Technical Memorandum 3B

NSMCSD Collection System Capacity Evaluation/Assurance, Management and Improvement Plan

Subject: **Task 3B: Collection System Capacity Evaluation**

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This Technical Memorandum (TM) presents the results and recommendations of the Collection System Capacity Evaluation for the North San Mateo County Sanitation District (NSMCSD, District). This TM entitled, "Task 3B: Collection System Capacity Evaluation" has been prepared by RMC for the District as part of the NSMCSD Collection System Evaluation Project. Previous TMs prepared as part of this project are entitled:

1. Task 2: Review of 1993 Collection System Master Plan
2. Task 3A: Flow Monitoring Results
3. Task 4: Condition Assessment and Rehabilitation Program Recommendations

The purpose of this capacity evaluation was to provide an up-to-date assessment of the capacity of the sanitary sewer system to handle both existing and future wastewater flows. In addition, this evaluation will help NSMCSD meet the requirements to complete a capacity evaluation and capacity assurance plan as part of preparing its Sewer System Management Plan (SSMP).

This TM is divided into the following sections:

1. Introduction
2. Planning Scenarios
3. Hydraulic Model Development
4. Existing and Future Wastewater Flows
5. Sewer System Capacity Analysis
6. Recommended Capacity Improvement Program

1 Introduction

The objective of the Collection System Capacity Evaluation is to help the District meet the requirements to complete a capacity evaluation and capacity assurance plan as part of preparing its Sewer System Management Plant (SSMP), as well as provide information to update projected sewer capacity improvement needs in the District's Capital Improvement Program (CIP). The SSMP addresses the overall management, operation, and maintenance of the sanitary sewer system and is required for all sewer system agencies in the San Francisco Bay Area by the Regional Water Quality Control Board (RWQCB), as well as under the Statewide General Waste Discharge Requirements (WDRs) adopted in 2006 by the State Water Resources Control Board (SWRCB). In addition, this TM updates the findings and recommendations presented in the 1993 City of Daly City Collection System Master Plan (1993 Master Plan) and ensures that the SSMP reflects the most up-to-date information about the capacity of the collection system.

1.1 Study Area

The study area for this capacity evaluation consists of the entire sewer network draining to the NSMCSD wastewater treatment plant (WWTP). The service area for the NSMCSD WWTP is primarily located within Daly City (City), but also includes the unincorporated community of Broadmoor (which is also part of the District), some portions of the Town of Colma, and the Westborough Water District (WWD). WWD serves the western portion of the City of South San Francisco.

Daly City is located in the northern part of San Mateo County and is bordered by the City and County of San Francisco on the north, the Town of Colma on the east, the City of South San Francisco on the southeast, and the City of Pacifica on the southwest. The sewers in the northeastern portion of the NSMCSD service area are combined sewers that are tributary to the City of San Francisco's sewer system and were not included in this evaluation. There are also several private sewer systems within the NSMCSD boundaries that are also not explicitly included in this study, although flows from those systems are accounted for in the capacity evaluation. A map of the study area is shown in **Figure 1**.

1.2 Scope

To meet the overall objectives of this evaluation, the following tasks were performed:

1. Development of wastewater flow projections for the District's collection system using up-to-date population, water use, and land use information, and flow monitoring data collected as part of this study;
2. Development of a hydraulic model of the trunk sewer system using a commercially available hydraulic modeling software;
3. Identification of existing capacity deficiencies and future capacity requirements using the calibrated hydraulic model;
4. Development of a phased CIP including budget estimates for implementing the needed capacity improvements.

