Sonoma Water
Penngrove Sanitation Zone

Sanitary Sewer
Capacity Evaluation

Final Report

Prepared By:

Woodard & Curran

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LIST OF ABBREVIATIONS

AC	Asbestos Concrete
ADU	Accessory Dwelling Unit
ADWF	Average Dry Weather Flow
BWF	Base Wastewater Flow
CCTV	Closed-Circuit Television
CDF	Cumulative Distribution Function
County	Sonoma County
DEM	Digital Elevation Model
DDF	Depth-Duration-Frequency
DU	Dwelling Unit
ESD	Equivalent Single-Family Dwelling Unit
fps	Feet per Second
gpd	Gallons per Day
GWI	Groundwater Infiltration
HGL	Hydraulic Grade Line
HWY	Highway
ICM	InfoWorks ICM
IDF	Intensity-Duration-Frequency
I/I	Infiltration and Inflow
LS	Lift Station
MACP	Manhole Assessment Certification Program
mgd	Million Gallons per Day
MFR	Multi-Family Residential
NGVD29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
PACP	Pipeline Assessment Certification Program
PDWF	Peak Dry Weather Flow
PG&E	Pacific Gas and Electric Company
PS	Pump Station
PSZ	Penngrove Sanitation Zone
PWWF	Peak Wet Weather Flow
PVC	Polyvinyl Chloride
RDI/I	Rainfall-Dependent Inflow and Infiltration
SCS	Soil Conservation Service
SF	Square Feet
SFR	Single-Family Residential
SSO	Sanitary Sewer Overflow
TR-55	Technical Release 55 – Urban Hydrology for Small Watersheds
UNK	Unknown (pipe material)
USDA	United States Department of Agriculture
USGS	United States Geological Survey
VCP	Vitrified Clay Pipe
WH	Workforce Housing
WDR	Waste Discharge Requirements

1. INTRODUCTION

This Penngrove Sewer Capacity Evaluation Report (Capacity Report) summarizes the development of the hydraulic model of the Sonoma Water Penngrove Sanitation Zone (Penngrove or PSZ) sewer collection system, and its subsequent use to assess system performance and identify projects that will improve the existing level of service. The purpose of the analysis presented in this Capacity Report was to evaluate the capacity of the system to handle peak wet weather flow (PWWF) as required by the State Water Resources Control Board's (SWRCB's) adopted Order WQ 2022-0103-DWQ[1], a General Waste Discharge Requirement (WDR) for nearly all publicly-owned sanitary sewer collection systems in California. The WDR requires a System Evaluation, Capacity Assurance, and Capital improvement Plan that includes procedures and activities for:

- Routine evaluation and assessment of system conditions;
- Capacity assessment and design criteria;
- Prioritization of corrective actions; and
- A capital improvement plan.

Penngrove is an unincorporated community in Sonoma County, California, situated between the cities of Petaluma and Cotati. The Penngrove collection system, serving the PSZ, is owned and operated by the Sonoma County Water Agency (Sonoma Water). PSZ has 376 service connections across roughly 475 acres of service area and provides sewer services to approximately 547 equivalent single family dwelling units (ESDs). Flow from Penngrove is conveyed to the Penngrove Lift Station and pumped to the City of Petaluma (City or Petaluma) collection system through a 6-inch diameter force main that can discharge either to the City's gravity sewer on Corona Road, which drains to the City's Wilmington Lift Station, or directly to the City's Wilmington Lift Station. Flows from Petaluma's collection system are conveyed to and treated at the City's Ellis Creek Water Recycling Facility.

[1]SWRCB's Order WQ 2022-0103-DWQ (Statewide Waste Discharge Requirements, General Order for Sanitary Sewer Systems) available at: https://www.waterboards.ca.gov/board_decisions/adopted_orders/water_quality/2022/wqo_2022-0103-dwq.pdf

2. HYDRAULIC MODEL DEVELOPMENT

This section describes the development of the hydraulic model that was used to assess the capacity of Penngrove's sewer collection system. The section provides an overview of the model development process, including descriptions of the modeled sewer network and subcatchments, the flow monitoring program conducted for this study, the basis for estimating wastewater flows, and the calibration of the model.

The fully dynamic hydraulic modeling software InfoWorks® ICM by Autodesk (InfoWorks or ICM) was used for the study and has been used to model many other collection systems in the Bay Area, including Petaluma, Santa Rosa, and Novato Sanitary District. Several of Sonoma Water's other wastewater collection systems, including the Sonoma Valley and Russian River County Sanitation Districts, the Airport-Larkfield-Wikiup Sanitation Zone, and the Geyserville Sanitation Zone, have also been modeled using ICM.

2.1 Modeling Terminology

Key modeling terms are defined below.

- **Network** refers to the representation of the physical facilities being modeled. Modeled network components include pipes (conduits), manholes, and lift stations (also referred to as pump stations), and force mains.
- **Nodes** are primarily manholes but also include lift station wet wells and outfalls (discharge points from the modeled system). Key data associated with nodes include manhole rim elevations and lift station wet well elevations and cross-sectional areas.
- **Pipes** or **conduits** are connections between nodes and include both gravity sewers and force mains. Key data associated with pipes are upstream and downstream node IDs, pipe length, diameter, roughness and headloss factors, and upstream and downstream invert elevations.
- **Pumps** are modeled individually, connecting lift station wet wells with the upstream node of associated force mains. Data associated with pumps include type (e.g., fixed or variable speed), on and off levels, pump capacities, and pump discharge curves.
- **Subcatchments** (also called **sewersheds** or **subbasins**) are areas that contribute flow to the modeled sewer network and represent the unmodeled sewers in the collection system. Data associated with subcatchments include sanitary flow (computed based on population, water use, or other available data), type of diurnal sanitary flow profile (which is a function of land use), infiltration/inflow (I/I) parameters, and the node at which the flow from the subcatchment enters the modeled system.
- **Model loads** are the flows entering the modeled sewer system from each subcatchment. Model loads include estimated residential and commercial sanitary or base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent I/I (RDI/I). As a sum, they represent the total wastewater flow applied to the model.
- **Models** are the combination of a modeled network, its associated subcatchments and loads, and other data (e.g., rainfall, diurnal profiles, inflows from other areas, etc.) that comprise a specific model scenario.

2.2 Modeled Network

The PSZ network modeled for this study incorporates 6, 8, and 10-inch sewers. The upstream portion of the modeled network includes three 8-inch diameter pipe branches along Old Redwood Highway and Malcolm Lane, Petaluma Hill Road, and Davis Lane and Bannon Lane. These three pipe branches discharge into one 10-inch diameter trunk sewer that runs parallel to the west side of Lichau Creek and terminates at the Penngrove Lift Station. Flows from the Penngrove Lift Station are then pumped through the Penngrove force main which discharges to the City of Petaluma's system. The model also includes 6-inch siphons: two parallel at the Lichau Creek crossing upstream of the Penngrove Lift Station, and another at the downstream end of the middle branch sewer. In total, the model network includes about 32 percent of the total length of sewers in the entire collection system (3.1 miles of modeled sewers out of 9.5 miles total). The modeled trunk network is shown in **Figure 2-1**.

The PSZ service area was divided into model subcatchments. Each subcatchment is represented as a single parcel that "loads" to a manhole in the modeled network. The model includes 435 subcatchments, representing one subcatchment for each parcel in the service area, and ranging in size from less than 0.1 acre to nearly 11 acres. Although the subcatchments capture all parcels in the service area, not all parcels are currently developed or connected to the sewer system.

2.2.1 Model Network Construction and Validation

The primary data used to create the Penngrove model network (i.e., pipes, manholes, sewer loads) were provided by Sonoma Water in the form of Geographic Information Systems (GIS) shapefiles. The pipes and manholes included in the modeled trunk network, described in **Section 2.2**, were extracted from those shapefiles before they were imported into the InfoWorks modeling environment. Publicly available GIS parcel data, sourced from Sonoma County (County), were used to represent the model subcatchments.

The model construction and validation process included the following steps:

- The modeled network was checked for connectivity, i.e., verifying that the correct upstream/ downstream manholes were identified for each pipe and that there were no missing pipes in the network;
- Model loading manholes were assigned to all subcatchments;
- Surveyed elevation data were incorporated where available;
- Profiles were plotted for each series of pipe segments in the modeled network to visually check for missing or suspect data;
- Where invert elevation data were missing or inconsistent with nearby elevations, and not determined through record drawings or survey, interpolations between known values were used as appropriate (and flagged, as noted in the next bullet). Elevations for this model are based on the NGVD29 datum, for consistency with the source data;
- The sources of model data (e.g., GIS shapefiles, record drawings, survey data, etc.) were documented using "flags" in the model database;

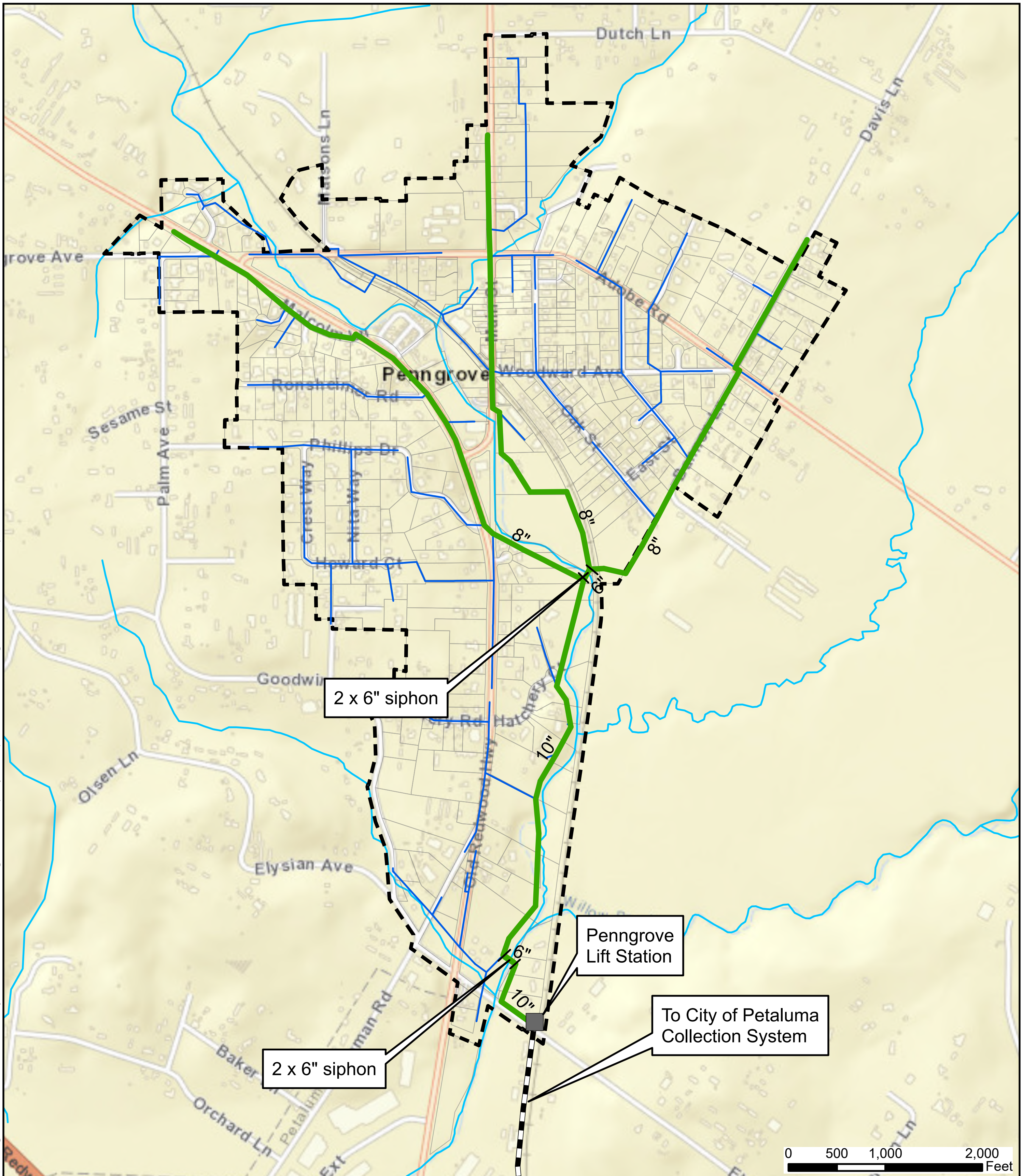

Figure 2-1
Modeled Network

Penngrove Sewer Capacity Evaluation Report

Penngrove Sanitation Zone

Legend

- Modeled Trunk Sewers
- Unmodeled Sewers
- Force Main
- Penngrove Lift Station
- Parcels
- PSZ Service Area
- Streams

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Map Created: April 2026

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- A ground model was built using 1-meter digital elevation model (DEM) tiles downloaded from the USGS National Map[1], and was used to define manhole rim elevations where survey or record drawing information were not available and;
- Gravity pipelines were modeled assuming a pipe roughness factor (i.e., Manning's n value) based on the pipe material type and in accordance with the values defined by Sonoma Water for all sewer capacity studies. The Manning's n values assigned to each pipe material, along with the total modeled pipe lengths in the PSZ system, are summarized in **Table 2-1**. As noted in the table, asbestos cement is the primary pipe material in the PSZ collection system.

TABLE 2-1: MODELED MANNING'S N VALUES

Manning's n Value	Pipe Material	Total Pipe Length (ft)
0.010	Polyvinyl Chloride (PVC)	1,532
0.014	Asbestos Cement (AC)	13,700
0.015	Cast Iron (CAS)	192
	Vitrified Clay Pipe (VCP)	446
	Unknown (UNK)	749

2.3 Flow Monitoring Program

To support development of the hydraulic model, a temporary flow monitoring program consisting of four (4) temporary flow meters and one (1) rain gauge was implemented for the three-month period from January 17 to April 17, 2024[2]. The purpose of the flow monitoring program was to obtain data to quantify flows and characterize I/I in the system and to use the data to calibrate the hydraulic model for both dry weather and wet weather conditions.

The temporary flow meters were located just upstream of the lift station, at manhole PSZ_M18-4, and on each of the 8-inch branch trunk sewers, at manholes PSZ_M14-12, PSZ_M15-12, and PSZ_M21-1. The locations of the 2024 flow meters are presented in **Figure 2-3**, which also shows the location of the Ely Road North temporary rain gauge that was installed in proximity to Penngrove's service area during this flow monitoring program.

Figure 2-4 shows a plot of measured data at the flow meter just upstream of the lift station for the January through April 2024 monitoring period; graphs of the data for all flow meters are included in **Appendix A**. As indicated in **Figure 2-4** by the depth measurement, up to approximately seven (7) feet of backup surcharge occurred in late January and early February 2024 and was caused by capacity limitations at the Penngrove Lift Station. Backup surcharge refers to wastewater backing up in the sewer pipe and rising above the top of the pipe due to downstream conditions. The measured surcharge height indicates how far the wastewater level rose within the sewer system during the event. During this period, Sonoma Water used

[1] Elevation data were downloaded from the USGS 3D Elevation Program (3DEP), available here: https://www.sciencebase.gov/catalog/item/5eaa4f2a82cefae35a220e0f (last updated 2013).

[2] A temporary flow meter was also installed along Penngrove's 10-inch diameter trunk sewer (at manhole PSZ_M18-4, approximately 10 feet upstream of the Penngrove Lift Station) for a two-month period (February 2 through April 2, 2022). However, the rainfall that season was not considered adequate for wet weather model calibration.

trucks to transport wastewater flows to Petaluma's sewer collection system, discharging them at Corona Road, while the Penngrove Lift Station simultaneously conveyed wastewater to Petaluma through the Penngrove force main. Because the nozzle discharging sewage into the trucks was located upstream of the Penngrove Lift Station's permanent flow meter, the permanent flow meter recorded only pumped flow, but the temporary flow meter incorporated both pumped flow and trucked flow.

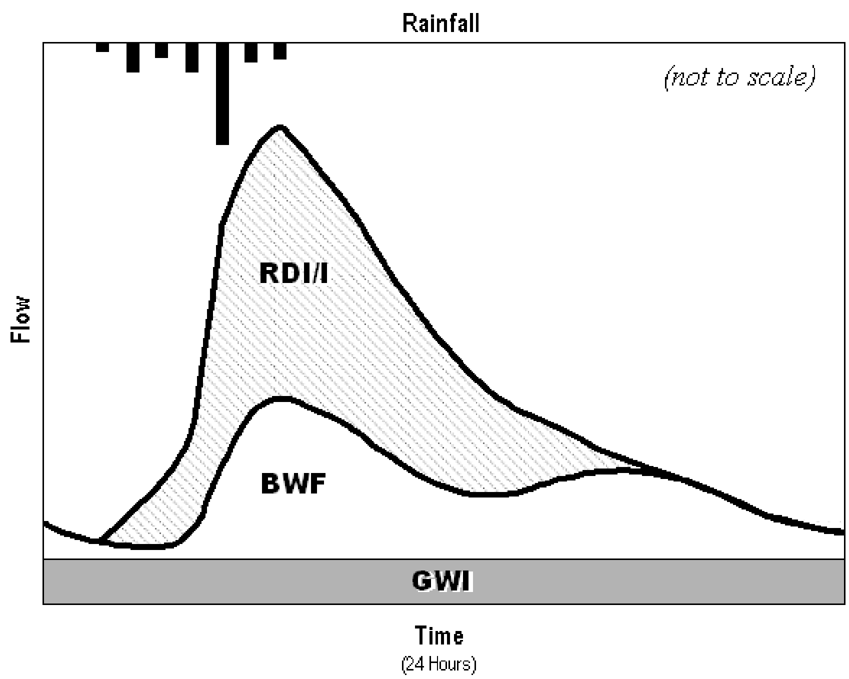
2.4 Flow Estimating Methodology

This section presents the basis of wastewater flow estimates for the PSZ system. It describes the wastewater flow components used in the hydraulic model and the existing and projected future land uses for Penngrove, which form the basis for generating base wastewater flows that are applied to the model.

2.4.1 Wastewater Flow Components

Wastewater flows include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I), as illustrated conceptually in **Figure 2-2.**

FIGURE 2-2: WASTEWATER FLOW COMPONENTS



Base Wastewater Flow (BWF)

BWF represents the sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system. BWF varies throughout the day but typically follows predictable diurnal patterns depending on the type of land use.

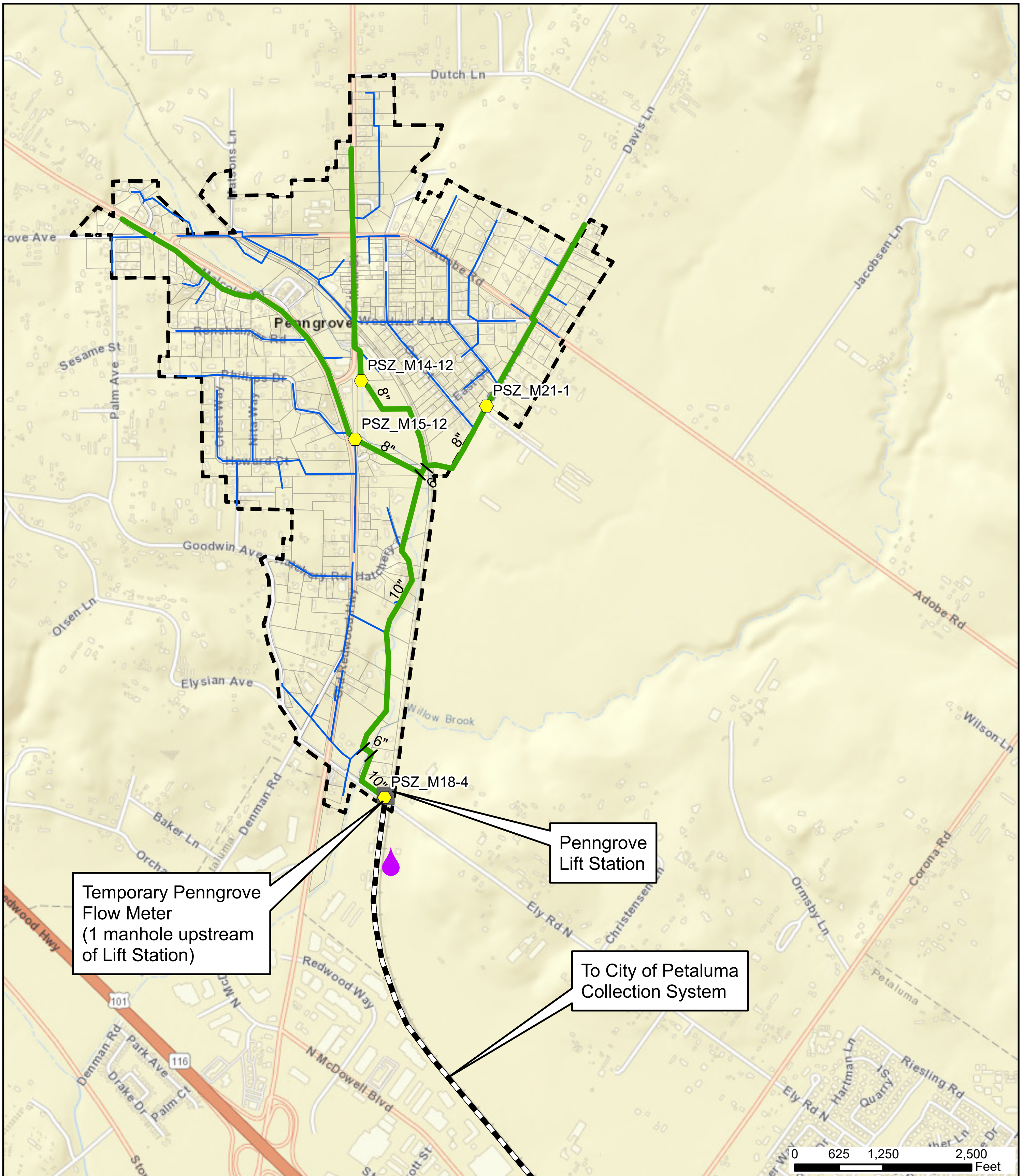


Figure 2-3: 2024 Flow Meter Locations

Penngrove Sewer Capacity Evaluation Report

Penngrove Sanitation Zone

Legend

- Ely Road Rain Gauge
- Penngrove Flow Meters
- Penngrove Lift Station
- Force Main
- Modeled Trunk Sewers
- Unmodeled Sewers
- Parcels
- PSZ Service Area

