operated by the Channel Islands Beach Community Services District (CIBCSD). The sewage is directed to the City of Port Hueneme and the City of Oxnard's Sanitary Sewer Collection System, eventually reaching the Oxnard Wastewater Treatment Plant (WWTP). The pollutants associated with this diverted dry-weather flow are removed from the flow path to Kiddie Beach; however, the MS4 Permit calls for a greater reduction in bacteria pollutant loads than is provided by the existing dry-weather flow diversion. Therefore, the County needs to identify additional options for reducing or eliminating bacteria from dry- and wet-weather flows entering Kiddie Beach to meet the bacteria TMDL of the MS4 Permit.

3 – Hydrology Analysis / Determination of Design Inflow

To determine the size of the system needed for each alternative, a hydrology analysis was performed, utilizing precipitation gauge data from the Oxnard Airport Rain Gauge, spanning from October 1, 1999, to October 1, 2020 (20 years). To convert the precipitation value into a runoff, the rational method was used (**Equation 1**), in accordance with the Ventura County Watershed Protection District *Hydrology Manual* (2017). The Rational Method is useful in estimating runoff for relatively small areas, 20 to 80 acres, with generally uniform cover type and grade. This equation was applied to every ordinate of data from the Oxnard Airport Gauge.

Equation 1: $$Q = CiA$$

where,

$Q = \text{Peak rate of runoff (cfs)}$
$C = \text{Runoff coefficient}$
$i = \text{Average intensity of rainfall (in/hr)}$
$A = \text{Watershed area (ac)}$

The average rainfall intensity was obtained from the gauge data, which is given in increments of inches for hourly intervals. As such, the precipitation value for each ordinate is equal to the intensity.

The watershed area draining to the SNPS was obtained from the *Silver Strand Watershed – Design Hydrology Update - Revised Final Report,* developed by the Hydrology Section of the Watershed Resources and Technology Division of the Ventura County Watershed Protection District in December of 2013. Figure 2 from that report has been included below as **Figure 1**, which shows the watershed boundaries for the San Nicolas, Santa Paula, and Santa Monica pump stations. **Table 1** breaks down the area associated with each watershed.

Table 1: Silver Strand Watershed Areas

Pump Station	Area (ac)
Santa Paula	36.4
Santa Monica	26.0
San Nicolas	31.1

The runoff coefficient represents the ratio of runoff to rainfall and can be described as the percentage of rainfall on a watershed that occurs as runoff, ranging from zero to 0.95. It includes the composite effect of watershed variables such as infiltration, ground slope, ground cover, surface and depression storage, antecedent precipitation and soil moisture, and shape of the drainage basin. The equation given in the *Ventura County Technical Guidance Manual for Stormwater Quality Control Measures* (Manual update 2011, Errata update 2018) was used to calculate C for the present analysis (**Equation 2**). The runoff coefficient calculated for the San Nicolas watershed is presented in **Table 3**.

Equation 2: $$C = 0.95 \times imp + C_p \times (1 - imp)$$

where,

$C = \text{Runoff coefficient (0 to 0.95)}$
$Imp = \text{impervious fraction of watershed}$
C_p = *pervious runoff coefficient, based on soil type, using* ***Table 2*** *below*

Table 2: Ventura Soil Type Pervious Runoff Coefficients

Ventura Soil Type (Soil Number)	Cp Value
1	0.15
2	0.10
3	0.10
4	0.05
5	0.05
6	0
7	0

Figure 1: VCWPD Watershed Delineation

Table 3: San Nicolas Runoff Coefficient Calculation

Pump Station	Land Use	Soil No.	Area (ac)	Impervious (%)	C_p	C	C (Wtd. Avg)
San Nicolas	Commercial and Services	7	1.356	90	0	0.86	0.72
San Nicolas	High Density Residential	3	0.160	65	0.1	0.65	
San Nicolas	High Density Residential	7	2.048	65	0	0.62	
San Nicolas	Industrial	3	0.136	72	0.1	0.71	
San Nicolas	Industrial	7	0.137	72	0	0.68	
San Nicolas	Low Density Residential	3	1.095	65	0.1	0.65	
San Nicolas	Low Density Residential	7	16.004	65	0	0.62	
San Nicolas	Open Space and Recreation	7	0.587	0	0	0.00	
San Nicolas	Road	3	0.460	100	0.1	0.95	
San Nicolas	Road	7	9.136	100	0	0.95	