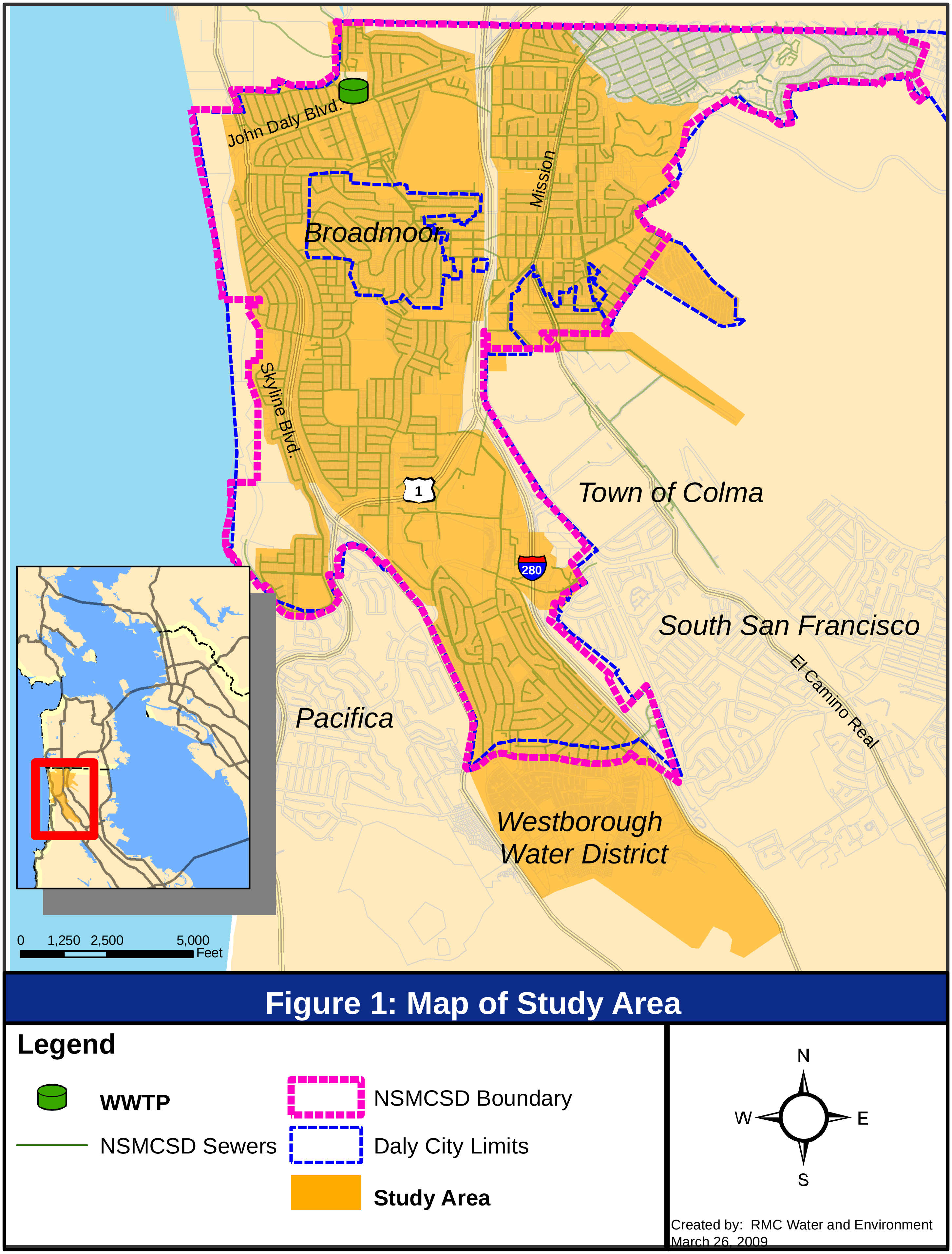

Figure 1: Map of Study Area

2 Planning Scenarios

The first step in assessing sewer capacity is to define the spatial distribution and magnitude of residential, commercial, institutional, and industrial land uses that generate sewer flows. The second step is to make a determination about how these characteristics may change over time. This section describes current population and land use as well as the future changes in residential and commercial developments and the impact these changes will have on future wastewater flows.

This Collection System Capacity Evaluation considers two planning scenarios: Existing and Future. For the purposes of this analysis, the Existing Scenario is considered to be 2008. Population estimates are based on 2000 Census data adjusted to include residential development that has occurred since 2000. The Future Scenario represents (approximately) 2030 conditions and incorporates new residential and commercial projects as well as redevelopment projects that may occur in the next 20 years.

2.1 Land Use

Daly City consists of a range of development types ranging from low density, single family homes to high density, multi-family developments, as well as open spaces, commercial and industrial uses. Data about existing land uses by parcel was available from the City's Geographic Information System (GIS) database.

Land use GIS data was primarily used to identify residential and non-residential developments. To that end, all land uses were simplified into these two categories, Residential and Non-Residential, as shown in **Table 1** and **Figure 2**. The delineation of residential and non-residential land-uses was also verified using recent aerial photographs. This distinction was necessary in order to match individual parcels with a methodology for determining base wastewater flows. Wastewater flows for Residential parcels were determined based on population, whereas historic water billing data was used to determine flows from Non-Residential parcels (this is discussed in detail in Section 4). Existing mixed-use developments were designated as non-residential, and therefore flows for these types of developments were developed based on water-billing data. Using billing data to estimate flows for mixed use developments ensured that both residential and commercial flow components were accounted for.