N

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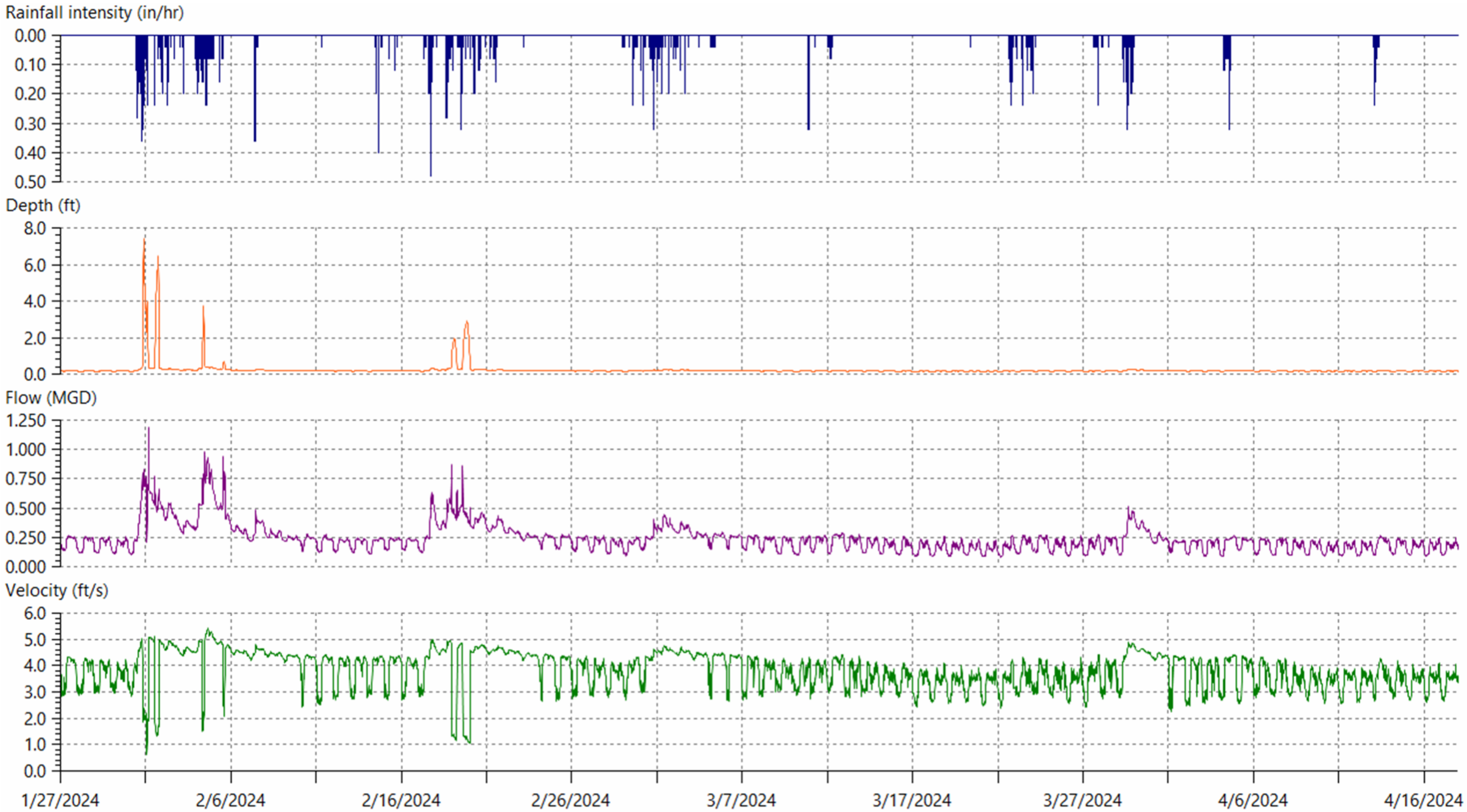
FIGURE 2-4: PLOT OF FLOW DATA FOR FLOW MONITORING PERIOD (PSZ_M18-4 – UPSTREAM OF PENNGROVE LS)



Groundwater Infiltration (GWI)

GWI represents groundwater that infiltrates into defects in sewer pipes and manholes, particularly in winter and springtime in low-lying areas and locations subject to frequent flooding. GWI is typically seasonal in nature and remains relatively constant during specific periods of the year. However, rainfall typically has long-term impacts on GWI rates, as evidenced by measurable increases in GWI after prolonged periods of rainfall.

Rainfall-Dependent Infiltration and Inflow (RDI/I)

RDI/I represents storm water inflow and infiltration that enter the system in direct response to rainfall events, either through direct connections such as holes in manhole covers or illegally connected roof leaders or area drains, or, more commonly, through defects in sewer pipes, manholes, and service laterals. RDI/I typically result in short term peak flows that recede relatively quickly after the rainfall ends. The magnitude of RDI/I flows are related to the intensity and duration of the rainfall, the relative soil moisture at the time of the rainfall event, and the condition of the sewers. Base Wastewater Flow

Existing and future residential and non-residential base wastewater flows were estimated using information compiled at the parcel level.

Existing Average BWF

There are 547 equivalent single family dwelling units (ESDs) in the Penngrove service area based on 376 service connections. ESD data were included in a GIS dataset[1] that was provided by Sonoma Water. The shapefile was joined to the publicly available Sonoma County parcels shapefile to aggregate the ESD data by parcel. The resulting parcel shapefile containing the ESD data was imported into the InfoWorks model network as 435 total subcatchments (including currently unconnected parcels in the service area). An appropriate loading manhole was assigned to each subcatchment in the modeled sewer system.

A representative unit flow factor of 170 gpd per ESD was estimated during calibration (discussed in **Section 2.5**), resulting in a total BWF to the PSZ system of approximately 92,990 gpd (0.093 mgd). A table representing the number of ESDs and corresponding existing BWF for different land use categories is summarized in **Table 2-2**. The existing BWF estimates are presented as daily averages.

TABLE 2-2: EXISTING AVERAGE BASE WASTEWATER FLOW

Site Category	ESDs[1]	Average BWF (gpd)
Single Family (without existing ADU)	287	48,790
Single Family (with existing ADU)	45	7,650
Multi-Family	124	21,080
Commercial/Industrial	91	15,470
Total Existing	**547**	**92,990**

[1]Rounded to the nearest ESD.

[1] *PSZ_ParcelConnection.shp* (June 15, 2022)

Future Average BWF

Future BWF was estimated on a parcel basis using development planning information from Sonoma County, specifically potential housing opportunity sites[1] (including rezone sites) within the Penngrove portion of the Rohnert Park/Cotati planning area. **Figure 2-5** shows the parcels identified as potential future housing sites within the PSZ.

Note that some of the proposed rezone sites were assigned a new zoning code, "WH" or "Workforce Housing Combining District", which allows for the construction of workforce housing on existing commercial-zoned and industrial-zoned parcels. Per the County's Housing Element, this zoning designation "*allows high-density residential development on commercial- or industrial-zoned land, in place of or accompanying commercial or industrial uses.*" Where residential units are proposed on these parcels, the future BWF estimates for the proposed residential units were added on top of the existing commercial and/or industrial loads. One of the parcels with the proposed "WH" zoning designation does not have existing loads; for that parcel, new commercial retail loads were estimated based on 50-percent lot coverage per Sonoma County's zoning code, in addition to the proposed residential units.

Future BWF for the housing element sites was estimated by multiplying the unit flow factors shown in **Table 2-3** (based on zoning / proposed land use type) by the number of planned dwelling units (DUs). The estimated BWF for each future development parcel was then converted to an ESD-based flow (assuming 1 ESD = 170 gpd).

TABLE 2-3: UNIT BASE WASTEWATER FLOW FACTORS FOR FUTURE DEVELOPMENT

Development Type	Unit	BWF Factor[1]
Single Family Residential (SFR)	Dwelling Units	170 gpd/DU[2]
Multi-Family Residential (MFR)	Dwelling Units	136 gpd/DU[3]
Commercial/Retail	Square Footage	0.13 gpd/SF

[1]BWF Factors are consistent with other Sonoma Water sewer capacity studies.
[2]BWF from 1 ESD based on dry weather calibration.
[3]Assumed as 80-percent of SFR flow factor (consistent with Sonoma Water billing structure).

Future BWF estimates were also added to the model future load scenario to account for 1) potential future connection to the sewer system of non-vacant parcels currently on septic systems, and 2) potential development and connection of currently vacant parcels with developable land use classifications. Sonoma Water's "PSZ_Pts_ML" GIS layer (December 2024) was used to identify unconnected parcels where the ESD and charge code fields were equal to zero (0). From that list, parcels were overlaid with the 2020 General Plan Land Use to determine allowable density used to estimate number of potential future units for each parcel.

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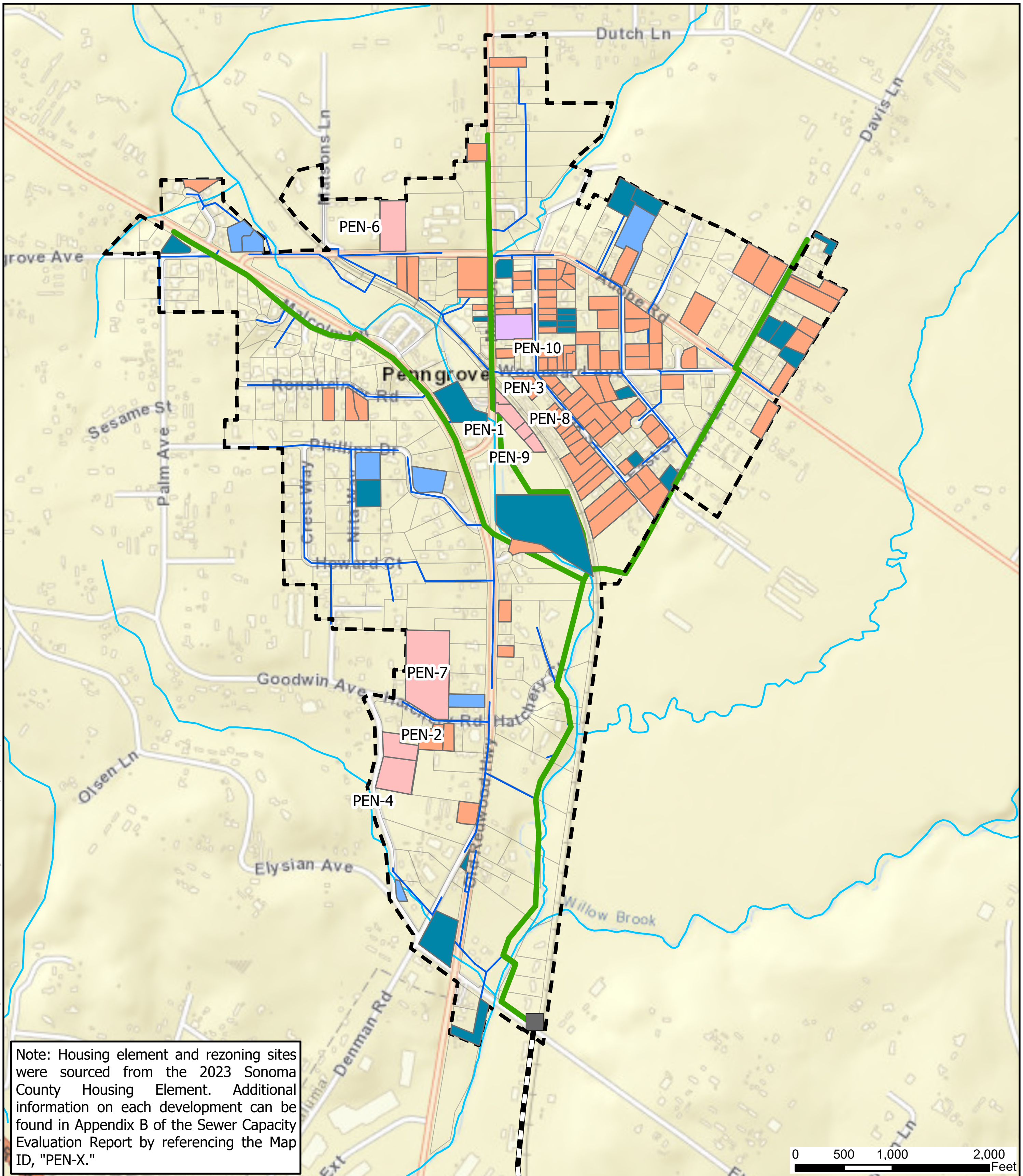
[1] County Housing Element Update (adopted August 2023), available at: https://permitsonoma.org/housingelement.



Note: Housing element and rezoning sites were sourced from the 2023 Sonoma County Housing Element. Additional information on each development can be found in Appendix B of the Sewer Capacity Evaluation Report by referencing the Map ID, "PEN-X."

Figure 2-5
Potential Future Development

Penngrove Sewer Capacity Evaluation Report
Penngrove Sanitation Zone

Legend

- Penngrove Lift Station
- Modeled Trunk Sewers
- Unmodeled Sewers
- Force Main
- All Parcels
- PSZ Service Area
- Potential Septic Conversion Parcels
- Housing Element Sites
- Rezoning Sites
- Vacant Developable Parcels
- Existing Residential Parcel with Possible ADU (assumed 10% of SFD parcels)
- Streams

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Existing parcels with a Use Code Description equal to "Single Family Dwelling" were assumed to have additional potential future loads from construction of Accessory Dwelling Units (ADUs). Overall, it was assumed that 10 percent of existing residential parcels would add ADUs in the future. This was accounted for in the model future load scenario by adding 10-percent of the existing ESD value to each single-family parcel subcatchment's estimated load. Similar ADU assumptions were applied to existing multi-family parcels with existing ESDs; for these parcels, either an additional 10-percent of the existing ESD value was applied, if the parcel had less than 6.4 existing ESDs (8 MFR DUs), or 10-percent of 6.4 ESDs (i.e., 0.64 ESD) was applied, if the parcel had at least 6.4 existing ESDs. Total future BWF assumptions, including the potential housing opportunity / housing element and rezoning sites, assumed non-vacant septic to sewer connections, assumed vacant developable connections, and estimated ADUs are shown in **Table 2-4**. The future BWF estimates are presented as daily averages.

TABLE 2-4: ESTIMATED FUTURE AVERAGE BASE WASTEWATER FLOW

Site Category	ESDs[1]	Average BWF (gpd)
Existing	**547**	**92,990**
Future Development	**318**	**54,060**
Housing Element[2]	178	30,260
ADU Assumption Single Family	29	4,930
ADU Assumption Multi-Family	9	1,530
Non-Vacant Septic Connection	11	1,870
Vacant Developable	91	15,470
Total Future	**865**	**147,050**

[1]Rounded to the nearest ESD.

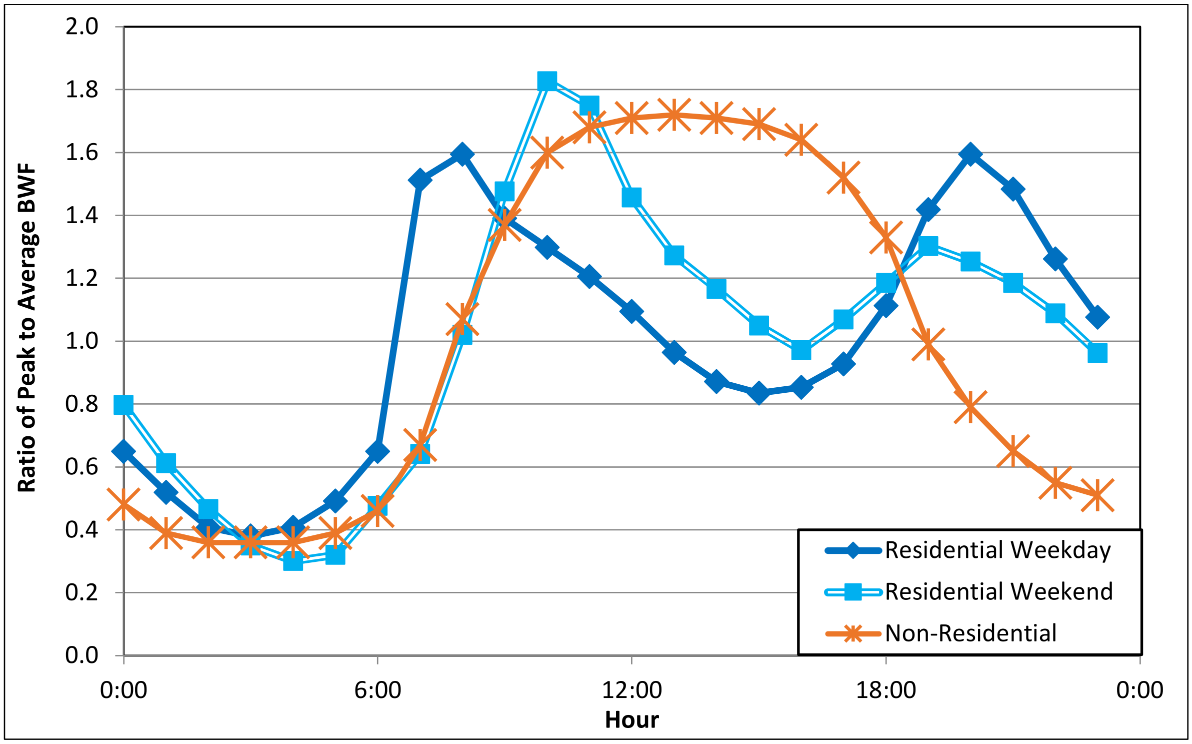
[2]The Housing Element includes rezoned sites.

The list of individual future development sites, and their corresponding BWF estimates, is included as **Appendix B**. For developed parcels identified for potential redevelopment, it was assumed that the future BWF would replace the existing BWF (except for parcels rezoned as "WH", as discussed above). For developed parcels that are not identified for redevelopment, the existing BWF based on the ESD data was assumed to characterize their BWF in the future.

BWF Diurnal Patterns

BWF varies throughout the day in a typical way, generally peaking early in the morning in most predominantly residential areas, such as Penngrove. Typical hourly peaks from residential areas tend to be about twice the average daily flow. Higher peaks can occur on atypical days of the year (e.g., on major holidays such as Thanksgiving or at halftime on Super Bowl Sunday). For Penngrove, typical diurnal patterns (or profiles) were developed to represent the diurnal variations in residential and commercial/industrial (non-residential) wastewater flows, for both weekend and weekday conditions. The profiles were developed based on typical flow patterns used for similar areas as well as patterns observed in Penngrove's flow monitoring data. The profiles are applied to the subcatchment BWF in the model. Diurnal profiles used in the Penngrove model are shown in **Figure 2-6**.

FIGURE 2-6: DIURNAL PROFILES



2.4.2 Groundwater Infiltration

GWI represents a seasonal increase in wastewater flows due to infiltration into the sewers, typically in low-lying areas or areas close to creeks or other water bodies. GWI, if present, is applied in the model as a constant flow in addition to the BWF. The amount of GWI in any particular area of the sewer system is determined during model calibration by comparing the modeled flows to the actual observed dry weather (non-rainfall period) flows at points in the system where flow data are available (e.g., at flow meter sites). Where modeled BWF is less than monitored dry weather flow by a constant value throughout the day, the difference is assumed to represent GWI. The GWI determined at the monitoring location is then distributed to the upstream meter tributary area on a weighted per-contributing area basis. Contributing area for each connected parcel was set equal to the parcel area (lot size); exceptions included capping contributing area at 1 acre for non-vacant single family residential parcels and at 5 acres for non-vacant non-residential parcels to avoid over-weighting the GWI contributions of lightly developed parcels. Note that because GWI is seasonal in nature, the modeled GWI is intended to represent a typical GWI rate during the wet weather season (wintertime) rather than a dry season (summertime) GWI rate.

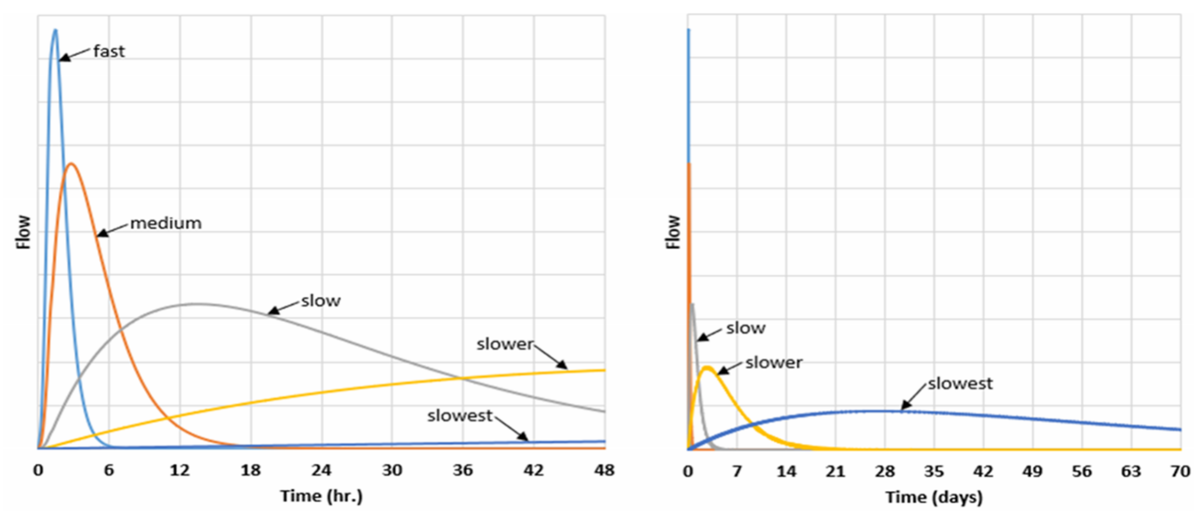
2.4.3 Rainfall-Dependent Inflow and Infiltration

RDI/I flows result from rainfall events that produce infiltration and inflow of storm water runoff into the sewer system. RDI/I can be quantified as the difference between the total flow during and immediately following a storm event and the non-rainfall "dry weather flow" (BWF plus GWI) that is estimated to have occurred during the storm period. RDI/I varies depending on many factors including the magnitude and intensity of the storm event, area topography, type of soil and the degree of soil saturation (due to antecedent rainfall) prior to the storm event, and the condition of the sewers, manholes, and service laterals. RDI/I is usually expressed as a volume or a percentage of the rainfall volume (termed the "R value") entering the sewer system from subcatchment contributing areas for each of several flow components representing different response patterns to rainfall events (e.g., fast, medium, slow).

RDI/I for any specific area can have more or less than 5 components depending on the flow response. For this modeling effort, five RDI/I response components were used, with each component identified by a percentage of the total RDI/I volume and other parameters that reflect the timing of the flow response, as illustrated in **Figure 2-7**. The "fast" component of the hydrograph has the largest impact on the magnitude of the peak wet weather flow response, while the slower components can contribute significantly to the total volume of the RDI/I response. The slowest response component can extend out many days or weeks after the rainfall (alternately, this component could be represented as an increase in GWI). Summing all the component hydrographs for the duration of the rainfall events results in the total RDI/I hydrograph for that area. R values and hydrograph parameters are determined through the process of wet weather model calibration, discussed in **Section 2.5.2** of this Capacity Report, in which actual observed rainfall events are simulated in the hydraulic model, and the resulting model hydrographs are compared to the measured flows at the flow meter locations.

The RDI/I parameters are adjusted as needed to achieve the best match of modeled to monitored flows. The same calibrated parameters are generally applied to all subcatchments within each meter area. Once calibrated, the model RDI/I parameters can be applied to a design storm to simulate wet weather flows for a design event.

FIGURE 2-7: RDI/I HYDROGRAPH COMPONENTS



2.5 Model Calibration

This section discusses the model calibration process and presents the results. Model calibration consists of comparing model-simulated flows to monitored (observed) flows and adjusting model parameters until a reasonably good match is achieved. During calibration, it is not expected that flow meter data matches model-predicted flow data at all times, but modeled flows should reasonably match the flow volumes and peak flows seen in the observed data. Model calibration is achieved first by comparing modeled versus metered flows during a dry weather (non-rainfall) period to achieve an accurate prediction of BWF plus GWI, and then during a wet weather period to estimate the RDI/I response.