Once a runoff flow rate value was obtained for each ordinate in the Oxnard Airport Gauge data, the results were further processed to determine rainfall depths, peak flow rates, and runoff volumes associated with various percentile storm events. For example, an 85th percentile storm event is one that would encompass 85% of storm events in a given year. An 85th percentile precipitation depth of 1.0 inches would indicate that, on average, 85% of storms in a year would produce 1.0 inches of rain or less. Through the use of the rational method, a runoff peak flow rate and volume can also be associated with this percentile storm event. The following assumptions were made in determining the various percentile events:

- <u>Dry Flow Diversion Stops at 0.1 inches:</u> The dry flow diversion at the SNPS, with its existing configuration, shuts off after 0.1 inches of rain. This is a requirement of the CIBCSD so that the capacity of the sewer system is not compromised by the rain event.
- <u>Minimum Time Between Rain Events = 24 hrs:</u> One rain event is defined as a period of precipitation preceded by 24 hrs of zero precipitation followed by 24 hrs of zero precipitation.
- <u>Existing Dry Diversion Pump Capacity = 70 gpm:</u> Until 0.1 inches of precipitation is achieved, the dry diversion pump at the San Nicolas Pump station continues operating, pumping up to 70 gpm from Kiddie Beach to the CIBCSD sewer collection system. This diverted volume is eventually treated at the WWTP, removing associated pollutants from out letting to the ocean. The remaining volume would then be targeted by one of the nine alternatives evaluated in **Section 4**.

Table 4 shows the calculated precipitation depths, runoff peak flow rates, runoff volumes, rainfall event durations, and diverted volumes associated with various percentile storm events. It should be noted that the 100th percentile was excluded for this analysis because an anomaly was observed in the precipitation data on November 8th, 2002 at 8:00am. The rain gauge data showed 2.92 inches of precipitation during this hour, which is approximately 330% larger than the next highest recorded hourly precipitation and approximately 4,300% higher than the average recorded hourly precipitation (excluding zero value data points). Each of the alternatives will be evaluated for cost, treatment effectiveness, and other metrics for multiple percentile storm events to make the best recommendation to the County.

Table 4: Percentile Storm Analysis Results

Percentile Event	Precipitation Depth (in)	Peak Runoff Flow Rate (**Before** Diversion) (cfs)	Runoff Volume (**Before** Diversion) (MG)	Runoff Volume (**After** Diversion) (MG)
20th	0.01	0.2	0.006	0.004
40th	0.09	1.3	0.053	0.043
50th	0.15	2.2	0.090	0.082
70th	0.45	4.4	0.272	0.269
85th	0.96	7.2	0.579	0.569
95th	1.84	12.7	1.110	1.109

4 – Alternatives Evaluated

In total, nine (9) alternatives were analyzed for their ability to reduce or eliminate the bacteria from wet- and dry-weather flows reaching Kiddie Beach. These alternatives all assume the installation of a hydrodynamic separator to protect the downstream processes and increase treatment efficiency. These alternatives are listed below, along with a brief description of what each entails:

- Store and Diversion to CIBCSD Sewer: Runoff reaching the SNPS would be diverted to an underground storage tank/wet well, where it could be held until demand on the Channel Islands sewer collection system is reduced. At that time, the stored water would be diverted to the CIBCSD in a manner that would minimize the impact to the existing sewer system.
- Pump, Store, and Divert to Sewer (Navy Golf Course): Runoff reaching the SNPS would be diverted/pumped to an abandoned golf course owned by the Navy, where it could be held until demand on the sewer collection system is reduced. The golf course would be repurposed into an above-ground, earthen detention basin. From there, the captured runoff would be released back into the sewer collection system at a slow, controlled rate, which would minimize the impact to the existing sewer system. This sewer connection is different from the existing connection with the CIBCSD.
- Diversion to Sewer (CIBCSD MH020): Evaluates an up-sized version of the SNPS that is capable of diverting flows to the CIBCSD at the flow rates needed to meet the bacteria TMDL limits of the MS4 Permit. Sewer infrastructure located downstream of San Nicolas Station would also require upgrades to accommodate the increased load on the system.
- Diversion to Sewer (MH on Patterson): Evaluates the construction of an additional pump station, which would be capable of diverting flows to the CIBCSD at the flow rates needed to meet the bacteria TMDL limits of the MS4 Permit. The pump station would bypass the smaller sewer lift stations and gravity pipes to pump directly into a trunk line located on Patterson.
- Store and Treat for Off-Site Reuse: Runoff reaching the SNPS would be diverted to a storage facility, where the water would then be treated to acceptable Title 22 standards. After treatment, the water would be diverted to an open space where it could be utilized for irrigation or other alternate use.
- Treat for Off-site Reuse: Runoff reaching the SNPS would immediately be treated to acceptable "purple pipe" or Title 22 drinking water standards. After treatment, the water would be diverted to an open space where it could be utilized for irrigation or other alternate use. This

option would have a higher rate of treatment than the “Store and Treat” option, since no storage facility is involved.

- Treat for Release: Runoff reaching the SNPS would immediately be treated. After treatment, the water would be released through the existing pump station. This option would have a higher rate of treatment than the “Store and Treat” option, since no storage facility is involved.
- Diversion to Santa Paula Pump Station: Runoff reaching the SNPS would be diverted to the Santa Paula Pump Station, instead of the CIBCSD. The diverted runoff would outlet to the Santa Paula ocean outfall, which has some excess capacity at lower percentile storm events and where the receiving waters are far more turbulent than at the existing San Nicolas ocean outfall. Pollutants associated with the diverted runoff would become mixed/diluted at the Santa Paula outfall; whereas, pollutants reaching the more stagnant waters at the SNPS remain more concentrated.
- San Nicolas Pump Station Ocean Outfall: The existing SNPS would remain unchanged, with exception to the location of the ocean outfall. The existing ocean outfall outlets to a particularly stagnant area of water, resulting in a concentrated area of pollutants. If the outfall were relocated to an area where the receiving water is more turbulent, the discharged pollutants would be better able to disperse and be diluted.

<u>5 – Feasibility Criterion Ranking</u>

In order to present the most viable option for bacteria removal for the County, PACE created a list of weighted criterion with which to judge each of the alternatives. Each alternative was rated on a weighted scale, with a higher score indicating greater feasibility. The alternative with the highest score will be recommended to the County for selection. Eight criterion with assigned weight and definitions are listed below:

- Capital Cost (4): The total cost to construct the proposed alternative. When two or more alternatives have a preliminary cost within 5% of the lowest, the alternatives are scored the same. The evaluation of capital costs will encompass purchasing of all equipment, land, material, and labor associated with the installation of an alternative.
- 50-Year Life Cycle Cost (2): The evaluation of the combination capital cost, operation, maintenance over the expected useful life of an alternative in today’s value. Automation, equipment hours of operation, operation man-hour requirements, required scheduled maintenance, maintenance requirements, and reliability will be factored into the life cycle cost.
- Cost / Acre-Foot / Year (5): The cost per acre-foot per year for bacteria TMDL reduction. This criterion will be utilized to analyze the proposed alternatives capability to efficiently meet bacteria TMDL reduction at various flow rates. Cost efficient removal is critical to the feasibility of an alternative.
- Performance (5): Effectiveness of the alternative to reduce bacteria TMDL as defined in the MS4 Permit. The alternatives will be analyzed at various flow rates to determine if they are capable of meeting the design requirements specified. Meeting higher flow rates will benefit an alternative’s performance.

- Public Agency / Regulatory Board Support (5): The support of public agencies and regulatory boards for an alternative. Due to the location of project, the California Coastal Commission may have a large impact in the feasibility of an alternative.
- Regulatory Requirements (1): Low need for an EIR, CEQA Analysis, and involvement of the California Coastal Commission will be scored favorably.
- Public Perception / Impact (2): Support of an alternative. The public perception of a project allows for residents who are impacted to feel that needs are addressed. Alternatives that minimize the impact to the public will be scored favorably.
- Constructability (3): Feasibility of being able to construct the proposed alternative. The complete system must be able to be constructed at the scale required to meet the bacteria TMDL removal requirements. The more complex a project is to implement, the lower the alternative will be scored. This includes impacts to existing infrastructure.