Table 1: Zoned Land Uses

Zone Code	Zone Description	Category
BC	BART Commercial	Non-Residential
BOC	BART Office/Commercial	Non-Residential
BRM	BRM BART Residential/Multi-Family	Residential
C1	Light Commercial	Non-Residential
C2	Heavy Commercial	Non-Residential
CEM	Cemetery	Non-Residential
CO	Office - Commercial	Non-Residential
ID	Interim	Non-Residential
M	Industrial	Non-Residential
OS	Open Space	Non-Residential
PD	Planned Development	Non-Residential
Pre-PD	Pre-Planned Development	Non-Residential
R1	Single Family	Residential
R1A	Single Family/Duplex	Residential
R2	Two Family	Residential
R2A	Duplex/Single Family	Residential
R3	Multiple Family	Residential
S1	Design Rev (Overlay)	Non-Residential

2.1.1 Existing Population

The existing population in the study area was determined based on 2000 Census block data. Census blocks are generally small, with an average size of about 8 acres per block within Daly City, and therefore provided a very good resolution for distributing existing population. Distributed population was increased to 2008 levels by incorporating additional population that has resulted from the construction of new residential development since 2000.

Existing population was disaggregated from census blocks to individual parcels using the following methodology:

- Year 2000 census block population was distributed to existing residential parcels within a given census block in proportion to parcel area.
- The year 2000 population was increased to the year 2008 population based on information about the number of dwelling units constructed on parcels that have had additional development since 2000. Newly constructed developments and the number of new dwelling units per development were provided by the Daly City Planning Department and are shown in **Table 2**. A factor of 3.36 persons/dwelling unit was used to calculate “new” population (based on the average household population for Daly City from the California Department of Finance, January 2008)
- Distributed population was aggregated to sewersheds for use in estimating residential sewer loads, as discussed in Section 4.