The resulting flow calibration graphs of all four (4) flow meters, for the period between January and March 2024, can be found in **Appendix C**.

2.5.1 Dry Weather Calibration

The flow monitoring period did not include any extended dry weather periods. A dry period in mid-March (March 16-March 21) was used to initially compare flow meter data to the model results because the meters showed relatively consistent readings with minimal influence from previous rainfall. The primary focus of the dry weather calibration was to confirm that the calculated average BWF was consistent with the measured flows at the meter location. Another objective of the dry weather calibration was to confirm the diurnal profiles used to represent the hourly variations in BWF.

Table 2-5 summarizes the total estimated average dry weather flow (ADWF) at each flow meter location based on the model calibration and the existing loads (BWF plus GWI) described previously. As indicated in the table**,** the dry weather model calibration resulted in a reasonably good match of modeled to metered flow. However, it should be noted that there are several uncertainties related to this dry weather calibration:

- As the study lacked any extended dry weather periods, there is significant uncertainty in the distinction between GWI rates, unit flow factors, and diurnal patterns used to estimate dry weather flows. It is likely that some GWI is underestimated in **Table 2-5**.
- Relatively small upstream sewersheds result in significant daily and weekly variability in the daily diurnal pattern. Therefore, a typical overall pattern was applied for the entire sewershed.

TABLE 2-5: ESTIMATED NON-RAINFALL FLOW SUMMARY

Location	Total Contributing Area (acre)	GWI (gpd/ acre)	Meter ADWF (mgd)	Model ADWF (mgd)	Difference (mgd)	Difference (%)
PSZ_M21-1	67	0	0.052	0.051	0.001	2%
PSZ_M14-12	69	0	0.043	0.044	-0.001	3%
PSZ_M15-12	108	65	0.033	0.030	0.003	9%[1]
PSZ_M18-4	336[2]	0	0.162	0.156	0.005	3%

[1]This difference between the metered and modeled flow was higher upstream of PSZ_M15-12 as this portion of the system appeared to be more impacted by antecedent rainfall, meaning the metered flows may include RDI/I from previous storm events.

[2]Total contributing area is incremental, representing the contributing area between PSZ_M18-4 and the upstream flow meter locations.

2.5.2 Wet Weather Calibration

During wet weather calibration, the percentage volume of each of the RDI/I components (example pictured in **Figure 2-7**) are adjusted to simulate the volume and timing of RDI/I for monitored storm events in order to best match the overall wet weather hydrograph's shape and magnitude of peak flows. To simulate a rainfall event in the model, observed rainfall data from the temporary rain gauge located on Ely Road North was assigned to all subcatchments in the Penngrove model network. The model-predicted wet weather response, which is based on the assigned rainfall intensity and the RDI/I components, is then compared to observed flows.

The flow monitoring conducted during the period between January 17 and February 11, 2024 was used as the wet weather calibration period for comparing flow meter data to the model results. Rainfall for key events referenced during the wet weather calibration process is summarized in **Table 2-6**.

TABLE 2-6: RAINFALL EVENTS REFERENCED FOR WET WEATHER CALIBRATION

Date of Event	Total 24-Hour Rainfall (in)[1]	Peak 1-Hour Intensity	24-hour Storm Return Period[3]	1-hour Storm Return Period[3]
Jan 21, 2024	0.84	0.37	<1-year	<1-year
Feb 2, 2024	2.00	0.96	<1-year	10-year
Feb 4, 2024	1.64	0.17	<1-year	<1-year

[1]Rainfall totals are from a temporary rain gauge on Ely Road North.
[2]Intensity is reported as an hourly average of 15-minute rainfall data.
[3]Approximate return period based on local NOAA Atlas 14 precipitation statistics and 24-hour rainfall total. The storm return period is defined in **Section 3.1.1**.

During the wet weather calibration process, RDI/I hydrograph parameters were developed for each metered area. **Figure 2-8** presents the wet weather calibration results at the farthest downstream flow meter (PSZ_M18-4), closest to the Penngrove Lift Station. The wet weather calibration graphs for all of the four flow meters for the January through March 2024 period are included in **Appendix C**. The resulting graphs show that a reasonably good match between modeled and metered peak flows and volumes was achieved during all the rainfall events simulated.

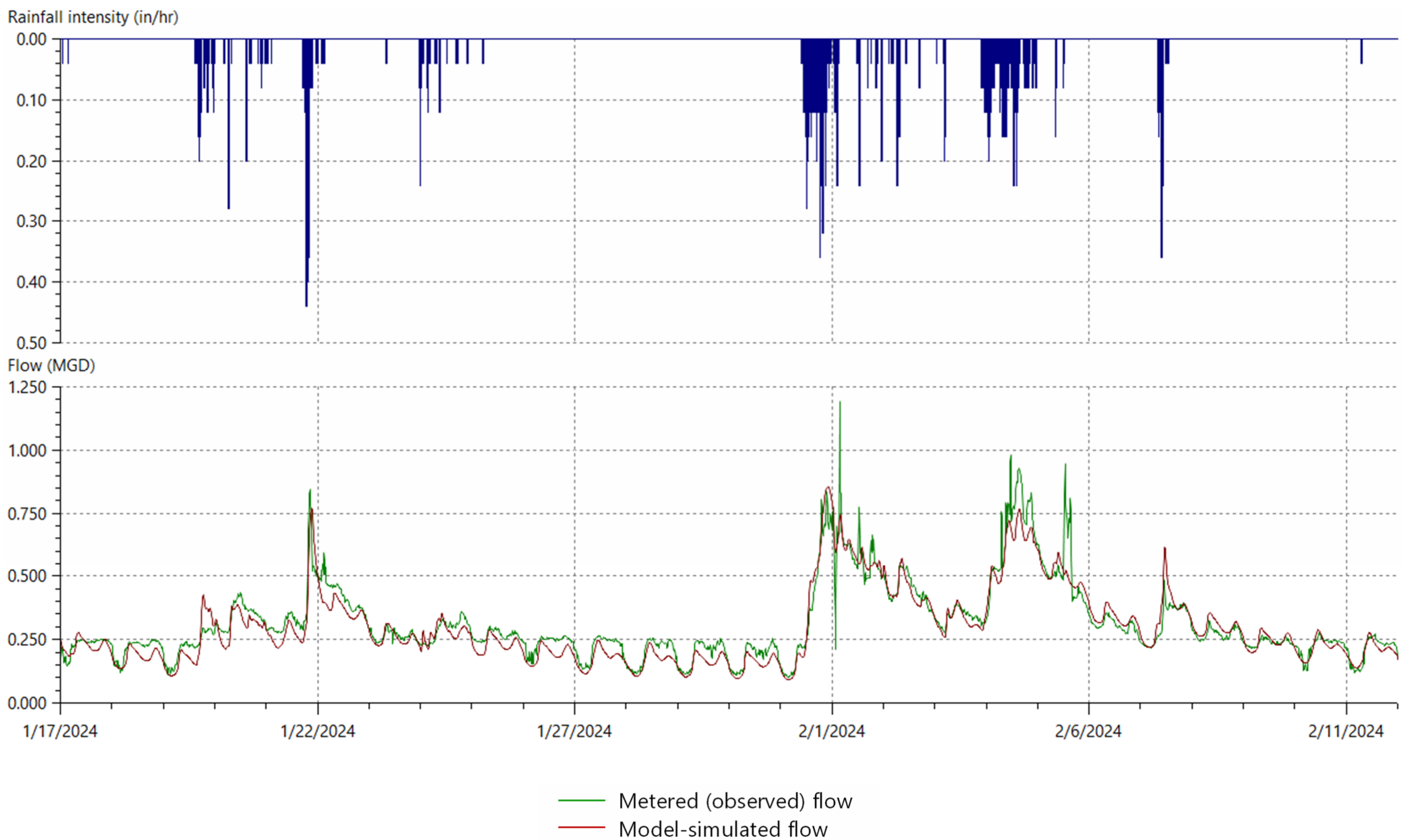
It should be noted that there are several uncertainties associated with the wet weather calibration, including:

- Lichau Creek runs parallel and adjacent to the trunk sewer upstream of the Penngrove lift station, with several manholes located in the creek floodplain. Sonoma Water operations staff have noted that flows escalate significantly when creek levels are elevated. However, while creek levels were elevated during the February 2 and 4, 2024 monitored rainfall events, the data were insufficient to draw a clear distinction between I/I contributions from rainfall versus I/I due to elevated creek levels.

- The observed storms used for calibration had 24-hour rainfall depths with approximate return frequencies of less than one year and were therefore significantly smaller events than the design storm (discussed in **Section 3.1**). However, the Feb 2, 2024 storm did have one hour rainfall intensities with an approximate return frequency of 10 years. The modeling approach to escalate I/I proportional to the size of the storm may be inaccurate for significantly larger storm events.

- Antecedent conditions can significantly impact I/I response. It was noted that the February 4, 2024 event occurred under highly saturated conditions, compared to the February 2, 2024 event, and resulted in a larger response despite lower rainfall rates and depths. To avoid overestimating potential flows, model calibration targeted the February 2, 2024 event, and slightly underestimated flows from the February 4, 2024 event.

FIGURE 2-8: WET WEATHER CALIBRATION FOR JANUARY 2024 – FEBRUARY 2024 (PSZ_M18-4)



3. CAPACITY ANALYSIS

The performance of the Penngrove sewer system and potential need for capacity improvements were evaluated using the calibrated hydraulic model described in **Sections 1** and **2**. This section discusses the performance and design criteria upon which the capacity assessment was based and presents the results of the capacity evaluation.

3.1 Design Event Criteria

Sewer system capacity is assessed with respect to the system's performance under a design flow condition, which includes all components of wastewater flows: BWF, GWI, and RDI/I. Criteria for computing existing BWF, GWI, and RDI/I (developed as part of model calibration) and anticipated future BWF were discussed in the previous sections. However, the peak design flow criteria must also specify the set of conditions – such as the design storm rainfall and when it occurs relative to seasonal GWI and diurnal BWF – that produce the highest peak flows that the sewer system must hydraulically convey to meet or exceed the performance criteria described in **Section 3.2**.

The following subsections discuss the design event used for the Penngrove system capacity evaluation. Key factors needed to define a design event include the storm return period and duration, distribution and amount of rainfall, and storm timing. These factors are discussed in more detail in **Sections 3.1.1 – 3.1.5** and the selected design event for Penngrove is summarized in **Section 3.1.6**.

3.1.1 Storm Return Period

The return period defines the probability that the design rainfall will be exceeded in any given year. For example, a storm with a 5-year return period means that there is a 1 in 5 chance, or 20 percent probability, of exceeding the design rainfall in any given year. The chosen return period reflects the degree of risk an agency will tolerate in terms of sanitary sewer overflows (SSOs) during future storm events. However, choosing a design storm with a very high return period (reflecting a very low risk tolerance) could lead to the identification of a significant number of system capacity deficiencies that would make the cost of improvements prohibitive. Additionally, sizing a system for a very rare event could mean that the system does not function well under typical conditions during much lower flows (due to slow velocities in oversized pipes, or oversized lift stations).

Although there is no regulatory standard for design return periods for wastewater collection systems, most San Francisco Bay Area agencies that have adopted a specific return period have selected return periods of 5 to 20 years. Some of the storms chosen by these agencies are historical rainfall events; others are synthetic storms. As discussed further in the subsequent paragraphs, depending on the type of rainfall distribution chosen, although the total storm rainfall will be the same for storms of the same stated return period and overall duration, shorter durations within each storm may or may not be an equivalent return period. It should also be noted that the return period from a rainfall event can differ from the return period of a resulting peak flow occurring in the collection system due to other factors such as timing of the storm with respect to the normal diurnal wastewater pattern and the antecedent conditions (e.g., groundwater levels, soil saturation, prior rainfall) under which the storm occurs.

For consistency with the sewer capacity assessments for other Sonoma Water districts and sanitation zones, and the City's Sewer Master Plan (currently in progress), Sonoma Water has selected a 10-year return period for this study.

3.1.2 Storm Duration

A storm duration must be specified along with the return period. Most Bay Area agencies use a 24-hour storm, though shorter or longer durations may sometimes be appropriate (e.g., a shorter duration in a very small system with fast response to rainfall or in an area where storm events are typically very brief; a longer duration in a very large system or one with a very slow response to rainfall). Typically, 24-hour duration storms are constructed such that the more intense rainfall occurs during a shorter (e.g., 4- to 6-hour) period.

For consistency with other Sonoma Water sewer capacity studies and the City's Sewer Master Plan (currently in progress), Sonoma Water has selected a 24-hour duration for this study.

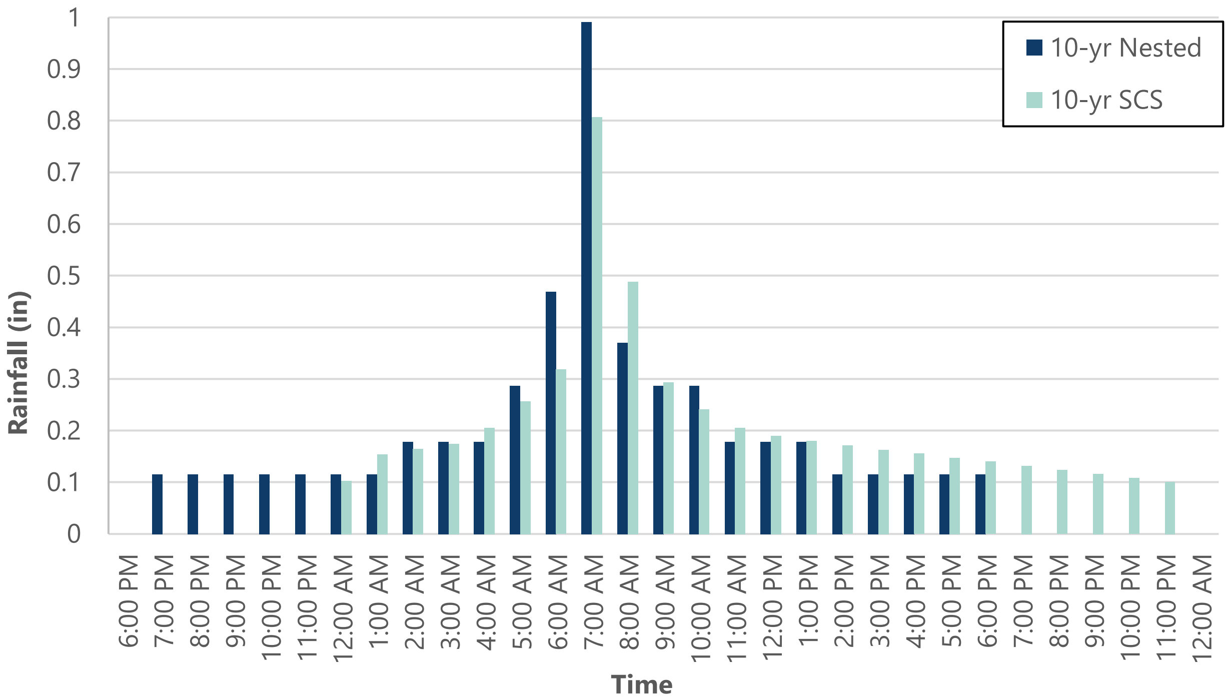
3.1.3 Rainfall Temporal Distribution

The temporal rainfall distribution of a design storm may be a synthetic storm or an actual historical event. A historical event is most beneficial as a design storm when a long enough record of both sewer flow data (for instance at a wastewater treatment plant) and hourly rainfall data exist to identify the types of rainfall events that tend to generate the highest flows in the sewer system. In general, the Penngrove system does not have sufficient historical hourly or sub-hourly flow data available to identify a potential historical rainfall event that could be used as a design event.

Commonly used synthetic storm distributions include nested storms (which include the rainfall amounts for the storm return period for all durations within the total storm duration) or the 24-hour "SCS Type IA" storm distribution, a general distribution characteristic of northern California coastal areas, as defined by the USDA guidance document Urban Hydrology for Small Watersheds TR-55 (June 1986). Nested storms are relatively conservative, as the peak intensities for all durations will not typically occur within the same storm event. The peak rainfall intensity for a nested synthetic storm is higher than the peak intensity in the SCS Type IA synthetic distribution, as illustrated in **Figure 3-1**. Note that for either distribution, the timing of the peak rainfall can be adjusted to occur at any hour of the day, depending on the desired timing of the storm with respect to the diurnal BWF (see discussion on storm timing in **Section 3.1.5**).

For consistency with other Sonoma Water sewer capacity studies and the City's Sewer Master Plan (currently in progress), Sonoma Water selected the SCS Type IA distribution for this study.

FIGURE 3-1: COMPARISON OF POTENTIAL DESIGN STORMS[1]



[1]Note: In this graphic, the hour timing represents a model simulation in which the peak RDI/I is assumed to coincide with the peak BWF in a typical residential community (see discussion in **Section 3.1.5**). Model simulation begins the day before the rainfall event.

3.1.4 Rainfall Amount

Synthetic design storms are typically based on rainfall depth-duration-frequency (DDF) or intensity-duration-frequency (IDF) statistics that have been compiled for a local area. These statistics give the rainfall depths or intensities for various return periods (e.g., 2-year, 5-year, etc.) and durations (e.g., 1-hour, 2-hour, etc.).

The rainfall frequency statistics in Sonoma Water's current *Flood Management Design Manual* (March 2020) are based on precipitation frequency statistics from NOAA Atlas 14[1]. **Table 3-1** summarizes the 10-year, 24-hour rainfall for the Penngrove service area.

TABLE 3-1: DESIGN STORM CHARACTERISTICS

Frequency	Distribution	Duration (hours)	Total Rainfall (in)[1]	Peak Hour Intensity (in/hr)[1]
10-yr, 24-hr	SCS Type IA	24	5.14	0.81

[1]Rainfall volume based on NOAA Atlas 14 precipitation frequency estimates at the PENNGROVE FS station. For comparison, the 10-year 24-hour rainfall volume at the Corona Road station in Petaluma is 4.77 inches.

[1] NOAA Atlas 14 precipitation frequency estimates available at: https://hdsc.nws.noaa.gov/pfds/pfds_map_cont.html.

3.1.5 Storm Timing

The timing of the rainfall distribution compared to the base wastewater flow (dry weather) diurnal profile should also be considered. For example, rainfall can be timed to generate RDI/I that peaks roughly at the same time as the dry weather flow diurnal peak (referred to as "peak-on-peak" timing). This consideration is most important in systems where flow from RDI/I is relatively small compared to BWF and in systems where the response to rainfall occurs relatively quickly (over hours instead of days). For consistency with all of Sonoma Water's sewer capacity studies, it was recommended that the design storm be timed to achieve "peak-on-peak" flow conditions. The City's Sewer Master Plan (currently in progress) also assumes "peak-on-peak" flow conditions.

3.1.6 Selected Design Event for Penngrove

In summary and as described in **Sections 3.1.1 – 3.1.5**, the following design event parameters were adopted for the Penngrove Capacity Assessment:

TABLE 3-2: DESIGN EVENT PARAMETERS

Storm Return Period	10 years
Storm Duration	24 hours
Rainfall Temporal Distribution	Synthetic, SCS Type IA
Rainfall Amount	Per NOAA Atlas 14
Storm Timing	Peak-on-peak

3.1.7 Future Conditions

It is not known if I/I will increase over time due to further deterioration of the sewer system, and/or whether an agency's future sewer rehabilitation efforts will be sufficient to prevent such increases. However, studies to assess such changes in other systems throughout the country have not produced conclusive results that such increases will occur. Furthermore, new sewer construction will primarily use watertight pipe materials that will minimize new sources of I/I. Therefore, this capacity assessment assumes that I/I in the existing system will not increase in the future and that any increases due to new sewer construction to serve future development will be minimal or offset by other I/I reduction efforts. Future rehabilitation efforts by the PSZ may demonstrate reduced I/I (through future flow monitoring or lift station records), at which time this assumption could be re-evaluated.

3.2 Capacity Deficiency Criteria

Capacity deficiency or "performance" criteria are used to determine if the capacity of an existing sewer facility is exceeded to the extent that a capacity relief project is needed. These criteria are sometimes called "trigger" criteria, in that they trigger the need for a capacity relief project, unlike design criteria that are applied to determine the size of a new facility. The difference between capacity deficiency criteria and design criteria is that the former reflects the fact that some existing facilities can continue to provide adequate, if not optimal, conveyance capacity, while the latter can inform if new facilities should be designed to a higher standard.

It is important that the capacity deficiency criteria be coordinated with the peak design flow criteria. For example, if the peak design flow were to be based solely on peak dry weather flow (PDWF), the capacity deficiency criteria would need to be conservative (e.g., require pipes to flow less than full to allow capacity for I/I). On the other hand, if the peak design flow includes I/I from an infrequent storm event, it may be appropriate to allow the sewers to flow surcharged to some extent, because the peak flows will be infrequent and relatively brief in duration. (Note: Surcharge means that the flow level is higher than the top of the pipe between manholes, indicating the pipe is flowing under pressure.)

Consistent with Sonoma Water's other sewer capacity studies, a capacity deficiency was identified for Penngrove's gravity sewers under the following conditions:

- Any modeled surcharging under PDWF; and/or
- Any modeled overflows or surcharge reaching within 5 feet of manhole rims under design storm PWWF.

Note that the Manning's 'n' values discussed in **Section 2.2.1** were assumed for the capacity evaluation of existing sewers.

For planning level capacity assessments, lift stations are typically considered capacity deficient if the peak design flow exceeds the station's estimated firm capacity (i.e., pumping capacity with largest pump out of service). Force mains are typically considered to be deficient if velocity under the peak design flow exceeds 10 feet per second (fps). These capacity deficiency criteria were applied to both Penngrove's and the City's systems.

3.3 Design Criteria for New Sewer Facilities

Sonoma Water's *Design and Construction Standards for Sanitation Facilities* (approved February 2009, amended November 2020) specify criteria for hydraulic design of sewer system mains and associated facilities. Although there are exceptions for side sewers and sewers serving cul-de-sacs, in general, Sonoma Water's standards include the following:

- Manning's 'n' of 0.013 for collector sewers;
- Minimum pipe size of 8 inches;
- Slope to provide a minimum velocity of 2 fps at full pipe for gravity sewers;
- Force main minimum and maximum velocities of 3 and 8 fps, respectively (3 fps would be considered a reasonable minimum velocity to keep solids from settling out if it can be reasonably achieved, and 8 fps would prevent excessive head losses in the force main); and
- Where possible, 4.5 feet minimum cover for sewers in streets or 3.5 feet for sewers in easements.

The standards presented above have been adopted for this Capacity Report, with the following clarifications:

- Maximum allowable depth-to-diameter ratio (d/D) of 1.0 (full pipe) under modeled future PWWF (design storm conditions), assuming Manning's 'n' per Sonoma Water standards and no downstream flow restrictions (i.e., pipe would be sized to convey the modeled peak flow, not to store backwater from downstream restrictions);

- For new pipes (new alignment);
    - o Minimum slope to achieve a cleaning velocity of 2 fps under PDWF, if feasible; and
- Where a need is identified to upsize an existing pipe, the existing slope and pipe invert elevations are assumed to remain.