Appendix E: 2014 Silver Strand Pump Station Deficiency Study 100-Year Storm Excerpts

DEFICIENCY STUDY

Silver Strand Pump Stations

May 2014

Prepared For:

Ventura County Watershed Protection District
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REGISTERED PROFESSIONAL ENGINEER
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5-12-14

PACE JN A201

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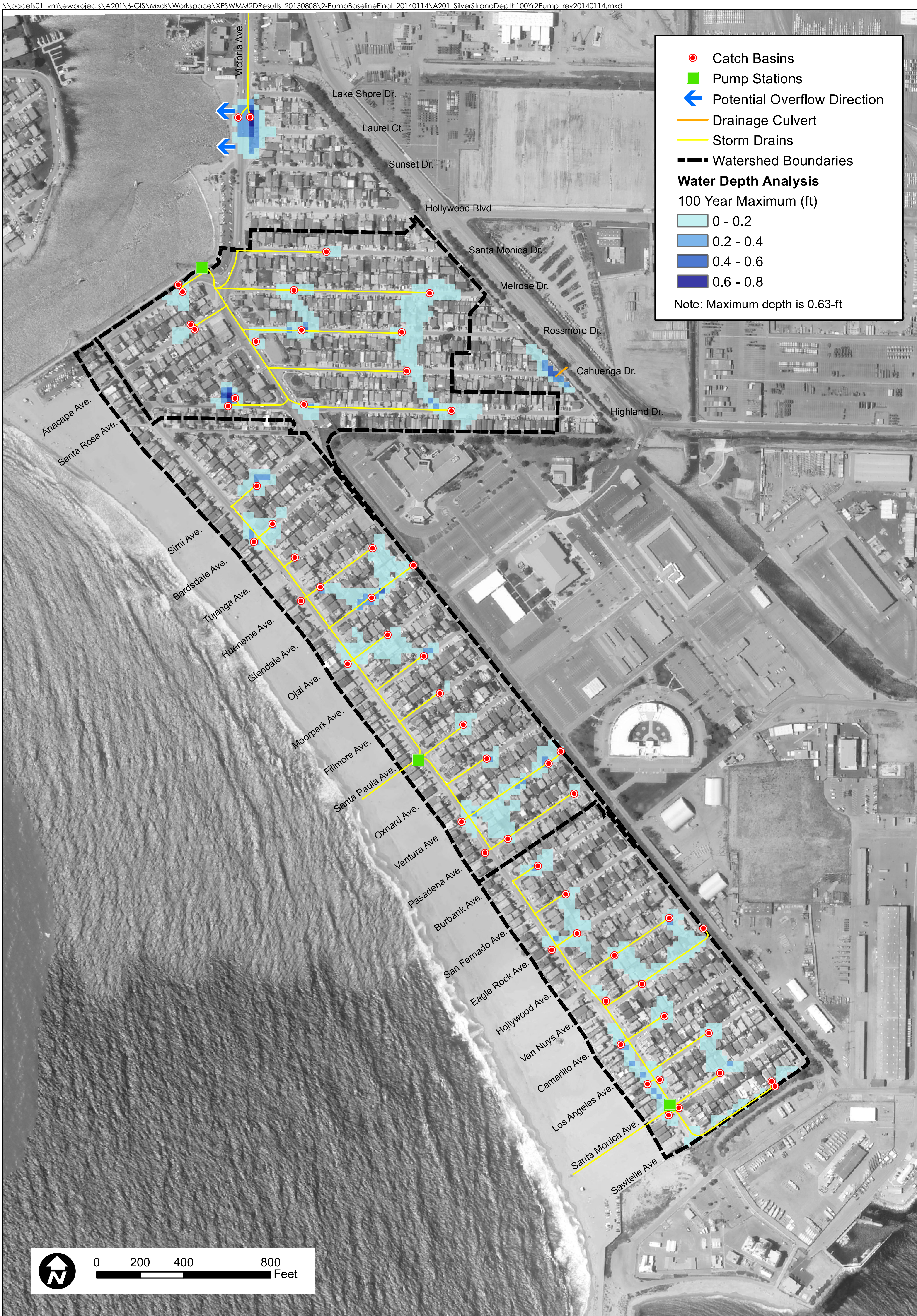