3.4 Capacity Analysis Results

Based on the criteria described in **Sections 3.1 – 3.2**, the calibrated hydraulic model was run for existing and future development conditions to identify areas of the system that would fail to meet the specified performance criteria under PDWF and design storm PWWF based on the model results. Capacity recommendations to address potential deficiencies are presented in **Section 4** and were developed in accordance with the design criteria for new sewer facilities defined in **Section 3.3**.

3.4.1 Lift Station Capacity Results

Lift station capacity was analyzed for the two existing discharge locations of the existing force main: 1) Corona Road and 2) the Wilmington Lift Station. Currently, the force main can discharge either to the City's gravity sewer on Corona Road, which drains to the City's Wilmington Lift Station, or directly to the manhole upstream of Wilmington Lift Station. **Figure 3-2** shows the pump and system curves for the Penngrove Lift Station and these two different force main discharge locations. **Table 3-3** shows the estimated firm and total pump station capacities for these scenarios as well as the estimated force main velocities. **Appendix D** contains the assumptions used to create the system curves, and the manufacturer pump curves which are based on recent impeller upgrades (installed in 2023) made to the station's pumps (Model NP 3153 MT 3, Impeller #433).

Based on the system curve, the lift station has a firm capacity limitation between 0.26 and 0.42 mgd, depending on the discharge location, and the pumps appear to operate far left on their performance curve, indicating higher total system head. It should be noted that the system curve is based on an assumed Hazen-Williams C factor (representing force main roughness) of 110. While this system curve, in combination with the pump curves, appears to be a good match to observed lift station capacity, it is possible that the force main has a higher C factor (representing a smoother pipe) and the pumps are underperforming their pump curves. Pressure data downstream of the pumps would be needed to confirm the system curve.

Model results indicate predicted overflows upstream of the lift station during the design event. Furthermore, SSOs would likely have occurred under past wet weather events if Sonoma Water staff had not used trucks to convey high flows. Given these results, it is evident that the Penngrove Lift Station does not have sufficient capacity to convey the predicted existing or future design storm PWWF.

No capacity deficiencies for the lift station and force main were identified for either existing or future PDWF conditions (future PDWF is projected to be approximately 0.26 mgd compared to 0.19 mgd under existing conditions).

FIGURE 3-2: PENNGROVE LIFT STATION PUMP AND SYSTEM CURVES

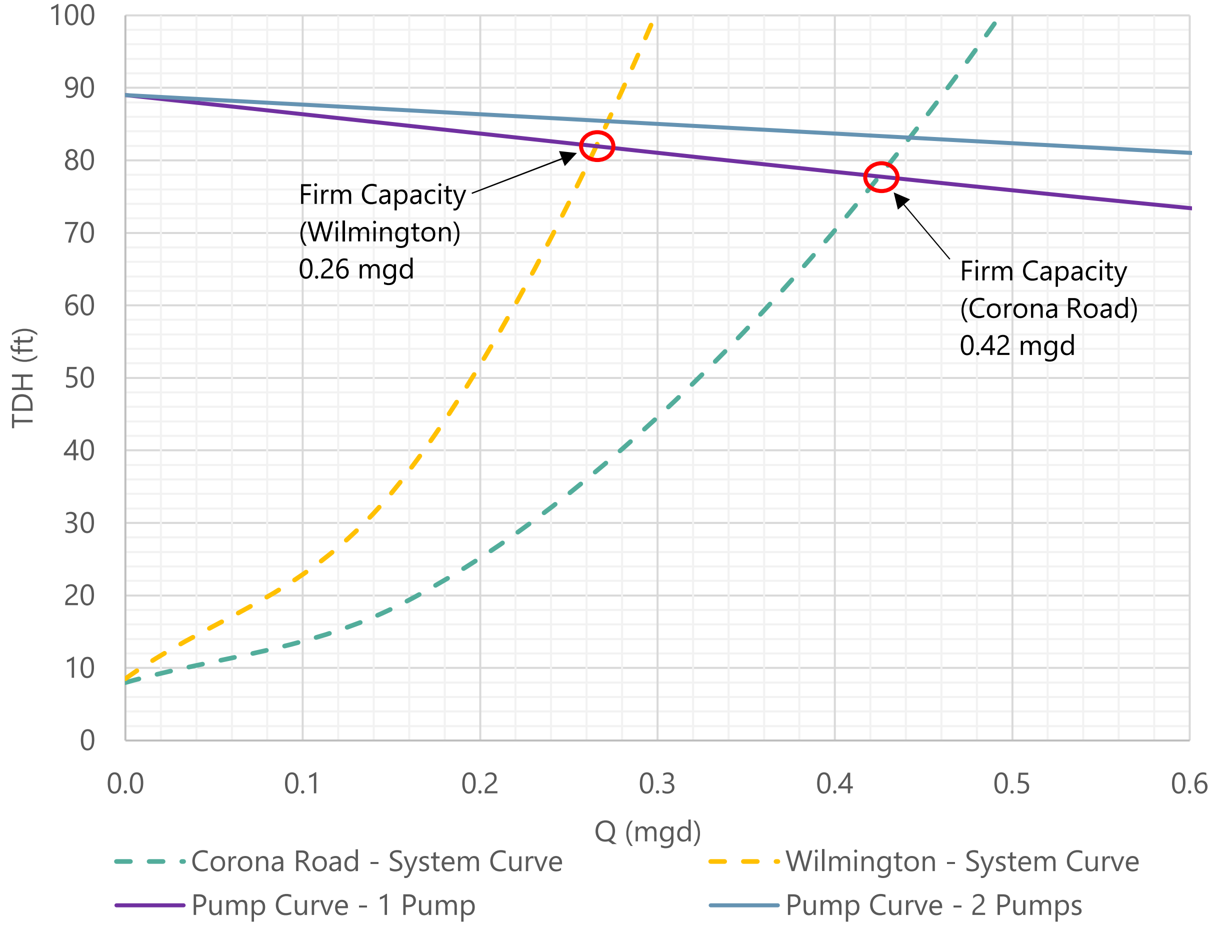


TABLE 3-3: PENNGROVE ESTIMATED LIFT STATION CAPACITIES AND FORCE MAIN VELOCITIES

Capacity Type	Discharge Location #1: Corona Road		Discharge Location #2: Wilmington Lift Station	
	Pump Capacity (mgd)	Force Main Velocity at PS Capacity (fps)	Pump Capacity (mgd)	Force Main Velocity at PS Capacity (fps)
Firm Capacity	0.42	3.4	0.26	2.1
Total Capacity	0.44	3.5	0.27	2.2

Preliminary model runs with the design storm indicate potential peak wet weather flows (PWWF) of up to 1.57 mgd under existing development conditions, or up to 1.67 mgd under future conditions, as summarized in **Table 3-4**. For existing conditions, the modeled flowrate predicts more than a 15-fold increase in flow for the design PWWF compared to the BWF (0.093mgd), indicating excessively high wet weather I/I contributions to the existing system. This not only suggests that opportunities for I/I reduction should be further investigated, but also supports the perspective that the accuracy of these preliminary modeled flowrates is subject to the wet weather calibration uncertainties described in Section 2.5.2. and warrants further validation pending the collection of additional data for larger storm events.

Nonetheless, at these flowrates, the existing 6-inch force main velocities would exceed 12 fps; substantially higher than the deficiency criteria of 10 fps. Sonoma Water staff have noted that pressure just upstream of the Corona Road discharge location reads at zero psi while the pumps are operating, indicating that the force main contributes significant headloss and is likely a significant contributor to overall lift station capacity limitations. As noted previously, there are significant uncertainties in the potential design storm flows. Therefore, while Sonoma Water should replace the force main in the near term, any planning or design for upsizing of the force main should evaluate the pumping system in tandem and determine whether additional mechanical pumping or electrical improvements are also needed.

TABLE 3-4: PENNGROVE LIFT STATION POTENTIAL PEAK FLOWS AND FORCE MAIN VELOCITIES

Model Scenario	Peak Flow at Penngrove Lift Station (mgd)	Existing Force Main Velocity at Peak Flow (fps)
Existing Loads	1.57	12.4
Future Loads	1.67	13.2

3.4.2 Gravity Sewer Capacity Results

No capacity deficiencies in the gravity sewer system were identified for dry weather conditions. Preliminary model runs, assuming the lift station's firm capacity, have been performed to identify potential capacity deficiencies in the gravity system. The location of model-predicted surcharged sewers during design storm PWWF conditions with existing land uses are shown in **Figure 3-3**. The figure identifies pipes that are predicted to surcharge due to a throttle condition or a backwater condition due to a downstream throttle. Throttle conditions indicate that the full flow capacity of the pipe is predicted to be less than the model-predicted peak flow, resulting in, or increasing, backup surcharge. It should also be noted that the location(s) of model-predicted surcharge may not reflect actual physical conditions (e.g., root intrusion or debris) that are not reflected in the model, or system storage that is available in the smaller diameter, unmodeled pipes.

Figure 3-4 presents a hydraulic profile of the trunk sewer upstream of Penngrove Lift Station and along the middle branch sewer showing the model results under the design storm PWWF for existing development, and assuming the lift station is operating at firm capacity (one pump on, one pump off). Future development conditions resulted in incrementally increased modeled overflow volumes and surcharge, but did not result in any new modeled overflow locations or additional capacity deficient sewers. Additional hydraulic profiles for the west, middle, and east branch sewers are presented in **Appendix F**, assuming the lift station is replaced with a free outfall (i.e., capacity limitations are relieved such that the wet well level stays below the pipe invert) during design storm PWWF conditions.

In addition to the pipe surcharge conditions, **Figure 3-3** also shows the location of manholes that have model-predicted overflows during the design storm PWWF condition. As noted above, predicted surcharge in a particular pipe does not necessarily indicate a capacity deficiency at that particular location, as flows can back up due to a downstream capacity deficiency and cause extensive surcharging or even overflows upstream due to backwater effects. However, relieving upstream deficiencies can also create additional or more severe capacity deficiencies downstream of the relieved pipe, and therefore these downstream areas would also require relief.

It should be noted that Sonoma Water has no record of any capacity-related wet weather overflows in the Penngrove collection system while the lift station is operational and trucking excess flows. Due to the calibration limitations discussed in **Section 2.5.2**, there is significant uncertainty in the potential flows and associated surcharge in the gravity sewer during design storm conditions. Further study is recommended prior to implementing any significant capacity improvement projects.

Figure 3-3: Model Results, Design Storm PWWF, Existing Land Use

Penngrove Sewer

Capacity Evaluation Report

Penngrove Sanitation Zone

Legend

Modeled Sewers
- Backup Surcharge
- Throttle Condition
- No Surcharge
- Force Main
- Unmodeled Sewers
- Parcels
- Penngrove Lift Station
- Modeled Manholes
- Manholes with Predicted Overflow
- Criteria Exceeded MHs (<5' Freeboard)
- PSZ Service Area
- Streams

N

0 230 460 920 Feet

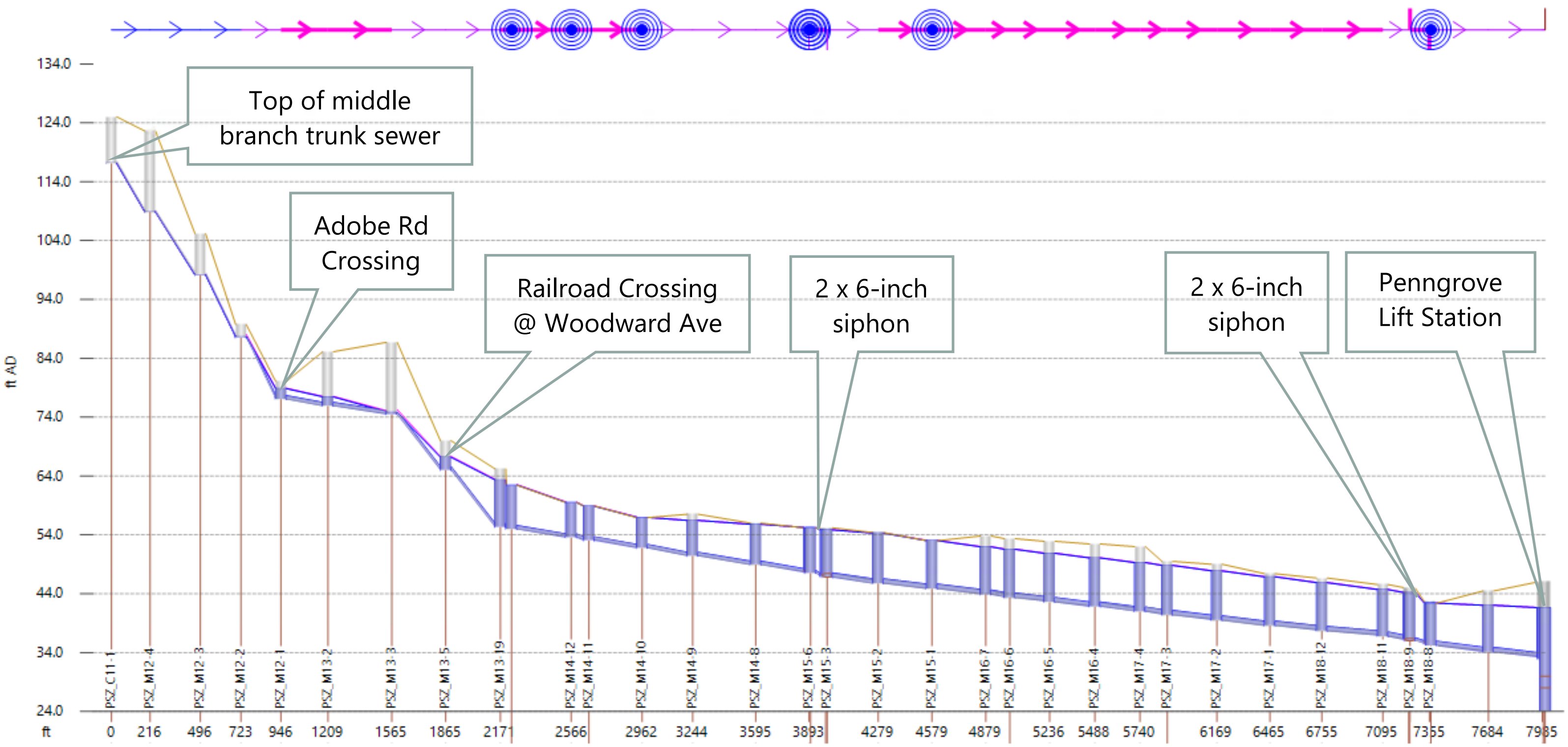
Woodard & Curran

Sonoma Water

Project #: 0012206.00

Map Created: April 2026

FIGURE 3-4: MODELED PROFILE UNDER DESIGN STORM CONDITIONS, EXISTING LAND USE, LIFT STATION FIRM CAPACITY[1]



[1]The hydraulic profile assumes the lift station is operating at firm capacity, with one pump running and one pump offline, resulting in backup from the Penngrove LS wet well. While trucks are operating to convey flow, the actual HGL would be lower.

LEGEND

- No Surcharge
- Backwater Surcharge
- Throttle Surcharge
- Model-predicted Overflow

4. SYSTEM CAPACITY IMPROVEMENTS

This section summarizes the capacity improvements that may be needed for Penngrove's force main, lift station, and gravity sewers. These include near-term mitigation projects to address current wet weather capacity limitations. It is understood that the scope of these near-term projects may be adjusted as adequate data becomes available to better inform and define long-term capacity needs and next steps. These recommended interim projects are detailed below and summarized later in **Table 4-1** and **Figure 4-1**, along with recommendations for long-term capital projects.

4.1 Force Main Capacity Improvements

As discussed in **Section 3.4.1**, the model results indicate that there is a significant capacity deficiency at the Penngrove Lift Station. Head losses in the force main are a significant contributor to current lift station capacity limitations, and under peak flows, velocities would exceed 15 fps (under future development conditions after improvements).

Based on the model-predicted flows, a new 10-inch diameter force main would satisfy the hydraulic design criteria identified in **Section 3.3** to transport the estimated PWWF, resulting in a peak velocity of 5.7 fps at a flow of 1.67 mgd. Under dry weather conditions, velocities in the force main could be somewhat lower depending on logic controls; pump selection and logic controls would need to consider minimum velocities to allow for sufficient sediment cleansing.

Several different force main discharge locations and alignments could be considered. These locations may have different impacts on Petaluma's sewer collection system and should be coordinated with the City. Locations could include either discharging near the current discharge location, or utilizing existing City infrastructure along North McDowell Boulevard for a shorter alignment. The choice of force main discharge location and alignment will affect both the cost of the force main itself, as well as the pump selection, as discussed in the next section.

Sonoma Water is planning to initiate an alternatives analysis in coordination with the City this year to identify the preferred alignment. Once the alternatives analysis is complete, Sonoma Water will move forward with design.

4.2 Lift Station Capacity Improvements

Despite recent upgrades to the Penngrove Lift Station pump impellers, Woodard & Curran's investigation indicated that the pumps are operating far left on their curves, indicating that the pumps may not be designed for their current operating point and are operating inefficiently. A follow-up study is recommended to determine the appropriate pump size that could be installed without necessitating upgrades to the wet well, Pacific Gas & Electric (PG&E) service, or other significant lift station equipment. Pump selection should consider pump operation with the new force main in service.

Sonoma Water is planning to initiate this study in conjunction with the force main alternatives analysis. New pumps may be installed at approximately the same time as the new force main if deemed necessary. The existing pumps may be adequate with modified force main sizing, and this should be further confirmed following completion of the wet weather recalibration described in **Section 4.3** below.

4.3 Gravity Sewer System Improvements

Based on the current wet weather calibration, the model predicts apparent capacity deficiencies in the upstream gravity sewers that could cause surcharging above the pipe crown within five (5) feet of the ground surface, or result in a manhole overflowing during design storm PWWF. The wet weather calibration indicates potential throttle conditions and resulting surcharge during the predicted design storm PWWF.

Considering that significant wet weather response has been observed in the system, even during smaller rainfall events, it is possible that substantial I/I is entering the system from the trunk sewer adjacent to the creek and/or from additional upstream areas of the system. Accordingly, a comprehensive I/I study should be considered prior to implementing any gravity sewer improvements, which is explained further in **Section 4.4**. Sonoma Water has already initiated an I/I investigation program to assess potential I/I sources in the trunk sewer adjacent to the creek and the middle branch of the upstream sewershed (which had the highest I/I response during the flow monitoring program).

To mitigate risk and evaluate potential surcharging, a smart cover depth monitoring device should be considered in the vicinity of Manhole M15-6 (see **Figure 4-1**).

Following the I/I study and implementation of I/I reduction measures, the current wet weather model calibration should be revisited to identify segments of pipe that would still require upsizing to relieve throttle conditions. Substantial reduction in I/I could reduce or eliminate some of the gravity sewer capacity limitations that trigger the need for improvements. Implementation of the force main and lift station pumping improvements (described in **Sections 4.1** and **4.2**) would also improve data collection by reducing or eliminating trucking during large events, which could help reduce some of the uncertainty in quantifying their potential peak flows.

4.4 I/I Investigations and System Rehabilitation

Sonoma Water should continue I/I investigations, using source detection methods described in **Appendix E**. Specific methods are discussed below:

- Investigations should include inspections of manholes, targeting those within the flooding extents of Lichau Creek, and rehabilitating those where significant I/I is found.
    - Note: Many manholes were sealed with mortar in 2008, and infiltration was subsequently observed within these manholes. They may therefore need further attention. A spin-cast process for manhole rehabilitation could be considered, which could cost \$40-\$50K per manhole according to Sonoma Water staff.
- Consider implementing flow monitoring or depth monitoring at various locations to track I/I contributions in the collection system. Sonoma Water's available smart covers could also be utilized and moved around to known problem areas/manholes to identify spikes in depth during storm events. Note that smart cover data is not suitable for quantifying flows for model recalibration, but can be used to qualitatively identify locations of high I/I.
- Smoke testing could be considered, but Sonoma Water has had limited success in past efforts. Smoke testing would be intended primarily to identify direct connections from the creek or storm drain systems, or direct inflow connections on private property. Smaller inflow sources, such as connected roof leaders or broken cleanout caps, are relatively common findings. Individually, these

sources typically contribute minor amounts to overall I/I; however, their cumulative effect can be notable when they occur in large numbers.

- Investigation of additional sources of I/I through closed circuit television (CCTV) inspections, including laterals. Sonoma Water has previously performed extensive CCTV inspections in the Penngrove system and found the sewers to be in reasonably good shape. A comprehensive CCTV program should consider laterals, which can be a significant contributor to I/I. Note that it may be easier to identify I/I sources if CCTV is performed during wet weather periods.

Sonoma Water has already initiated a depth monitoring program with RH Borden for the 2025/2026 wet weather season. Next steps should be considered based on the results of that program.

4.5 Summary of Potential Improvements

Potential near- and long-term improvements for the Penngrove system are summarized below and in **Table 4-1.**

As described in **Section 4.3**, there are several interim actions that Sonoma Water could undertake to identify and mitigate sources of I/I along the main trunk sewer and middle branch gravity sewer. These include manhole investigations and subsequent rehabilitation, as well as gravity sewer investigations (such as CCTV).

Long term, Sonoma Water should continue to monitor the sewer system for sources of I/I and implement upsizing projects, if needed, to relieve surcharged sections of the sewer main. Note that if significant I/I reduction is achieved, then it may be possible to reduce or eliminate the scale of gravity sewer improvements needed. The force main downstream of the Penngrove Lift Station should also be upsized to accommodate future flows and eliminate the need for trucking, and pumps replaced to improve performance with the new system curve.

TABLE 4-1: PENNGROVE POTENTIAL CAPACITY IMPROVEMENTS AND RECOMMENDED STUDIES

Schedule	Improvement	Status	Notes
Near-Term	I/I Investigations	Underway	Investigations should include inspections of all manholes in the creek floodplain and rehabilitating those with significant I/I. Use of smart covers to identify spikes in depth during storm events, smoke testing, and CCTV investigations are other viable methods to locate sources of I/I.
	Perform an Alternatives Analysis and Install New Force Main	Underway	Assess potential force main alignment alternatives and select a preferred alternative. Construct replacement force main.
	Perform Pump Study and Install Larger Pumps, as needed.	Planned	Perform a pump study to evaluate station wet well configuration and electrical service to determine size of new pumps that can be installed without significant lift station modifications. Based on results of pump study and force main alternatives analysis, purchase and install new pumps, as needed.
Long-Term	Collect Additional Data and Recalibrate Model	Planned	After completing or concurrent with the above near-term improvements, collect additional flow monitoring data and recalibrate the model to predict potential peak flows more accurately.
	Identify and Implement Additional Upgrade Needs	Planned	Based on the findings of the model recalibration, identify additional upgrade needs (for both the pump station and the upstream gravity sewers), and implement any additional capacity solutions required.