EXHIBIT 8	PACE Advanced Water Engineering	SCALE 1"= 400' DESIGNED AR/KH DRAWING TH CHECKED AR DATE 02/04/14 JOB NO. A201	JOB SILVER STRAND OXNARD CA

TITLE

100 YEAR MAXIMUM WATER DEPTH ANALYSIS

Baseline Alternative

Link Description	Link Name	Upstream Invert Elev (ft)	Downstream Invert Elev (ft)	Upstream Node Name	Downstream Node Name	Shape	Length (ft)	Diameter / Height (ft)	Bottom Width (ft)	Roughness	Design Full Flow (cfs)	Max Flow (cfs)	Max Velocity (ft/s)	Max Depth (ft)	Max Water Elev U/S (ft)	Max Water Elev D/S (ft)	Duration of Normal Flow (hrs)
Bardsdale Sta 0+00 to 0+98 Storm Drain	BD-1-E	6.43	6.23	BD-201-E	BD-200-MH	Circular	98	1.5	0	0.013	5	6	4.0	1.6	8.0	7.8	0
Bardsdale Sta 0+00 to 0+32 Storm Drain	BD-1-W	6.83	6.23	BD-201-W	BD-200-MH	Circular	32	1.25	0	0.013	9	2	5.1	1.5	7.7	7.8	1
Fillmore Sta 0+00 to 2+26 Storm Drain	FM-1	4.31	3.86	FM-201	FM-200-MH	Circular	226	1.5	0	0.013	5	3	3.0	1.3	5.4	5.2	10
Glendale Sta 0+00 to 2+40 Storm Drain (Grade Change Removed)	GD-1/2	5.43	4.63	GD-201/2	GD-200-MH	Circular	240	1.5	0	0.013	6	4	2.6	2.2	7.1	6.8	22
Glendale Sta 2+40 to 2+42 Storm Drain	GD-3	5.44	5.43	GD-203	GD-201/2	Circular	2	1.5	0	0.013	5	4	3.4	1.7	7.1	7.1	19
Glendale Sta 2+42 to 2+44 Storm Drain	GD-4	5.44	5.44	GD-204	GD-203	Circular	2	1	0	0.013	2	0	-1.4	1.7	7.1	7.1	19
Glendale Sta 2+44 to 4+87 Storm Drain	GD-5	7.38	5.44	GD-205	GD-204	Circular	243	1	0	0.013	3	0	1.5	1.7	7.6	7.1	33
Hueneme Sta 0+00 to 0+76 Storm Drain	HN-1-E	6.10	5.88	HN-201-E	HN-200-MH	Circular	76	1.5	0	0.013	6	2	2.7	1.3	7.3	7.2	0
Hueneme Sta 0+76 to 0+78 Storm Drain	HN-2-E	6.11	6.10	HN-202-E	HN-201-E	Circular	2	1.5	0	0.013	5	3	3.1	1.2	7.3	7.3	25
Hueneme Sta 0+78 to 0+80 Storm Drain	HN-3-E	6.11	6.11	HN-203-E	HN-202-E	Circular	2	1.5	0	0.013	5	1	1.3	1.2	7.3	7.3	35
Hueneme Sta 0+80 to 3+80 Storm Drain	HN-4-E	7.01	6.11	HN-204-E	HN-203-E	Circular	300	1.5	0	0.013	6	1	2.5	1.1	7.5	7.3	35
Hueneme Sta 0+00 to 0+32 Storm Drain	HN-1-W	6.93	6.23	HN-201-W	HN-200-MH	Circular	32	1.25	0	0.013	10	1	5.5	1.0	7.3	7.2	0
Moorpark Sta 0+00 to 2+75 Storm Drain	MP-1	4.53	3.97	MP-201	MP-200-MH	Circular	275	1.5	0	0.013	5	5	2.9	1.9	6.5	5.9	4
Ocean Sta 4+99.01 to 6+93 Storm Drain	OC-1-N-SP	2.82	2.43	OC-201-N	SP-200	Circular	194	3	0	0.013	30	36	6.2	2.4	5.2	4.5	17
Ocean Sta 6+93 to 6+95 Manhole	OC-2-N-SP	2.95	2.82	FM-200-MH	OC-201-N	Rectangular	2	4.5	4	0.013	545	56	6.5	2.4	5.2	5.2	20
Ocean Sta 6+95 to 6+97 Manhole	OC-3-N-SP	3.07	2.95	OC-202-N	FM-200-MH	Rectangular	2	4.5	4	0.013	524	49	7.5	2.2	5.2	5.2	20
Ocean Sta 6+97 to 8+73 Storm Drain	OC-4-N-SP	3.42	3.07	OC-203-N	OC-202-N	Circular	176	2.75	0	0.013	24	33	6.0	2.4	5.9	5.2	16
Ocean Sta 8+73 to 8+75 Manhole	OC-5-N-SP	3.43	3.42	MP-200-MH	OC-203-N	Rectangular	2	4.5	4	0.013	107	33	3.4	2.4	5.9	5.9	27
Ocean Sta 8+75 to 8+77 Manhole	OC-6-N-SP	3.43	3.43	OC-204-N	MP-200-MH	Rectangular	2	4.5	4	0.013	107	28	2.9	2.4	5.9	5.9	28
Ocean Sta 8+77 to 10+53 Storm Drain	OC-7-N-SP	3.78	3.43	OC-205-N	OC-204-N	Circular	176	2.75	0	0.013	24	28	4.9	2.5	6.3	5.9	32
Ocean Sta 10+53 to 10+55 Manhole	OC-8-N-SP	3.91	3.78	OJ-200-MH	OC-205-N	Rectangular	2	4.5	4	0.013	534	31	5.1	2.5	6.3	6.3	33
Ocean Sta 10+55 to 10+57 Manhole	OC-9-N-SP	4.03	3.91	OC-206-N	OJ-200-MH	Rectangular	2	4.5	4	0.013	534	28	5.7	2.4	6.3	6.3	33
Ocean Sta 10+57 to 12+33 Storm Drain	OC-10-N-SP	4.56	4.03	OC-207-N	OC-206-N	Circular	176	2.5	0	0.013	23	24	5.3	2.3	6.8	6.3	1
Ocean Sta 12+33 to 12+35 Manhole	OC-11-N-SP	4.57	4.56	GD-200-MH	OC-207-N	Rectangular	2	4	4	0.013	91	24	2.8	2.3	6.8	6.8	35
Ocean Sta 12+35 to 12+37 Manhole	OC-12-N-SP	4.57	4.57	OC-208-N	GD-200-MH	Rectangular	2	4	4	0.013	91	21	2.4	2.3	6.8	6.8	35
Ocean Sta 12+37 to 14+13 Storm Drain	OC-13-N-SP	5.09	4.57	OC-209-N	OC-208-N	Circular	176	2.5	0	0.013	22	21	4.8	2.3	7.2	6.8	23
Ocean Sta 14+13 to 14+15 Manhole	OC-14-N-SP	5.10	5.09	HN-200-MH	OC-209-N	Rectangular	2	3.5	4	0.013	76	21	2.6	2.1	7.2	7.2	35
Ocean Sta 14+15 to 14+17 Manhole	OC-15-N-SP	5.10	5.10	OC-210-N	HN-200-MH	Rectangular	2	3.5	4	0.013	76	18	2.2	2.1	7.2	7.2	35
Ocean Sat 14+17 to 15+83 Storm Drain	OC-16-N-SP	5.35	5.10	OC-211-N	OC-210-N	Circular	166	2.5	0	0.013	16	18	4.1	2.2	7.5	7.2	0
Ocean Sta 15+83 to 15+85 Manhole	OC-17-N-SP	5.36	5.35	TJ-200-MH	OC-211-N	Rectangular	2	3	4	0.013	62	19	2.4	2.2	7.5	7.5	35
Ocean Sta 15+85 to 15+87 Manhole	OC-18-N-SP	5.36	5.36	OC-212-N	TJ-200-MH	Rectangular	2	3	4	0.013	62	17	2.2	2.2	7.5	7.5	35