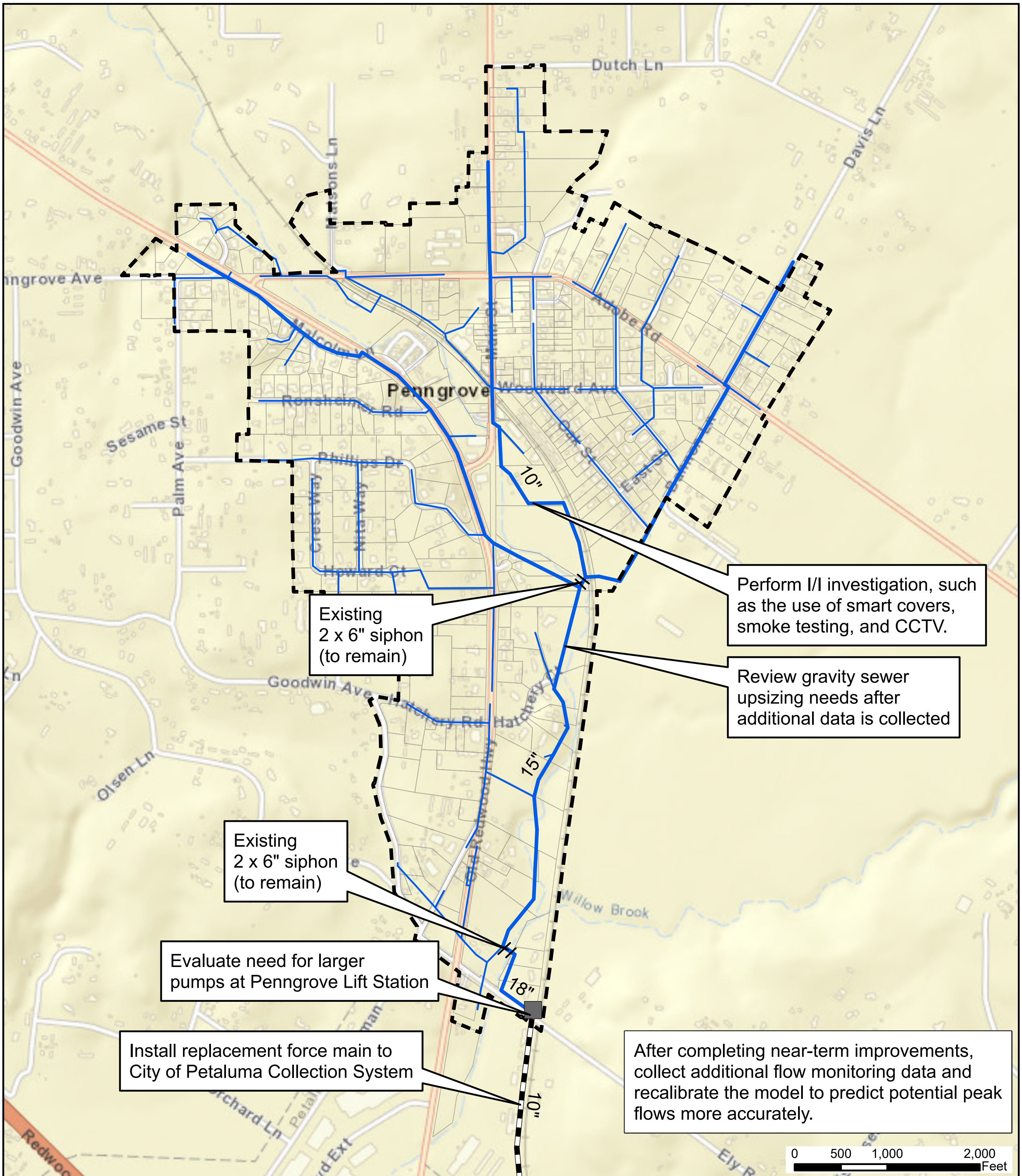


Figure 4-1
Capacity Improvements and Recommended Studies

Penngrove Sewer Capacity Evaluation Report

Penngrove Sanitation Zone

Legend

- Penngrove Lift Station
- Force Main
- Modeled Trunk Sewers
- Unmodeled Sewers
- Parcels
- PSZ Service Area

N

Woodard & Curran | Sonoma Water

Project #: 0012206.00
Map created: April 2026

Figure Exported: 5/18/2026 By: ELWilson Using: \\woodardcurran.net\shared\Projects\CA SCWA\0012206.00 Penngrove Sewer Capacity Eval\wip\GIS\ArcGIS Pro\Penngrove Sewer Model 2024.aprx

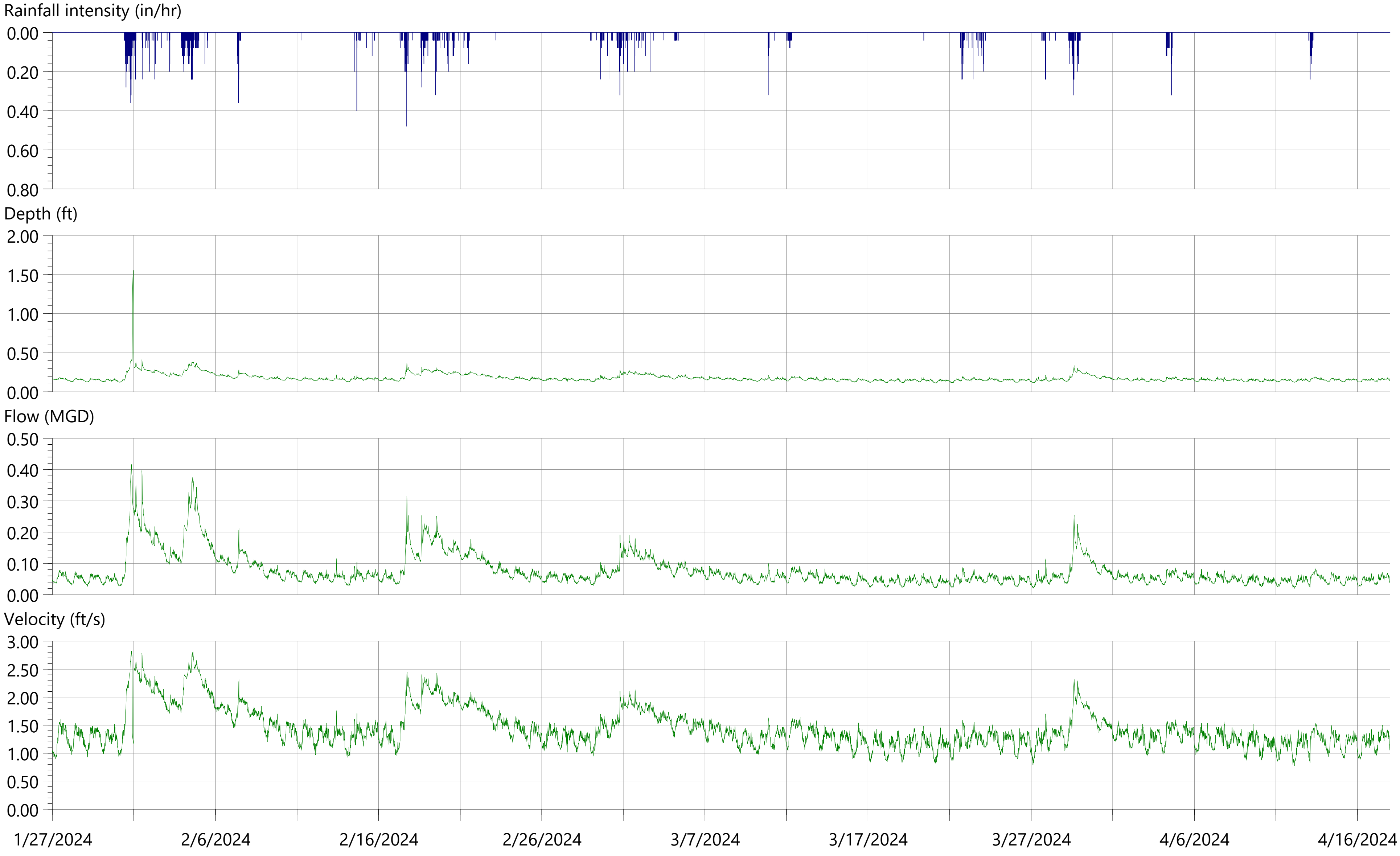
APPENDIX A: FLOW MONITORING DATA

Observed / Predicted Report Produced by cvanlienden (4/9/2025 4:21:11 PM) Page 1 of 4
Flow survey: >2022 Penngrove Model>03 DATA>A Flow Data>Flow Meter Data 2024 (6/7/2024 4:13:32 PM)
Sim: >2022 Penngrove Model>05 Runs>Calibration>Wet Weather Flow>WWF December 2023-April 2024>Free Outfall at PS ADS Rain Gauge + County RG at Corona Rd (Dec 2023 - Ap...

Flow Survey Location (Obs.) PENN_M15-12, Rainfall Profile: PENN_RG01

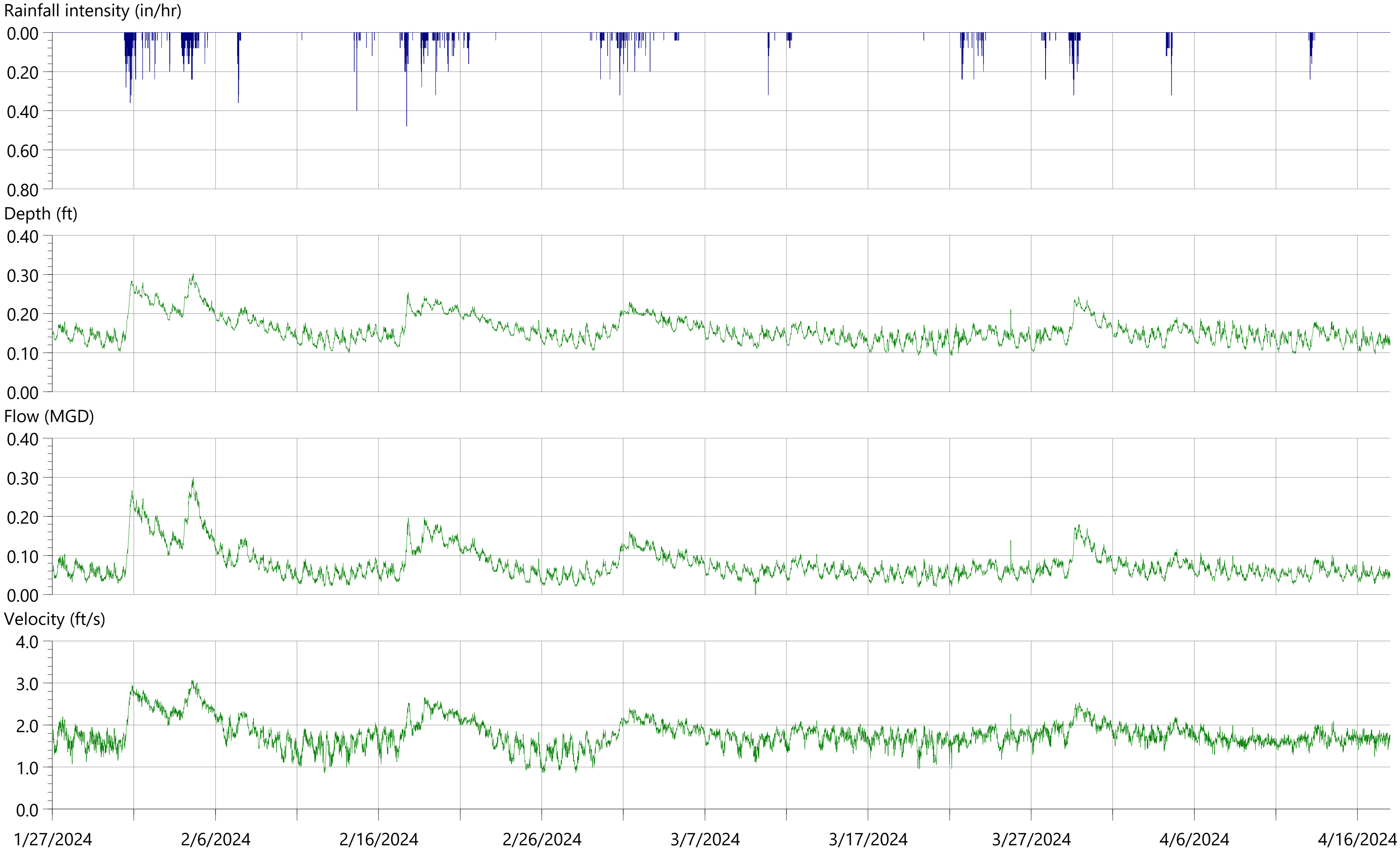
	Rainfall			Depth		Flow			Velocity	
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (ft)	Max (ft)	Min (MGD)	Max (MGD)	Volume (US Mgal)	Min (ft/s)	Max (ft/s)
Rain	12.430	0.480	0.006							
Observed				0.000	0.870	0.000	0.192	3.474	0.100	2.724

Observed / Predicted Report Produced by cvanlienden (4/9/2025 4:21:11 PM) Page 2 of 4
Flow survey: >2022 Penngrove Model>03 DATA>A Flow Data>Flow Meter Data 2024 (6/7/2024 4:13:32 PM)
Sim: >2022 Penngrove Model>05 Runs>Calibration>Wet Weather Flow>WWF December 2023-April 2024>Free Outfall at PS ADS Rain Gauge + County RG at Corona Rd (Dec 2023 - Ap...
Flow Survey Location (Obs.) PENN_M14-12, Rainfall Profile: PENN_RG01



	Rainfall			Depth		Flow			Velocity	
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (ft)	Max (ft)	Min (MGD)	Max (MGD)	Volume (US Mgal)	Min (ft/s)	Max (ft/s)
Rain	12.430	0.480	0.006							
Observed				0.114	1.553	0.021	0.418	6.214	0.774	2.827

Flow survey: >2022 Penngrove Model>03 DATA>A Flow Data>Flow Meter Data 2024 (6/7/2024 4:13:32 PM)
Sim: >2022 Penngrove Model>05 Runs>Calibration>Wet Weather Flow>WWF December 2023-April 2024>Free Outfall at PS ADS Rain Gauge + County RG at Corona Rd (Dec 2023 - Ap...
Flow Survey Location (Obs.) PENN_M21-1, Rainfall Profile: PENN_RG01



	Rainfall			Depth		Flow			Velocity	
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (ft)	Max (ft)	Min (MGD)	Max (MGD)	Volume (US Mgal)	Min (ft/s)	Max (ft/s)
Rain	12.430	0.480	0.006							
Observed				0.092	0.303	0.000	0.300	6.361	0.857	3.056

Observed / Predicted Report Produced by cvanlienden (4/9/2025 4:21:11 PM) Page 4 of 4

Flow survey: >2022 Penngrove Model>03 DATA>A Flow Data>Flow Meter Data 2024 (6/7/2024 4:13:32 PM)

Sim: >2022 Penngrove Model>05 Runs>Calibration>Wet Weather Flow>WWF December 2023-April 2024>Free Outfall at PS ADS Rain Gauge + County RG at Corona Rd (Dec 2023 - Ap...

Flow Survey Location (Obs.) PENN_M18-4, Rainfall Profile: PENN_RG01

	Rainfall			Depth		Flow			Velocity	
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min (ft)	Max (ft)	Min (MGD)	Max (MGD)	Volume (US Mgal)	Min (ft/s)	Max (ft/s)
Rain	12.430	0.480	0.006							
Observed				0.123	7.438	0.084	1.194	19.676	0.597	5.422

APPENDIX B: FUTURE DEVELOPMENT PARCELS

Appendix B – Future Development Parcels

Housing Element Sites (August 2023)

Planning Area	Map ID	APN	Type (Vacant, Nonvacant or Rezone)	Land Use	Zoning	Acres	Existing Uses	Density	Potential New Units	Zone	Zone Description	Maximum Lot Area Coverage	Res Flow Factor (gpd)	AFF (gpd)	AFF (mgd)	Res ESD	Non Res Flow Factor (gpd/SF)	Non Res SF	Trade Flow (gpd)	Trade Flow (mgd)	Non Res ESD	Existing ESD?	Total ESD
7 - Rohnert Park/ Cotati	PEN-10	047-173-016	Nonvacant	UR 6	R3 8	1.36	Residential (1)	6.0	7	R3	High Density Residential District		136	952	0.00095	5.6	0.13	0	0	0	0	0	5.6
7 - Rohnert Park/ Cotati	PEN-1	047-174-009	Rezone	LC	C2 WH	0.05	Single Story Store	16.0	0	C2	Retail Business and Service District	50%	136	0	0.000	0.0	0.13	1,089	141.6	0.0001	0.9	0	0.9
7 - Rohnert Park/ Cotati	PEN-2	047-152-020	Rezone	UR 20	R3 20	1.01	Vacant	20.0	20	R3	High Density Residential District		136	2720	0.003	16.0	0.13	0	0	0	0.0	0.0	16.0
7 - Rohnert Park/ Cotati	PEN-3	047-174-008	Rezone	LC	C2 WH	0.02	Single Story Store	16.0	2	C2	Retail Business and Service District	50%	136	272	0.000	1.6	0.13	348	45.3	0.0000	0.3	3.0	4.6
7 - Rohnert Park/ Cotati	PEN-4	047-152-019	Rezone	UR 20	R3 20	1.72	Vacant	20.0	34	R3	High Density Residential District		136	4624	0.005	27.2	0.13	0	0	0	0.0	0.0	27.2
7 - Rohnert Park/ Cotati	PEN-6	047-091-013	Rezone	UR 20	R3 20	1.95	Residential	20.0	37	R3	High Density Residential District		136	5032	0.005	29.6	0.13	0	0	0	0.0	0.0	29.6
7 - Rohnert Park/ Cotati	PEN-7	047-153-004	Rezone	UR 20	R3 20	5.00	Residential	20.0	99	R3	High Density Residential District		136	13464	0.013	79.2	0.13	0	0	0	0.0	0.0	79.2
7 - Rohnert Park/ Cotati	PEN-8	047-166-023	Rezone	GC	C2 WH	0.65	Warehouse	16.0	12	C2	Retail Business and Service District	50%	136	1632	0.002	9.6	0.13	14,157	1840.4	0.0018	11.5	1.3	10.9
7 - Rohnert Park/ Cotati	PEN-9	047-166-025	Rezone	GC	C2 WH	0.31	Warehouse	16.0	4	C2	Retail Business and Service District	50%	136	544	0.001	3.2	0.13	6,752	877.7	0.0009	5.5	1.0	4.2
																							178

Nonvacant Septic Connections

APN	District	ESD	Charge Code	Street Address	Street Name	County Code	CountyUseC	Land Use	Land Use Designation	Density	Lot Coverage	Acres	Assumed Flow Factor	Units	Res ESD	ADU ESD	Non Res SF	Non Res ESD	Total ESD
047-091-076	PSZ	0	0	125	Adobe	Residential	TWO SFD ON SINGLE PARCEL	UR 1	SF	1	0	0.55	170	2	2.1	0.2	0	0	2.3
047-310-008	PSZ	0	0	200	Phillips	Residential	RURAL RES W/MISC RES IMP	UR 1	SF	1	0	1.12	170	1	1.1	0.1	0	0	1.2
047-310-015	PSZ	0	0	509	Phillips	Residential	RURAL RES/SINGLE RES	UR 1	SF	1	0	1.00	170	1	1.1	0.1	0	0	1.2
047-152-013	PSZ	0	0	230	Goodwin	Residential	RURAL RES/2 OR MORE RES	UR 2	SF	2	0	0.23	170	2	2.1	0.2	0	0	2.3
047-191-050	PSZ	0	0	517	Adobe	Residential	RURAL RES/SINGLE RES	UR 1	SF	1	0	1.00	170	1	1.1	0.1	0	0	1.2
047-650-010	PSZ	0	0	820	Rainshine	Residential	RURAL RES W/MISC RES IMP	UR 1	SF	1	0	0.74	170	1	1.1	0.1	0	0	1.2
047-153-008	PSZ	0	0	5721	Old Redwood HWY	Residential	RURAL RES W/MISC RES IMP	UR 2	SF	2	0	0.72	170	1	1.1	0.1	0	0	1.2
																			11

Vacant Developable

APN	District	ESD	Charge Code	Street Address	Street Name	County Code	CountyUseC	Land Use	Land Use Designation	Density	Lot Coverage	Acres	Assumed Flow Factor	Units	Res ESD	ADU ESD	Non Res SF	Non Res ESD	Total ESD
047-212-005	PSZ	0	0	5579	OLD REDWOOD	Residential	RURAL RES/VACANT HOMESITE	UR 2	SF	2	0	0.11	170	1	1.1	0.1	0	0	1.2
047-212-002	PSZ	0	0	5575	OLD REDWOOD HWY	Residential	RURAL RES/VACANT HOMESITE	UR 2	SF	2	0	1.92	170	4	4.3	0.4	0	0	4.7
047-166-036	PSZ	0	0	0	NONE	Residential	RURAL RES/VACANT HOMESITE	RR 2	SF	2	0	6.81	170	2	2.1	0.2	0	0	2.3
047-166-045	PSZ	0	0	400	BANNON	Residential	VACANT RES LOT UNDEVEL W/UTIL	UR 4	SF	4	0	0.37	170	2	2.1	0.2	0	0	2.3
047-171-026	PSZ	0	0	9980	OAK	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 6	SF	6	0	0.18	170	2	2.1	0.2	0	0	2.3
047-171-027	PSZ	0	0	9982	OAK	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 6	SF	6	0	0.18	170	2	2.1	0.2	0	0	2.3
047-171-033	PSZ	0	0	0	NONE	Residential	VACANT RES LOT UNDEVEL W/UTIL	UR 6	SF	6	0	0.18	170	2	2.1	0.2	0	0	2.3
047-171-034	PSZ	0	0	0	NONE	Residential	VACANT RES LOT UNDEVEL W/UTIL	UR 6	SF	6	0	0.18	170	2	2.1	0.2	0	0	2.3
047-152-019	PSZ	0	0	0	GOODWIN	Residential	RURAL RES/VACANT HOMESITE	UR 2	SF	2	0	1.73	170	4	4.3	0.4	0	0	4.7
047-152-020	PSZ	0	0	0	GOODWIN	Residential	RURAL RES/VACANT HOMESITE	UR 2	SF	2	0	1.05	170	3	3.2	0.3	0	0	3.5
047-232-035	PSZ	0	0	0	DAVIS	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 1	SF	1	0	0.50	170	1	1.1	0.1	0	0	1.2
047-232-036	PSZ	0	0	0	DAVIS	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 1	SF	1	0	0.63	170	1	1.1	0.1	0	0	1.2
047-164-027	PSZ	0	0	0	NONE	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 4	SF	4	0	0.23	170	1	1.1	0.1	0	0	1.2
047-181-028	PSZ	0	0	11601	PETALUMA HILL	Residential	VACANT RES LOT UNDEVEL W/UTIL	LI	IND		0.5	1.73	170	0	0.0	0.0	37679	28.8	28.8
047-052-030	PSZ	0	0	0	PENNGROVE	Residential	RURAL RES/VACANT HOMESITE	UR 2	SF	2	0	0.48	170	1	1.1	0.1	0	0	1.2
047-213-009	PSZ	0	0	79	ELY	Commercial	VACANT COMMERCIAL LND W/UTIL	LC	COMM		0.5	0.91	170	0	0.0	0.0	19885	15.2	15.2
047-232-032	PSZ	0	0	0	DAVIS	Residential	RURAL RES/VACANT HOMESITE	UR 1	SF	1	0	0.29	170	1	1.1	0.1	0	0	1.2
047-164-019	PSZ	0	0	0	GROVE	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 4	SF	4	0	0.26	170	2	2.1	0.2	0	0	2.3
047-171-031	PSZ	0	0	0	NONE	Residential	VACANT RES LOT UNDEVEL W/UTIL	UR 6	SF	6	0	0.17	170	2	2.1	0.2	0	0	2.3
047-310-016	PSZ	0	0	200	NITA	Residential	RURAL RES/VACANT HOMESITE	UR 1	SF	1	0	1.00	170	2	2.1	0.2	0	0	2.3
047-173-018	PSZ	0	0	9500	MAIN	Commercial	VACANT COMMERCIAL LND/UNDEVEL	LC	COMM		0.5	0.46	170	0	0.0	0.0	10106	7.7	7.7
047-181-032	PSZ	0	0	9585	MAIN	Commercial	VACANT COMMERCIAL LND/UNDEVEL	LC	COMM		0.5	0.11	170	0	0.0	0.0	2309	1.8	1.8
047-232-038	PSZ	0	0	0	DAVIS	Residential	VACANT RESIDENTIAL LOT/UNDEVEL	UR 1	SF	1	0	0.47	170	1	1.1	0.1	0	0	1.2
047-670-003	PSZ	0	0	0	NONE	Residential	RURAL RES/VACANT HOMESITE	UR 1	SF	1	0	0.69	170	1	1.1	0.1	0	0	1.2
047-670-002	PSZ	0	0	0	NONE	Residential	RURAL RES/VACANT HOMESITE	UR 1	SF	1	0	1.01	170	2	2.1	0.2	0	0	2.3
																			100