Ocean Sta 15+87 to 17+62.54 Storm Drain	OC-19-N-SP	5.62	5.36	OC-213-N	OC-212-N	Circular	176	2.5	0	0.013	16	17	3.7	2.3	8.0	7.5	24
Ocean Sta 17+62.54 to 17+64.54 Manhole	OC-20-N-SP	5.87	5.62	BD-200-MH	OC-213-N	Rectangular	2	2.75	4	0.013	388	42	6.1	2.3	7.8	8.0	35
Ocean Sta 17+64.54 to 17+66.54 Manhole	OC-21-N-SP	6.12	5.87	OC-214-N	BD-200-MH	Rectangular	2	2.75	4	0.013	388	33	8.1	1.9	7.9	7.8	35
Ocean Sta 17+66.54 to 19+41.54 Storm Drain	OC-22-N-SP	6.47	6.12	OC-215-N	OC-214-N	Circular	175	2	0	0.013	10	8	3.5	1.8	7.9	7.9	0
Ocean Sta 19+41.54 to 19+43.54 Manhole	OC-23-N-SP	6.48	6.47	SI-200-MH	OC-215-N	Rectangular	2	3	4	0.013	62	8	1.5	1.5	7.9	7.9	35
Ocean Sta 19+43.54 to 19+45.54 Manhole	OC-24-N-SP	6.48	6.48	OC-216-N	SI-200-MH	Rectangular	2	3	4	0.013	62	0	-1.6	1.5	7.9	7.9	23
Ocean Sta 0+00 to 1+76 Storm Drain	OC-1-S-SP	3.14	2.93	OC-201-S	SP-200	Circular	176	2.5	0	0.013	14	14	4.5	1.7	4.8	4.5	20
Ocean Sta 1+76 to 1+78 Manhole	OC-2-S-SP	3.27	3.14	ON-200-MH	OC-201-S	Rectangular	2	3.5	4	0.013	382	33	6.1	1.7	4.8	4.8	26
Ocean Sta 1+78 to 1+80 Manhole	OC-3-S-SP	3.39	3.27	OC-202-S	ON-200-MH	Rectangular	2	3.5	4	0.013	382	26	6.0	1.5	4.8	4.8	25
Ocean Sta 1+80 to 3+56 Storm Drain	OC-4-S-SP	3.74	3.39	OC-203-S	OC-202-S	Circular	176	2	0	0.013	10	11	4.5	1.7	5.4	4.8	24
Ocean Sta 3+56 to 3+58 Manhole	OC-5-S-SP	3.94	3.74	VT-200-MH	OC-203-S	Rectangular	2	3.5	4	0.013	483	29	5.6	1.7	5.3	5.4	29
Ocean Sta 3+58 to 3+60 Manhole	OC-6-S-SP	4.14	3.94	OC-204-S	VT-200-MH	Rectangular	2	3.5	4	0.013	483	22	5.4	1.3	5.4	5.3	35
Ocean Sta 3+60 to 5+36 Storm Drain	OC-7-S-SP	4.66	4.14	OC-205-S	OC-204-S	Circular	176	1.5	0	0.013	6	4	3.6	1.3	5.7	5.4	0
Ocean Sta 5+36 to 5+38 Manhole	OC-8-S-SP	4.67	4.66	PD-200-MH	OC-205-S	Rectangular	2	3	4	0.013	62	4	1.1	1.0	5.7	5.7	35
Ocean Sta 5+38 to 5+40 Manhole	OC-9-S-SP	4.67	4.67	OC-206-S	PD-200-MH	Rectangular	2	3	4	0.013	62	0	-1.2	1.0	5.7	5.7	17
Ojai Sta 0+00 to 1+96 Storm Drain (Grade Change Removed)	OJ-1/2-E	5.22	4.60	OJ-201/2-E	OJ-200-MH	Circular	196	1.5	0	0.013	6	3	2.7	1.7	6.4	6.3	0
Ojai Sta 0+00 to 0+32 Storm Drain	OJ-1-W	6.03	5.18	OJ-201-W	OJ-200-MH	Circular	32	1.25	0	0.013	11	1	4.6	1.1	6.3	6.3	0
Oxnard Sta 0+00 to 2+30 Storm Drain	ON-1	4.70	4.01	ON-201	ON-200-MH	Circular	230	1.5	0	0.013	6	3	3.4	0.8	5.5	4.8	2
Pasadena Sta 5+38 to 6+34 Storm Drain	PD-1-E	5.03	4.67	PD-201-E	PD-200-MH	Circular	94	1.5	0	0.013	7	3	3.3	1.0	5.9	5.7	35
Pasadena Sta 6+34 to 6+36 Storm Drain	PD-2-E	5.04	5.03	PD-202-E	PD-201-E	Circular	2	1.5	0	0.013	5	3	3.4	0.9	5.9	5.9	0
Pasadena Sta 6+36 to 6+38 Storm Drain	PD-3-E	5.04	5.04	PD-203-E	PD-202-E	Circular	2	1	0	0.013	2	1	2.1	0.9	5.9	5.9	23
Pasadena Sta 6+38 to 9+98 Storm Drain	PD-4-E	6.83	5.04	PD-204-E	PD-203-E	Circular	360	1	0	0.013	3	1	3.1	0.8	7.4	5.9	35
Pasadena Sta 5+06 to 5+38 Storm Drain	PD-1-W	6.23	4.93	PD-201-W	PD-200-MH	Circular	32	1.25	0	0.013	13	1	4.9	0.7	6.5	5.7	1
Simi Sta 19+43.54 to 20+92.54 Storm Drain	SI-1	6.70	6.48	SI-201	SI-200-MH	Circular	149	1.5	0	0.013	4	8	4.5	2.0	8.8	7.9	1
Santa Paula Sta 0+00 to 2+30 Storm Drain	SP-1	4.88	4.19	SP-201	SP-200	Circular	230	1.5	0	0.013	6	3	3.2	0.8	5.6	4.8	0
Santa Paula Outfall Structure	SP-P-1	4.43	7.37	SP-P-201	SP-P-200	Circular	5	2.5	0	0.013	332	-58	-12.1	5.3	9.8	9.8	0
Sta 2+00 to 4+74.01 Santa Paula Outfall Pipe	SP-P-2	5.42	4.43	SP-P-202	SP-P-201	Circular	274	2.5	0	0.013	25	58	11.7	11.3	16.7	9.8	16
Sta 4+74.01 to 4+79.01 Santa Paula Pump Station Outlet Pipe	SP-P-3	5.43	5.42	SP-P-203	SP-P-202	Circular	5	1.67	0	0.013	8	27	12.1	11.5	16.9	16.7	16
Sta 4+79.01 to 4+99.01 Santa Paula Pump Station	SP-P-4	Pump		SP-200	SP-P-203						Pump						
Tujunga Sta 0+00 to 0+66 Storm Drain	TJ-1	6.23	6.04	TJ-201	TJ-200-MH	Circular	66	1.25	0	0.013	3	2	2.8	1.5	7.6	7.5	0
Ventura Sta 0+00 to 4+43 Storm Drain	VT-1-E	4.43	3.89	VT-201-E	VT-200-MH	Circular	443	1.5	0	0.013	4	4	2.6	1.4	5.9	5.3	12
Ventura Sta 4+43 to 4+45 Storm Drain	VT-2-E	4.44	4.43	VT-202-E	VT-201-E	Circular	2	1.5	0	0.013	5	4	2.9	1.4	5.9	5.9	35
Ventura Sta 4+45 to 4+47 Storm Drain	VT-3-E	4.44	4.44	VT-203-E	VT-202-E	Circular	2	1	0	0.013	2	0	-1.6	1.4	5.9	5.9	35
Ventura Sta 4+47 to 5+32 Storm Drain	VT-4-E	5.29	4.44	VT-204-E	VT-203-E	Circular	85	1	0	0.013	4	0	1.0	1.4	5.9	5.9	35
Ventura Sta 0+00 to 0+32 Storm Drain	VT-1-W	6.43	4.39	VT-201-W	VT-200-MH	Circular	32	1.25	0	0.013	16	0	5.3	0.9	6.6	5.3	0