APPENDIX C: FLOW CALIBRATION GRAPHS

FIGURE C-1: PLOT OF FLOW CALIBRATION FOR JAN-MARCH 2024 (PSZ_M18-4)

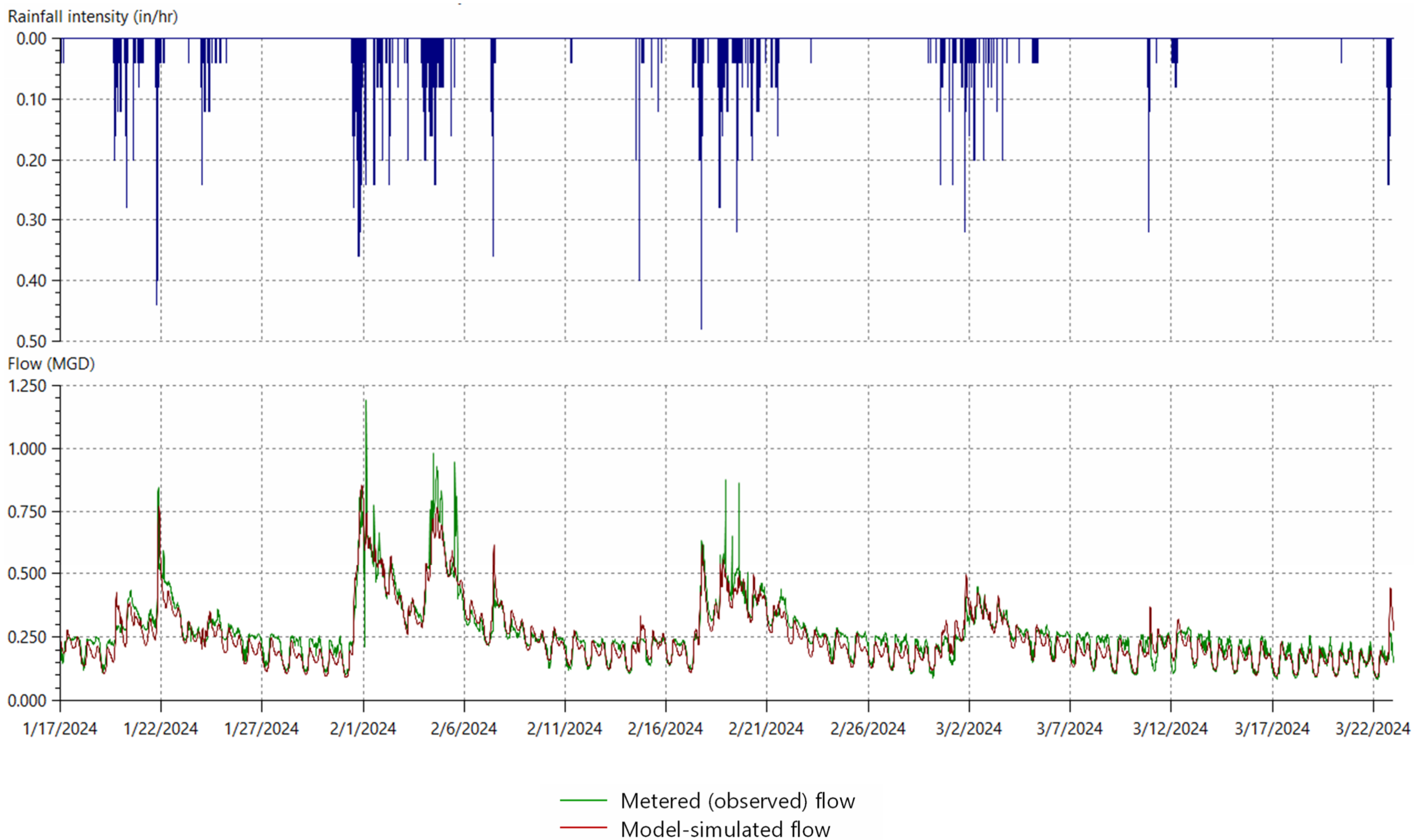


FIGURE C-2: PLOT OF FLOW CALIBRATION FOR JAN-MARCH 2024 (PSZ_M21-1)

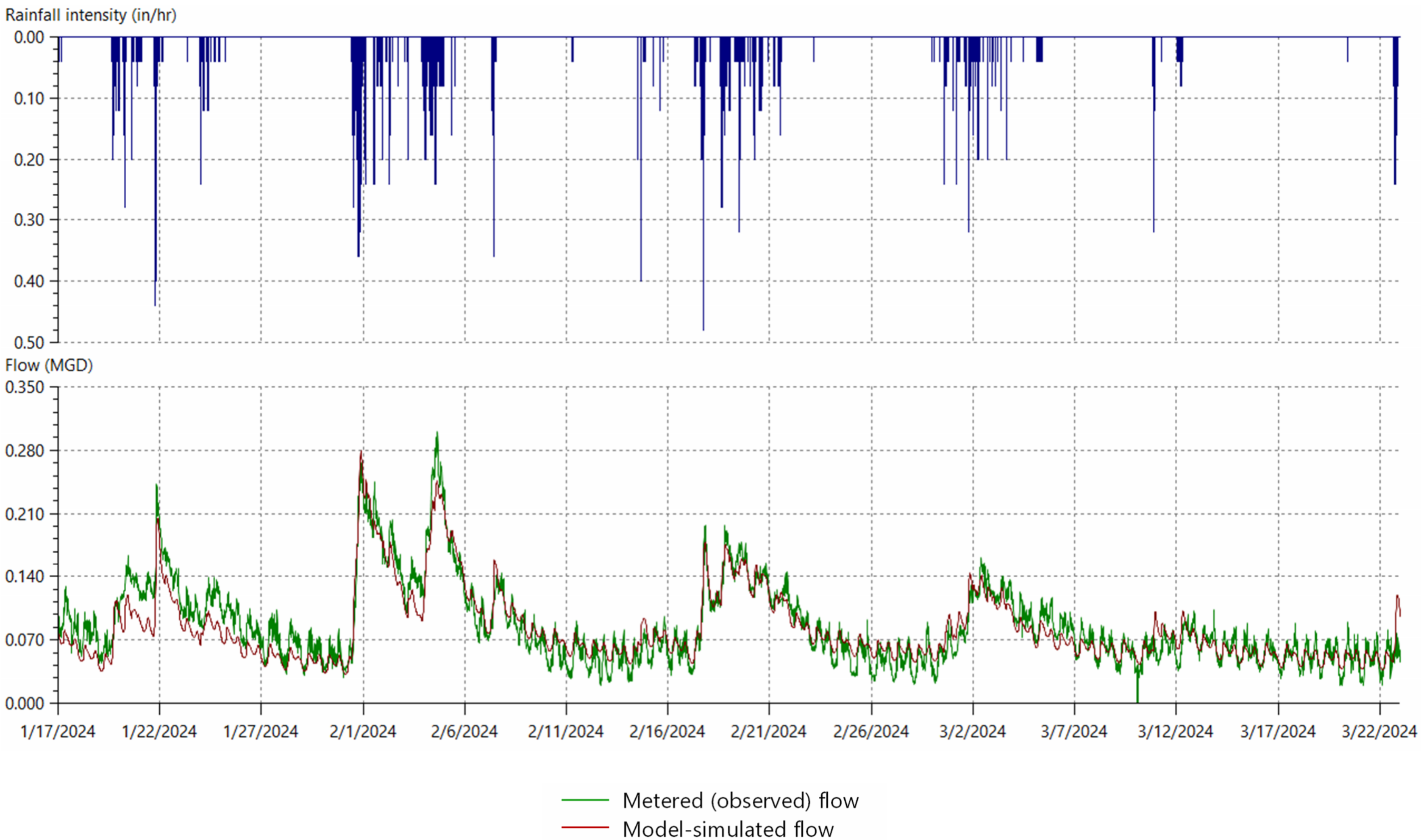


FIGURE C-3: PLOT OF FLOW CALIBRATION FOR JAN-MARCH 2024 (PSZ_M14-12)

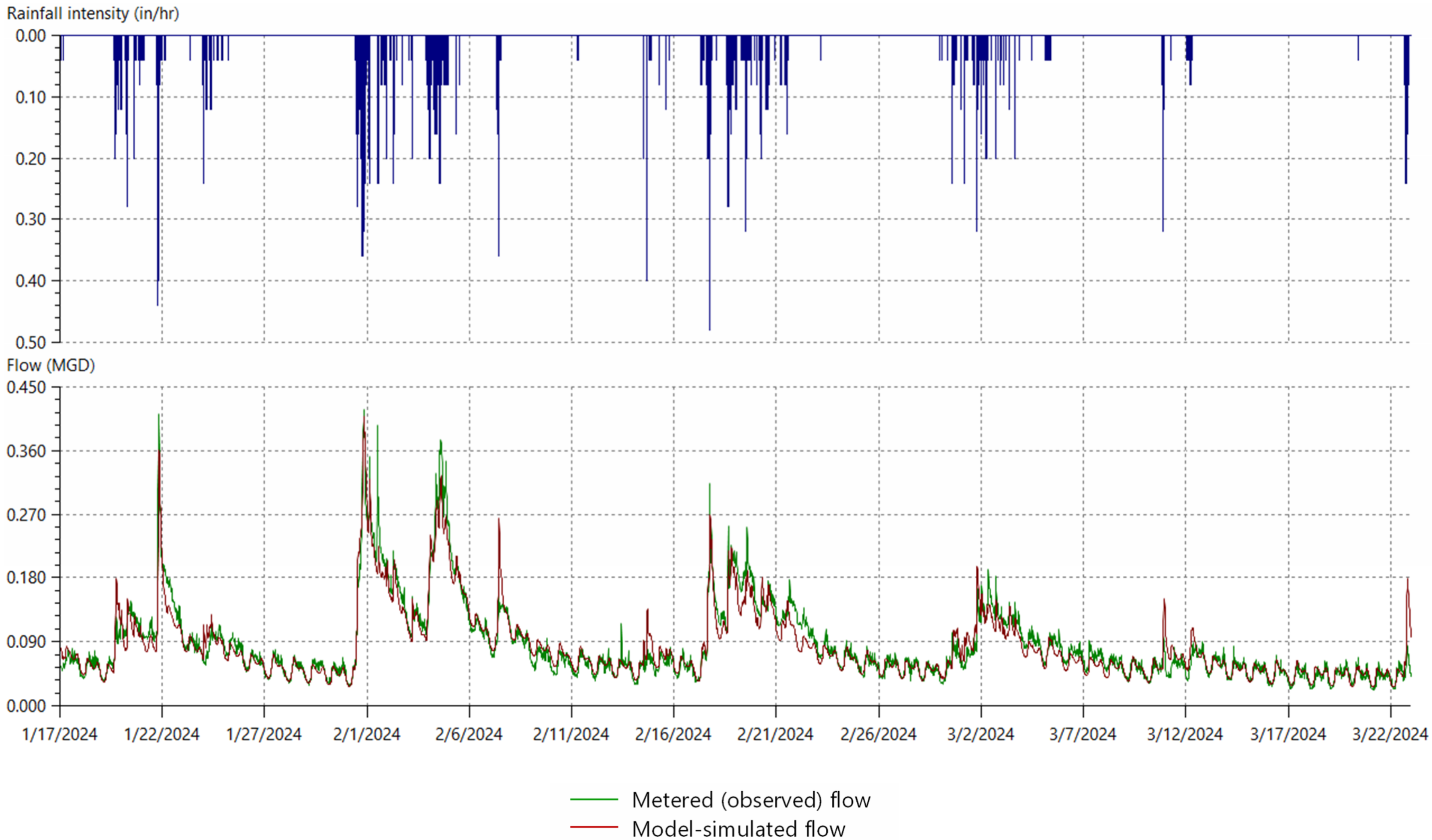
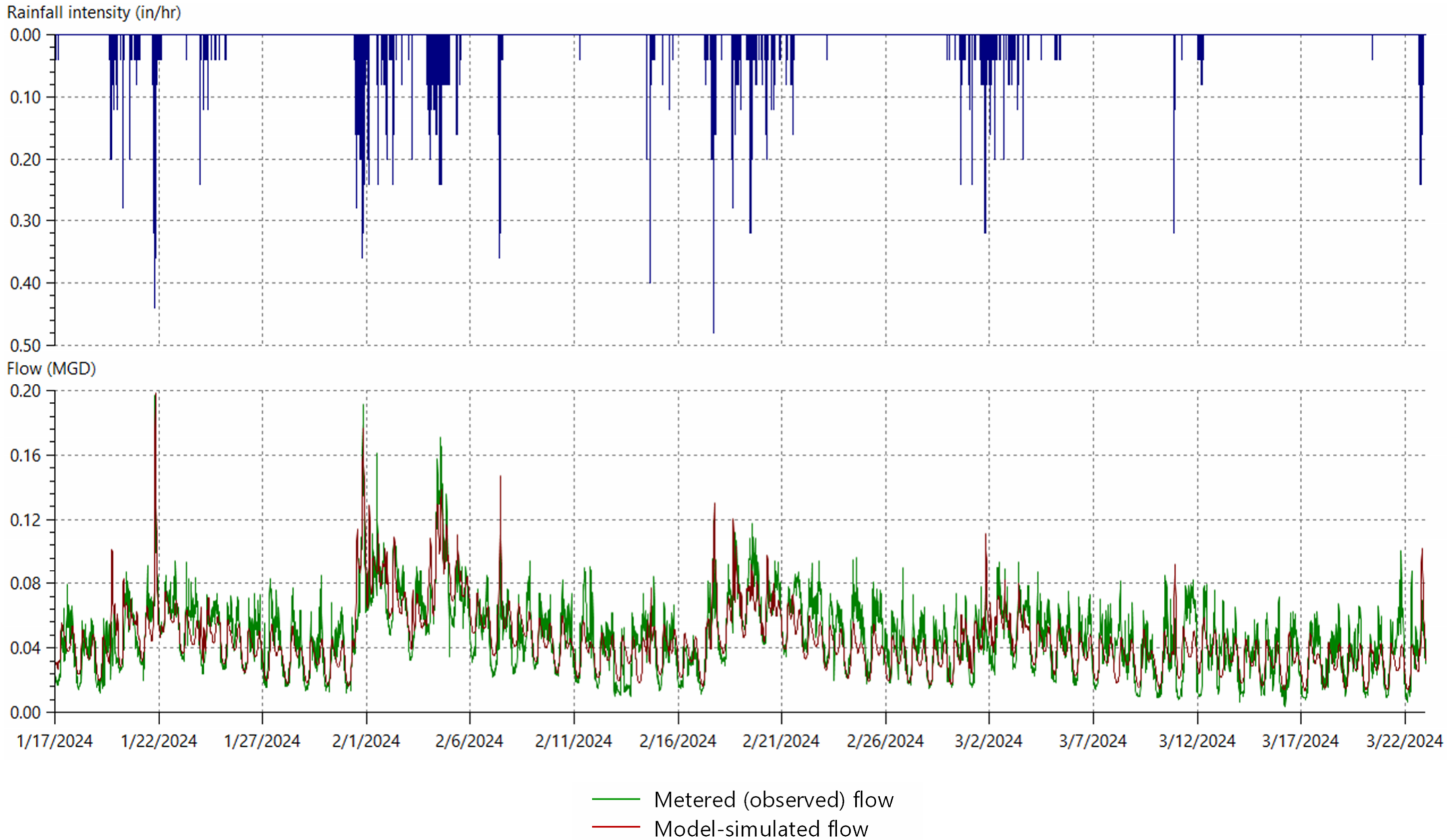


FIGURE C-4: PLOT OF FLOW CALIBRATION FOR JAN-MARCH 2024 (PSZ_M15-12)



APPENDIX D: PUMP AND SYSTEM CURVES

City of Petaluma
Penngrove Pump and System Curve (Corona Road)

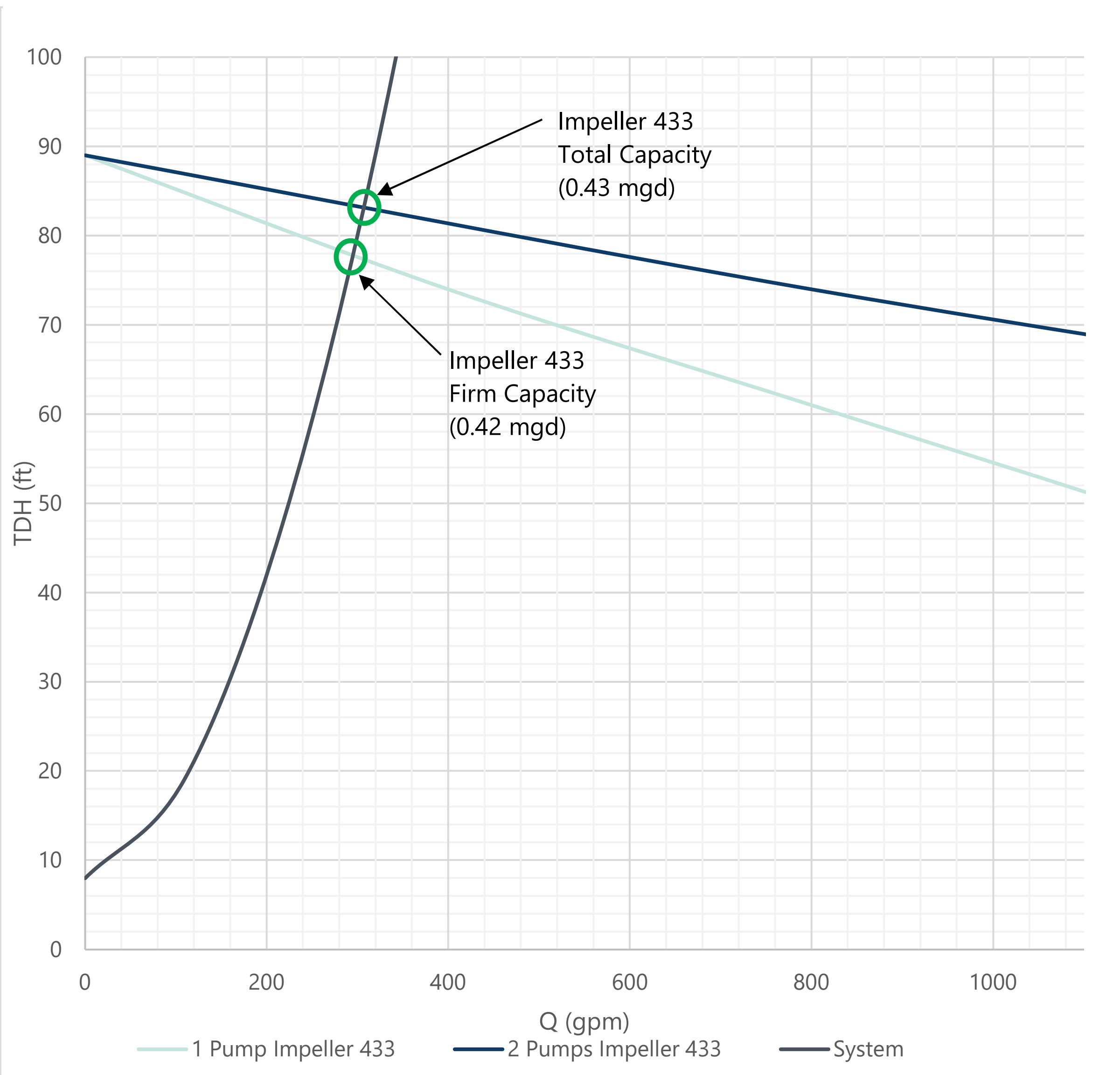
Pump Design Criteria	Value	Unit
Design Head		ft
Design Flow		gpm

Flow Range	gpm
Low	0
High	1500

Pump Configuration	1+1

	Value	Unit
Static Head	8.00	ft

	Diameter (in)	Length (ft)	Material	C factor	Total K
Force Main	6	6430.90	PVC	110	5.00



City of Petaluma
Penngrove Pump and System Curve (Just Beyond HWY-101 Crossing)

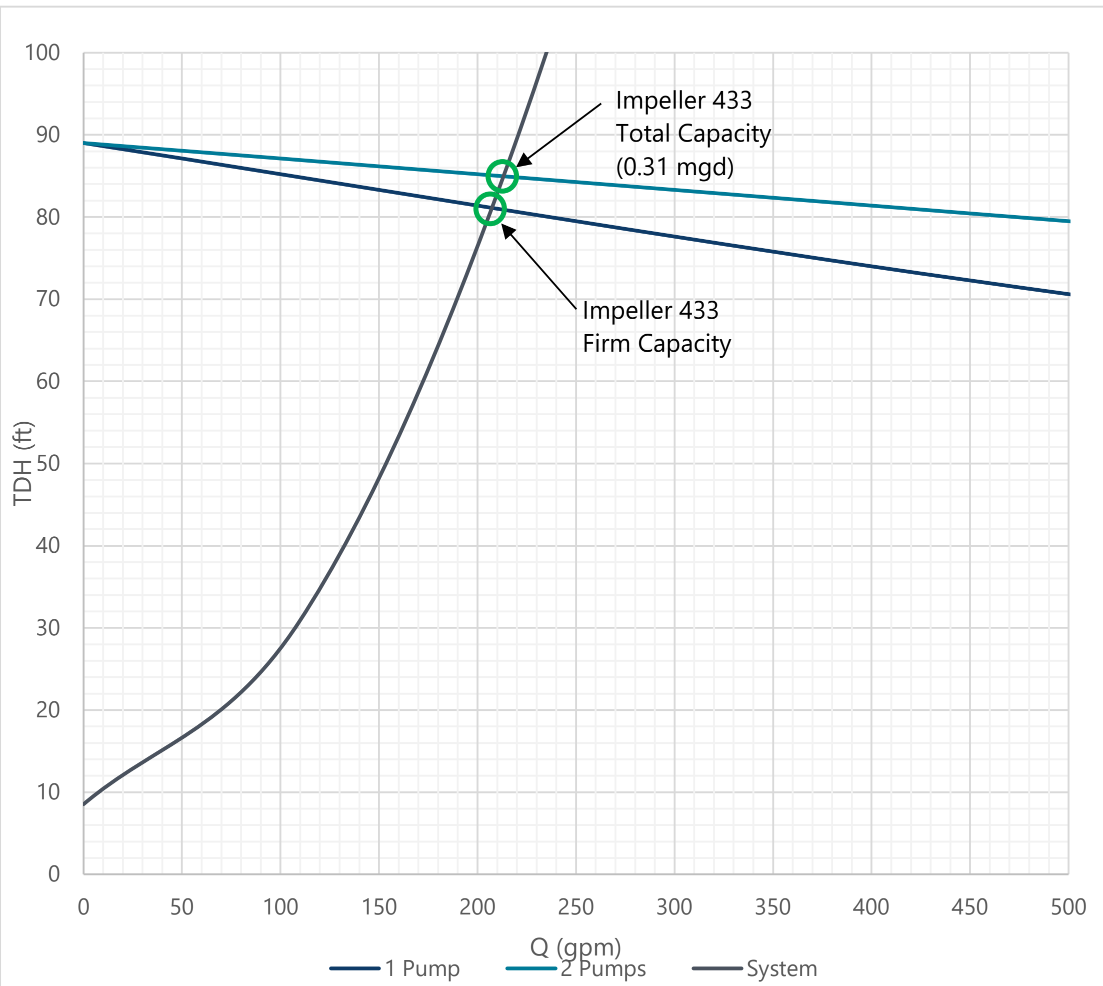
Pump Design Criteria	Value	Unit
Design Head		ft
Design Flow		gpm

Flow Range	gpm
Low	0
High	1500

Pump Configuration	1+1

	Value	Unit
Static Head	8.55	ft

	Diameter (in)	Length (ft)	Material	C factor	Total K
Force Main from Corona to just past HWY	6	6420.60	PVC	110	5.00
Force Main from Penngrove PS to Corona	6	6430.9	PVC	110	5.00



City of Petaluma
Penngrove Pump and System Curve (Wilmington PS)

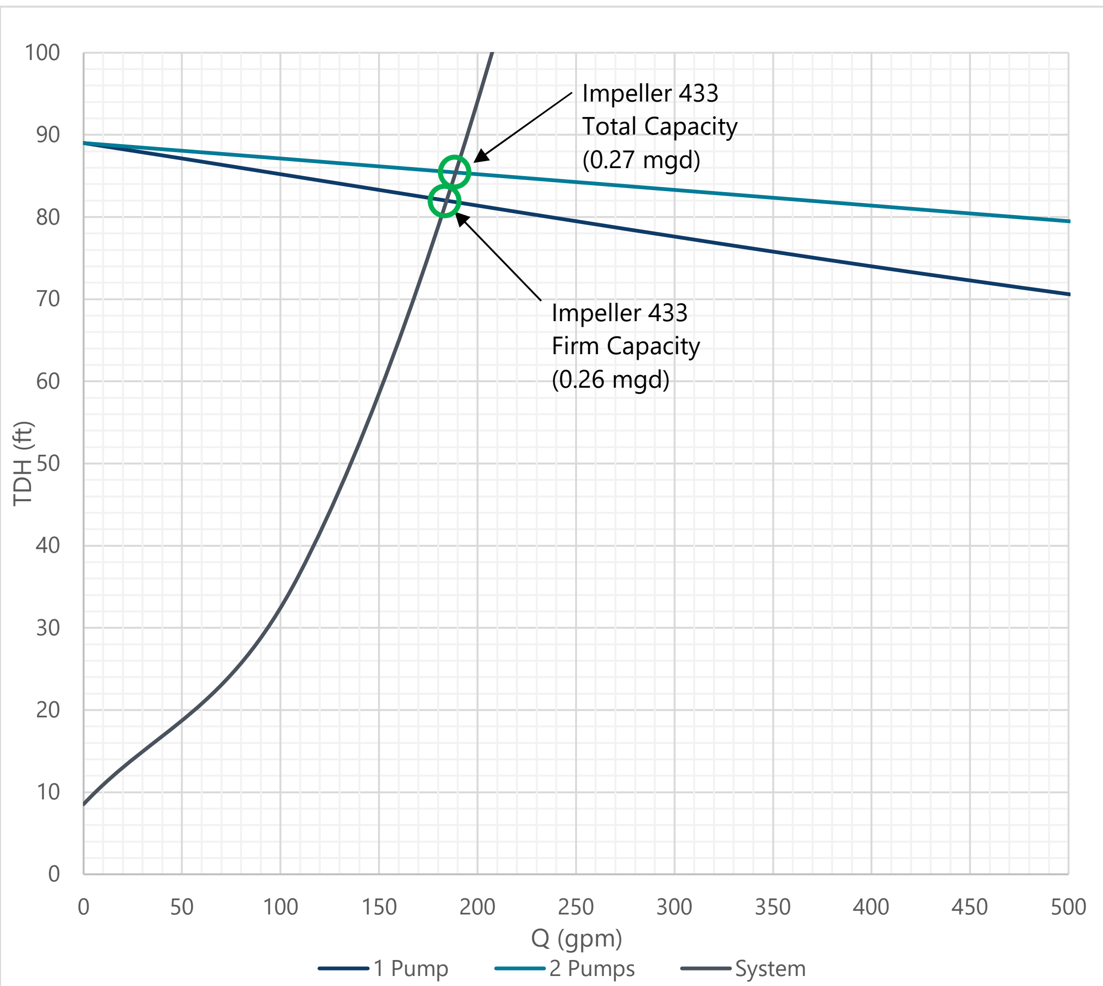
Pump Design Criteria	Value	Unit
Design Head		ft
Design Flow		gpm

Flow Range	gpm
Low	0
High	1500

Pump Configuration	1+1

	Value	Unit
Static Head	8.55	ft

	Diameter (in)	Length (ft)	Material	C factor	Total K
Force Main from Corona to Wilmington	6	9799.62	PVC	110	5.00
Force Main from Penngrove PS to Corona	6	6430.9	PVC	110	5.00



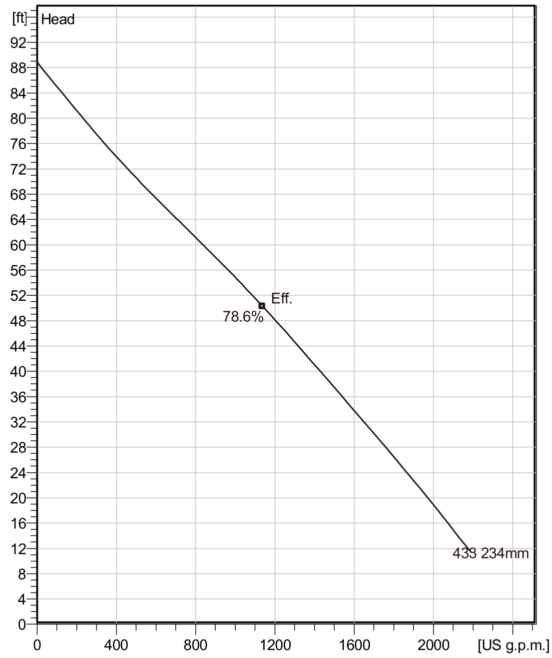
NP 3153 MT 3~ 433

Patented self cleaning semi-open channel impeller, ideal for pumping in waste water applications. Modular based design with high adaptation grade.

FLYGT a xylem brand

Technical specification

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Curve: ISO 9906

Configuration

Motor number
N3153.095 21-18-4AA-W
20hp

Installation type
P - Semi permanent, Wet

Impeller diameter
234 mm

Discharge diameter
6 inch

Pump information

Impeller diameter
234 mm

Discharge diameter
6 inch

Inlet diameter
150 mm

Maximum operating speed
1755 rpm

Number of blades
2

Max. fluid temperature
40 °C

Materials

Impeller
Hard-Iron ™

Project		**Created by**	Ricardo Garcia		
Block		**Created on**	6/1/2022	**Last update**	6/1/2022

NP 3153 MT 3~ 433

Technical specification

Motor - General

Motor number N3153.095 21-18-4AA-W 20hp	**Phases** 3~	**Rated speed** 1755 rpm	**Rated power** 20 hp
ATEX approved FM	**Number of poles** 4	**Rated current** 52 A	**Stator variant** 5
Frequency 60 Hz	**Rated voltage** 230 V	**Insulation class** H	**Type of Duty** S1
Version code 095			

Motor - Technical

Power factor - 1/1 Load 0.83	**Motor efficiency - 1/1 Load** 87.5 %	**Total moment of inertia** 0.0904 kg m²	**Starts per hour max.** 30
Power factor - 3/4 Load 0.77	**Motor efficiency - 3/4 Load** 89.0 %	**Starting current, direct starting** 296 A	
Power factor - 1/2 Load 0.66	**Motor efficiency - 1/2 Load** 89.0 %	**Starting current, star-delta** 98.6 A	

Project		**Created by**	Ricardo Garcia		
Block		**Created on**	6/1/2022	**Last update**	6/1/2022

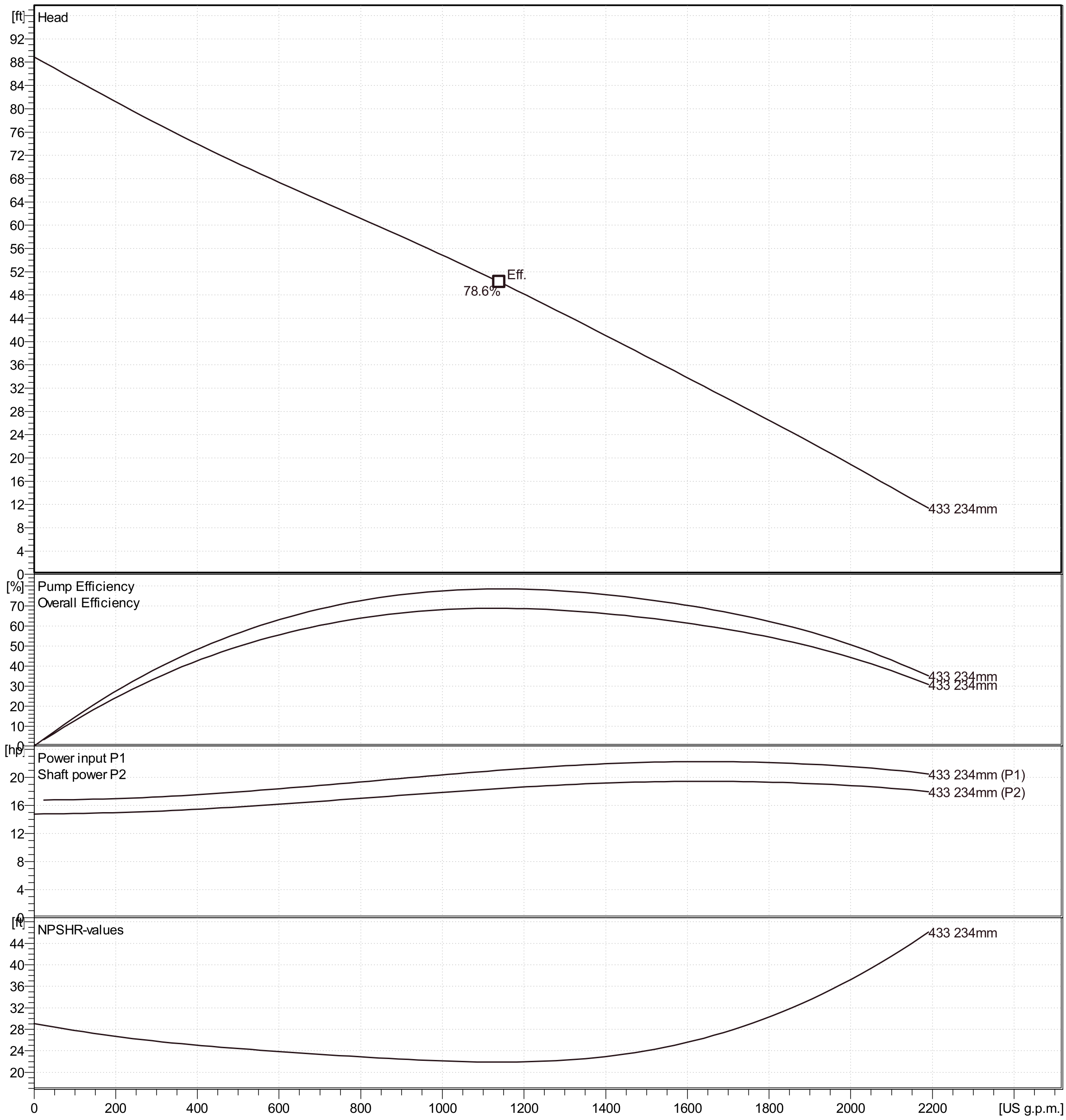
NP 3153 MT 3~ 433

Performance curve

Duty point

Flow **Head**

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Curve: ISO 9906

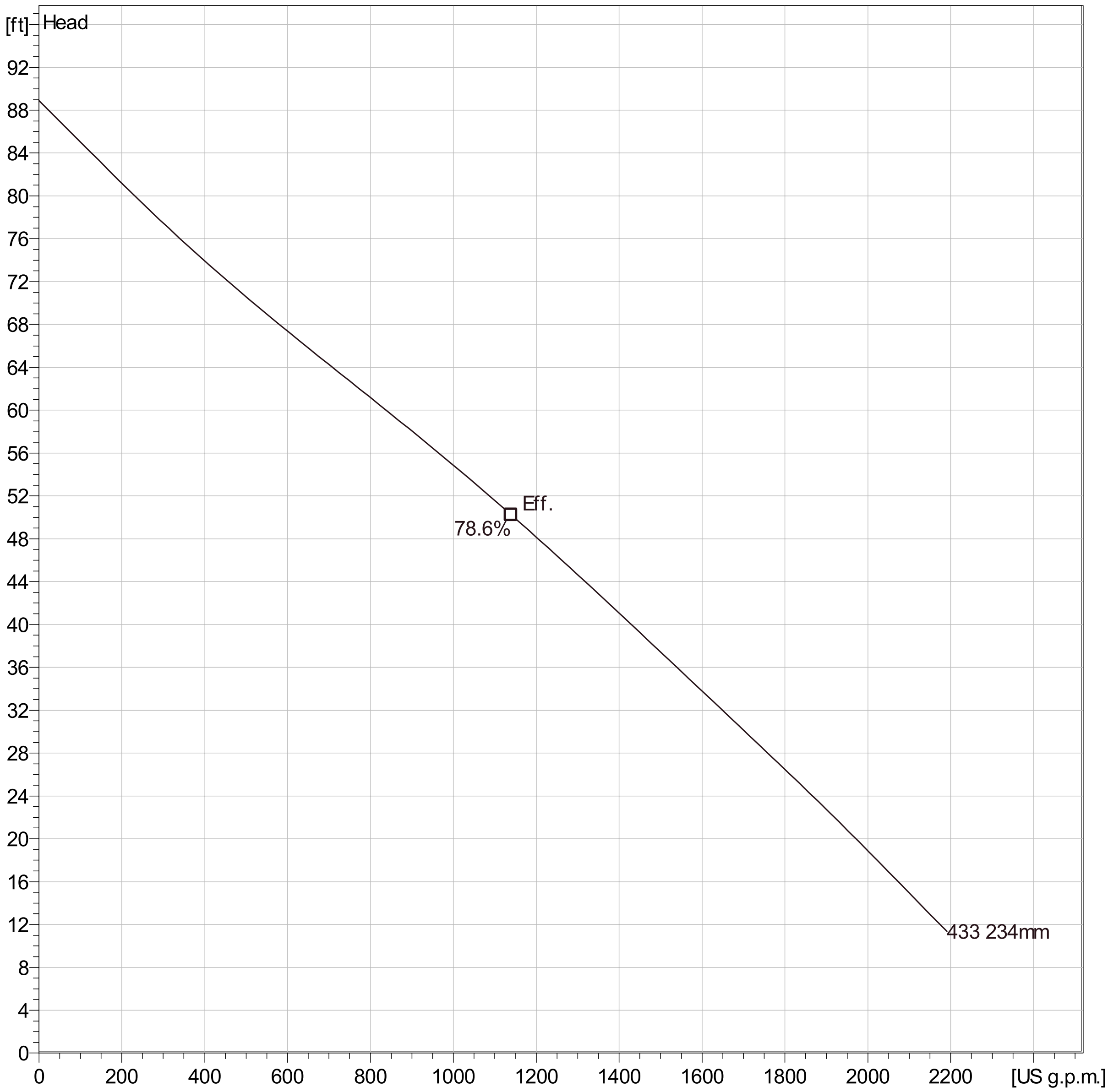
Ricardo Garcia

Created on 6/1/2022 **Last update** 6/1/2022

NP 3153 MT 3~ 433

Duty Analysis

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s



Operating characteristics

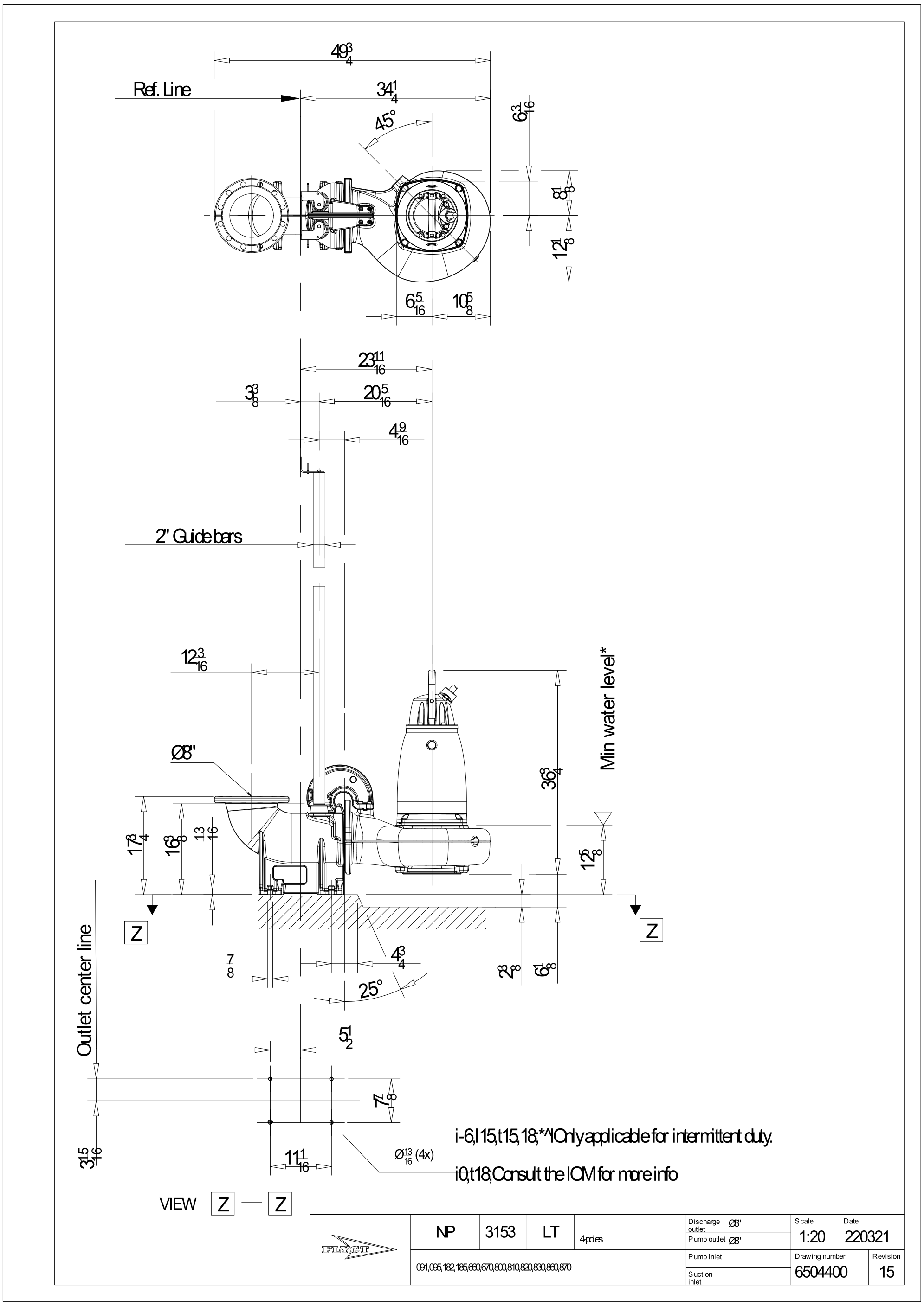
Pumps / Systems	Flow US g.p.m.	Head ft	Shaft power hp	Flow US g.p.m.	Head ft	Shaft power hp	Hydr.eff.	Spec. Energy kWh/US MG	NPSHre ft

Project		**Created by**	Ricardo Garcia
Block		**Created on**	6/1/2022

Last update 6/1/2022

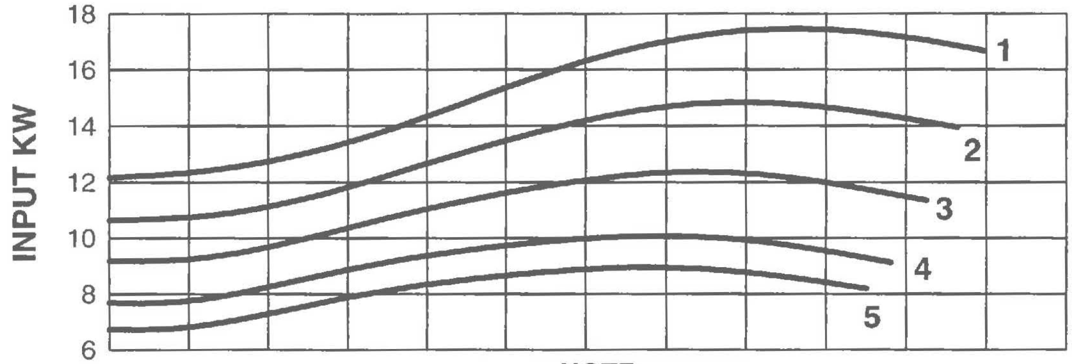
NP 3153 MT 3~ 433

Dimensional drawing

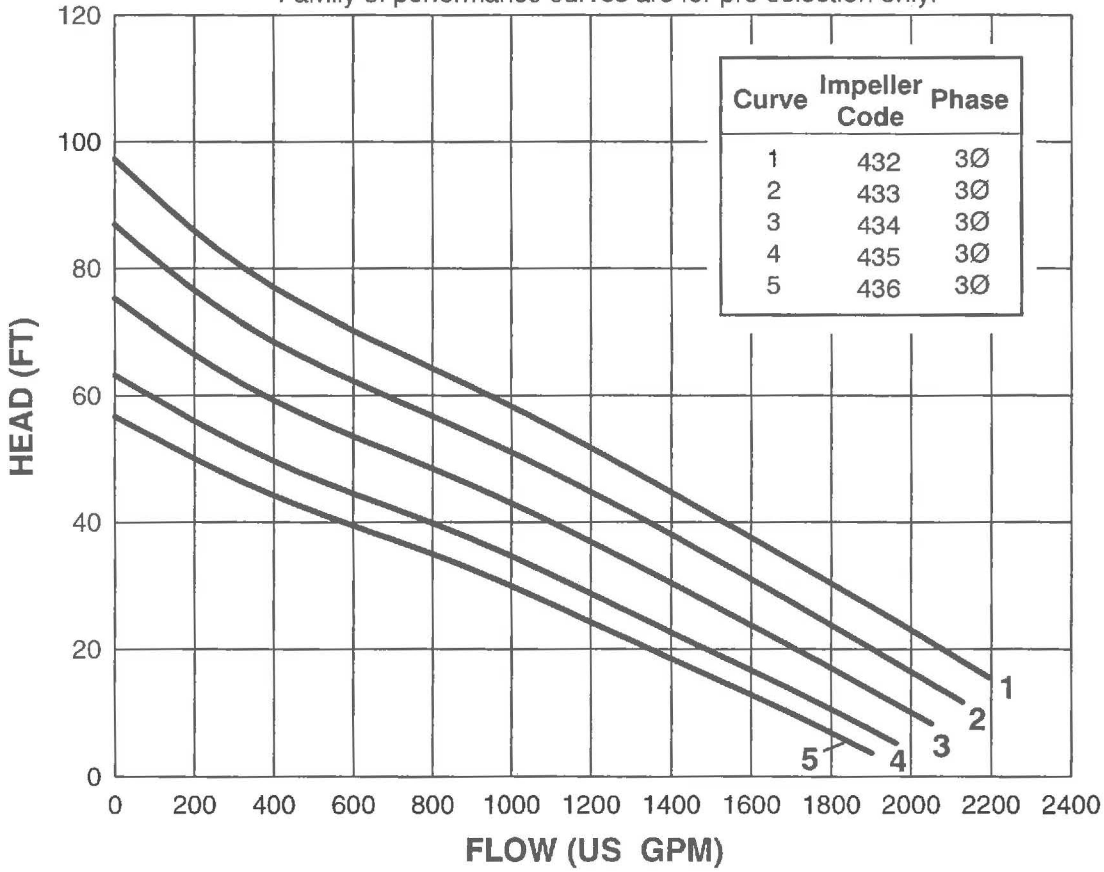


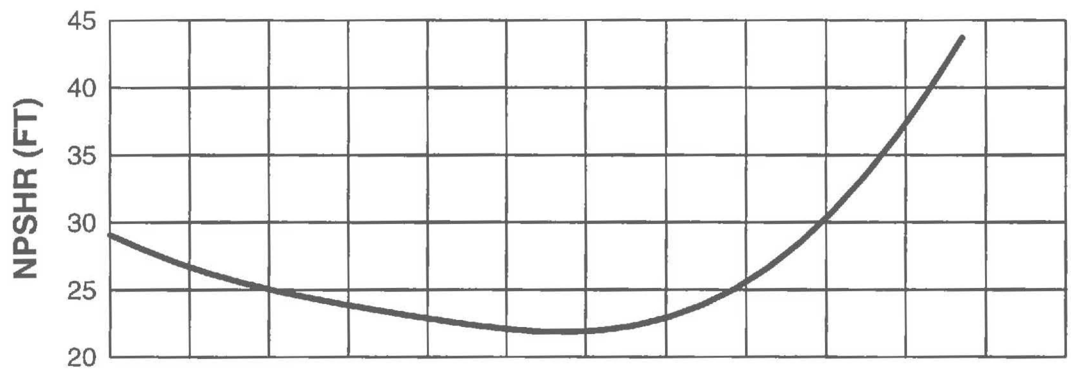
Project		**Created by**	Ricardo Garcia
Block		**Created on**	6/1/2022
		Last update	6/1/2022

N-3153
(MT Impellers)



NOTE:
Family of performance curves are for pre-selection only.





APPENDIX E: SOURCE DETECTION METHODS

Smoke Testing

Smoke testing has historically been one of the most commonly used methods for I/I source detection. Smoke testing involves blowing a non-toxic smoke into the sewers at selected manholes (typically spaced at 600 to 800-foot intervals) and observing and documenting the locations where the smoke emerges. These locations, called "smoke returns," are assumed to be locations where rainwater can enter the sewer system. These may be surface connections where stormwater runoff can enter the system directly ("inflow"), such as directly connected roof downspouts, driveway and area drains, open cleanouts, holes in manhole covers, and storm drain inlets where there is a piped connection from the storm drain to the sanitary system. Most cases, however, are found to be indirect connections where rainwater seeping into the ground enters the sewer system through cracks or leaky joints in sewer pipes, manhole walls, defective cleanouts, and service laterals ("infiltration"). The smoke returns are normally documented by photographs, sketches, and other information including their location (address), type of source, smoke intensity, and the estimated tributary drainage area of the I/I source.

In California, smoke testing is normally conducted during the summer or fall months under dry soil conditions to maximize the amount of smoke that can pass through the soil. However, it may not be an effective method for identifying infiltration sources in areas with year-round high groundwater. While a fairly effective method for identifying direct inflow sources, smoke testing is not necessarily conclusive for identifying sources of infiltration, as submerged defects will not be identified, and detection is not possible if the smoke cannot reach the ground surface, for example due to surface pavement or deep sewers.

Smoke testing is considered relatively inexpensive compared to other source detection methods. However, considerable public outreach and notification is usually required.

Dye Testing/Dye Flooding

Dye testing and dye flooding are used to identify potential cross connections between storm and sanitary sewers. Dye testing typically involves introducing a fluorescent dye into catch basins, storm drain manholes, or suspected directly connected area drains or roof downspouts and observing downstream manholes to detect if the dye has entered the sewer system. Dye water flooding involves plugging the ends of a section of storm drain and filling it with dyed water. Dye testing or flooding is often used as a verification method after a suspected storm drain cross connection is detected by smoke testing. CCTV inspection used in conjunction with dye testing or dye flooding can identify exact locations of cross connections between storm drain and sewer system and can sometimes detect indirect (infiltration) connections where water from a storm drain exfiltrates from defects in the storm drain pipe into defects in a sanitary sewer located at a lower elevation.

Closed-Circuit Television (CCTV) Inspection

CCTV is the primary method for evaluating the internal condition of sanitary sewer pipelines. CCTV inspection involves running a remotely controlled camera through the pipe from manhole to manhole and documenting observations of construction features (e.g., lateral connections) and defects. Documentation includes a video recording, still image photographs of observations, and other information input into a database using one of several available CCTV software programs. Over the past 10+ years, CCTV observation coding has become widely standardized using the National Association of Sewer Service Company (NASSCO) Pipeline Assessment Certification Program (PACP).

Commonly observed defects identified through CCTV include structural problems such as cracks, offset joints, corrosion, and sags; as well as maintenance-related issues such as root intrusion, debris, and grease. Active I/I can sometimes be observed (typically groundwater infiltration); however, rainfall-induced infiltration may also be seen if the inspection is conducted during or immediately following a rainfall event and the pipe is not overly full. Observed infiltration may range from a wet surface ("seeper") to a significant flow discharge ("gusher"). CCTV inspection, however, cannot observe active infiltration if the infiltration is entering the pipe below the water surface.

CCTV inspection is not usually done for the sole purpose of actually observing active infiltration. Rather, the defects observed during the inspection, such as cracks, open joints, defective lateral connections, and root intrusion. are assumed to be locations where infiltration could enter the sewer pipe under rainfall or high groundwater conditions in areas where flow monitoring has documented the presence of such extraneous flows.

While most CCTV inspection is done on sewer mains, CCTV of laterals is also possible, either through the use of "push cameras" inserted into lateral cleanouts, or sometimes through cameras that can be "launched" up the lateral from mainline during the mainline CCTV inspection.

Manhole Inspection

Visual inspections of manholes can be performed to identify structural, construction, or maintenance defects that may allow entry of I/I. As with CCTV inspection, NASSCO has developed standard codes and data format for manhole inspections under its Manhole Assessment Certification Program (MACP). Manhole inspections can be conducted from "topside" without entering the manhole (MACP Level 1) or a more detailed inspection by man-entry (MACP Level 2). The inspections can also be conducted using video camera similar to CCTV inspection of the pipe.

Focused Electrode Leak Location

Focused electrode leak location, or electro-scanning, is a proprietary method to detect defects in a non-metallic pipe (such as clay, concrete, or plastic) by measuring the electrical resistivity of the pipe wall. A pipe location that allows water to penetrate or leak also allows electrical current to escape. To detect pipe defects, electro-scanning involves filling the sewer pipe with water, then setting a fixed voltage between an electrode sent through the pipe (a "sonde") and an electrode at the surface. Current measurements are taken continuously while pulling the sonde through the pipe at a speed of 30 feet per minute. When the sonde comes within 20 to 30 mm of a pipe defect, electric current escapes through the defect, causing the current measurement to increase, peaking when the center of the sonde is aligned with the defect. The current measurements relative to distance along the pipe are recorded. The grade of the defect can be rated (e.g., "small," "medium," or "large,") based on a sonde current "threshold" established from comparison studies between previous electro-scan data and joint pressure tests. The electro-scan information can be used to prioritize pipes for further inspection and to guide a CCTV camera to the location of the leak to identify the type and size of the defect (if the defect can be seen). Focused electrode leak location can also be used to find leaks associated with non-visual defects such as cracked joint sealing material.

Pressure Testing

Pressure testing of sewer mains is used to determine the integrity of joints, which if leaky, are potential entry points for infiltration. Joint pressure testing uses an in-pipe packer to isolate the joint and then apply air to test the joint under pressure. Sometimes chemical grout is then injected to seal off any failed joints.

CCTV is typically used as part of this process to locate the joints and monitor the testing and sealing procedure.

Service laterals can also be pressure tested using air or water (exfiltration test) by plugging the lateral at the connection to the main and/or at property line or building cleanouts (depending on which portion of the lateral is tested). A failed test indicates that the lateral is leaky, although the specific defects would still need to be identified by CCTV inspection. Alternately, the lateral can be deemed to have "failed" the test and the entire pipe considered to be a discrete I/I source.

Rainfall Simulation

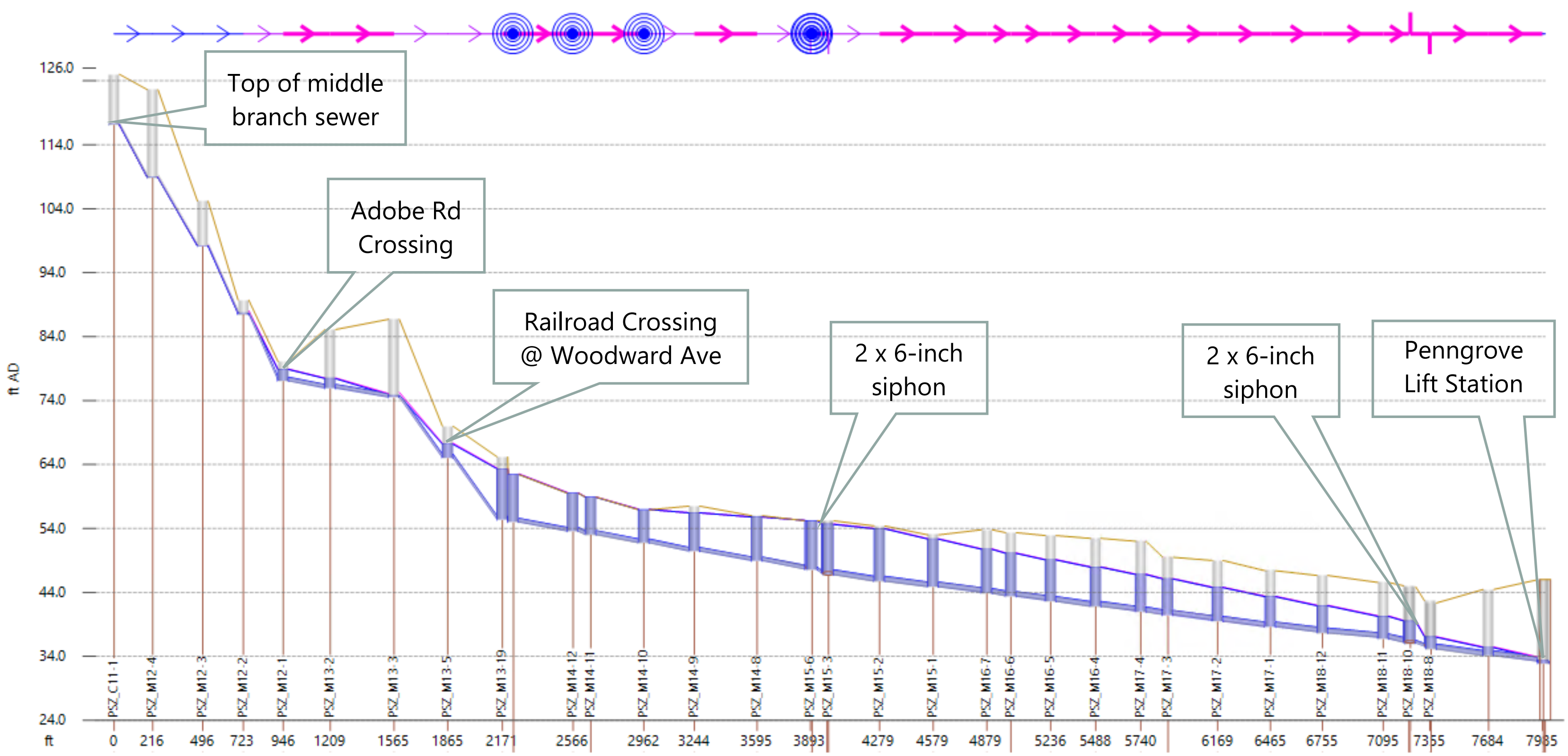
This method is typically used to identify and quantify potential infiltration from leaking service laterals. A rainfall event is simulated by applying water on an area along the lateral alignment using lawn sprinklers. The water is applied for several hours at a known rate in order to saturate the area. CCTV inspection is then performed in the sewer main to which the lateral is connected to observe and estimate the rate of flow from the lateral into the sewer.

Other Methods

Other I/I field investigation methods include temperature or wastewater strength sampling to assess the relative amount of non-sanitary flow in the wastewater stream, conductivity (salinity) monitoring to assess potential infiltration of seawater in tidal areas, as well as hydrogeological investigations to assess groundwater levels. These methods are more properly characterized as flow isolation approaches, as they may identify areas with I/I but not specific sources (defects).

APPENDIX F: HYDRAULIC GRADE PROFILES, LIFT STATION CAPACITY LIMITATIONS RELIEVED

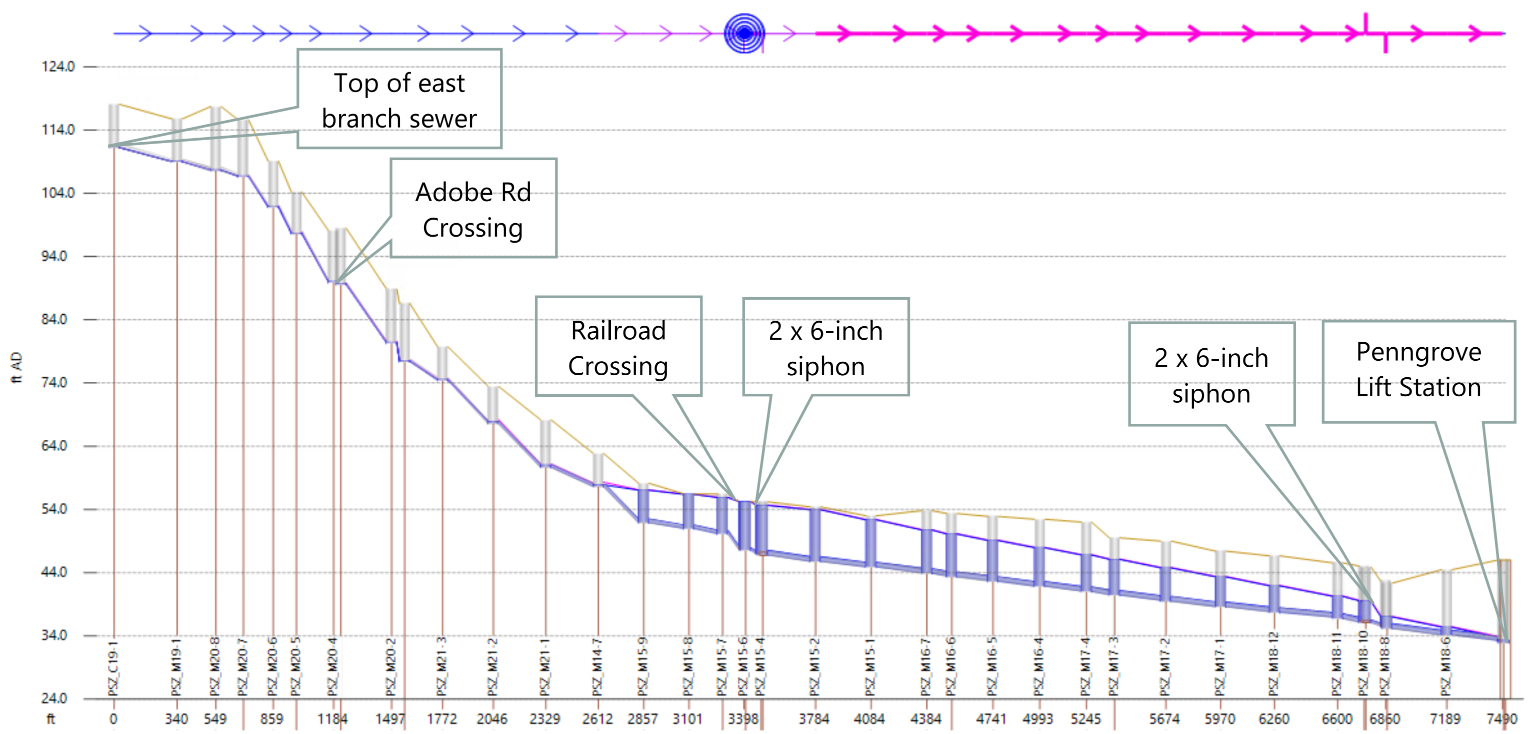
FIGURE F-1: MODELED PROFILE UNDER DESIGN STORM CONDITIONS, EXISTING LAND USE, MIDDLE BRANCH SEWER, LIFT STATION CAPACITY LIMITATIONS RELIEVED



LEGEND

- No Surcharge
- Backwater Surcharge
- Throttle Surcharge
- Model-predicted Overflow

FIGURE F-2: MODELED PROFILE UNDER DESIGN STORM CONDITIONS, EXISTING LAND USE, EAST BRANCH SEWER, LIFT STATION CAPACITY LIMITATIONS RELIEVED



LEGEND

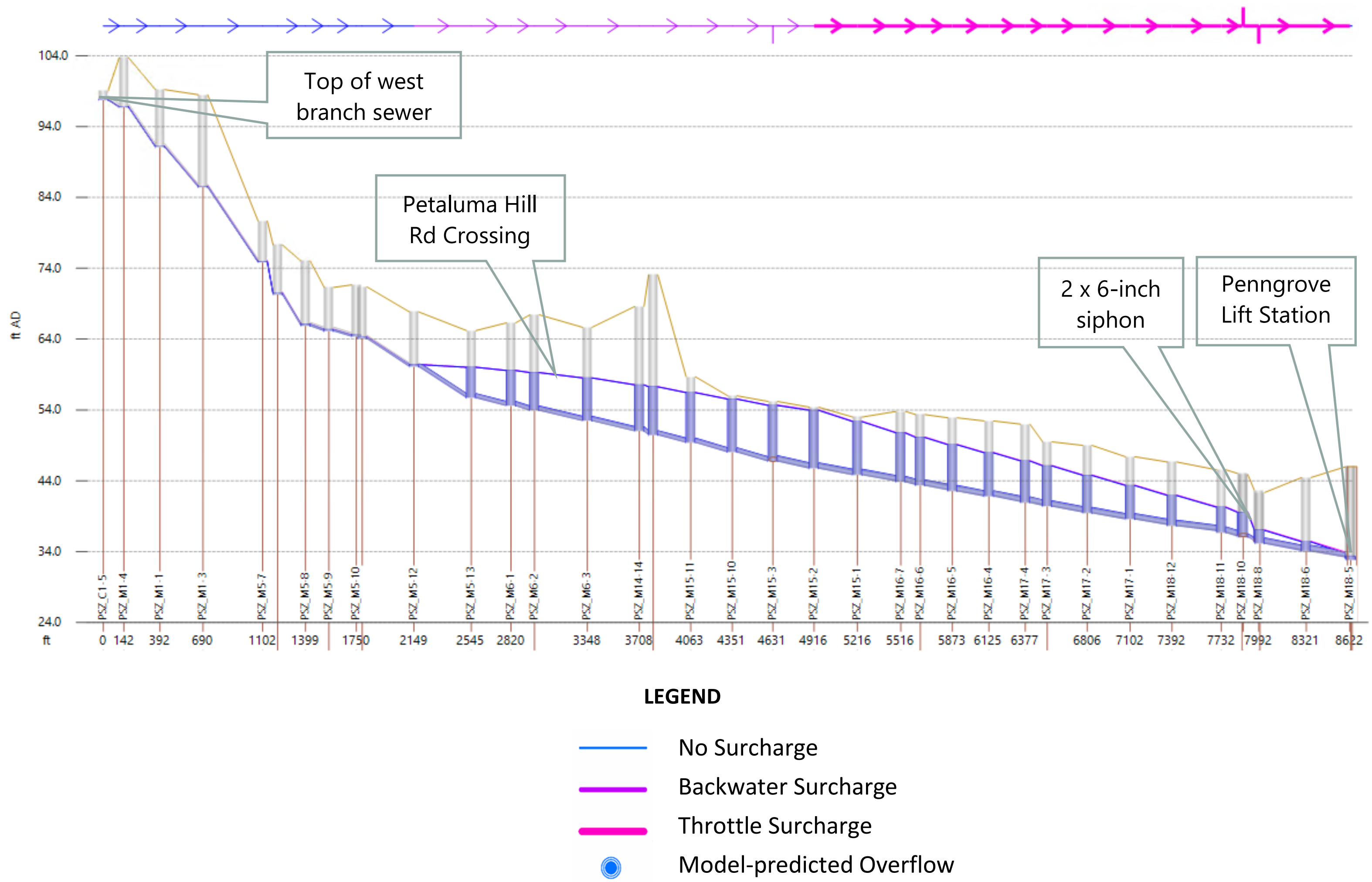
No Surcharge

Backwater Surcharge

Throttle Surcharge

Model-predicted Overflow

FIGURE F-3: MODELED PROFILE UNDER DESIGN STORM CONDITIONS, EXISTING LAND USE, WEST BRANCH SEWER, LIFT STATION CAPACITY LIMITATIONS RELIEVED



Woodard & Curran

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