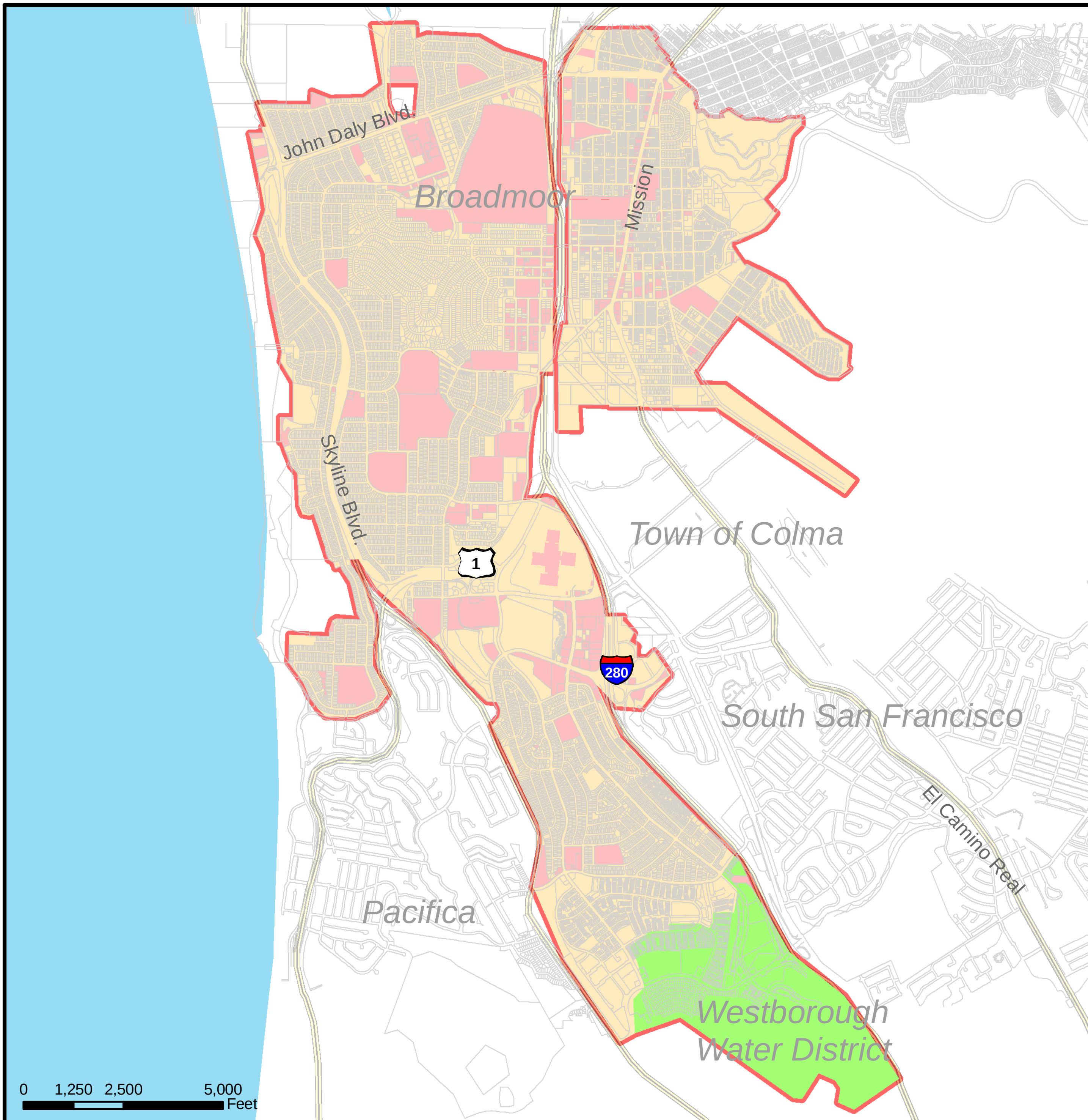

Figure 2: Residential and Non-Residential Areas

Legend

Wastewater Flows Based on Billing Data (Primarily Commercial)

Wastewater Flows Based on Population (Primarily Residential)

Wastewater Flows Based on Flow Monitoring Data

Study Area

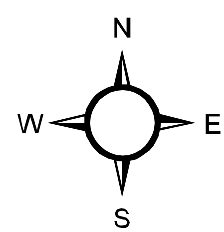

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Table 2: New Developments Constructed since 2000

Development Name	Approximate Location	Dwelling Units
Serravista Estates	103 - 177 Serravista Avenue	45
Hillcrest Gardens	35 Hillcrest Boulevard	40
Villa Lausanne	25 - 49 Lausanne Avenue	10
Lisbon Avenue homes	Lisbon Avenue	8
San Diego @ Delong	San Diego Street	5
Habitat for Humanity Multifamily	7555 Mission Street	36
La Terraza Apartments	7800 El Camino	153

2.2 Future Development

The majority of Daly City is developed; therefore, rezoning of large areas is unlikely. Even so, the City has several planned development projects, both commercial and residential, several of which consist of redevelopment of underutilized parcels. These future development projects have been identified by the City's Planning Department and are summarized in **Table 3** and their locations shown in **Figure 3** (based on "ID" number). This table also shows the expected timeframe for when these projects are projected to be constructed.

Table 3: Future Developments

ID	Development Name	Type[1]	Time (yrs)	Approximate Address	Dwelling Units	Commercial Area (sq. ft.)
1	Landmark Daly City	MF	0-1	6501 Mission Street	95	0
2	Mission/Westlake Mixed Use	MU	1-5	6800 Mission Street	36	3,000
3	Station Avenue Apartments	APT	1-5	130 Station Avenue	15	0
4	Habitat for Humanity Multifamily	MF	1-5	7555 Mission Street	36	0
5	Monarch Village (Assisted Living)	APT	1-5	165 Pierce Street	208	0
6	Serramonte Condominiums	MF	1-5	526 Serramonte Road	200	0
7	WestLake Shopping Center Mixed Use	APT	5-10	402 Westlake Ctr	50	0
8	Hotel	HTL	5-10	1901 Junipero Serra	350	0
9	New Commercial/Retail	CI	5-10	508 Westlake Drive	0	280,000
10	Gigli properties Redevelopment	MF	5-10	6824 Mission Street	31	0
11	Smith Property	MF	5-10	Chelsea Court	80	0
12	New Commercial/Retail	CI	5-10	6440 Mission Street	0	70,000
13	E. Market/First Street	MF	5-10	20th East Market	30	0
14	Bryant Street Apartments	APT	5-10	1698 Bryant Street	60	0
15	Hill Street Maintenance Surplus Site	MF	5-10	23 Hill Street	75	0
16	Bowling Alley + Retail Space	CI	5-10	297 San Pedro Road	0	150,000
17	Columbus Surplus Site	MF	5-10	60 Christopher Court	175	0
18	Zita/Eastmoor Townhomes	MF	5-10	211 Eastmoor Avenue	17	0
19	Serramonte Del Rey	MF	5-10	808 Campus Drive	175	0
20	Serramonte Hotel	HTL	5-10	527 Serramonte Road	150	0
21	Horse Stable Redevelopment	MF	10-15	Olympic Way	200	0
22	BART Mixed Use	MU	10-15	488 John Daly St.	125	10,000
23	Brunswick Street	MF	10-15	4619 Brunswick Street	50	0
24	St. Francis Court Condominiums	MF	10-15	St. Francis Boulevard	36	0
25	St. Francis Square	APT	10-15	11 St. Francis Square	20	0
26	Serramonte Shopping Center Residential	MU	10-15	532 Serramonte Blvd.	150	250,000
27	Samtrans Redevelopment	CI	15-20	3116 Junipero Serra	0	150,000

1. MF = Multi-Family, MU = Mixed-Use, APT = Apartment, CI = Commercial/Industrial, HTL = Hotel

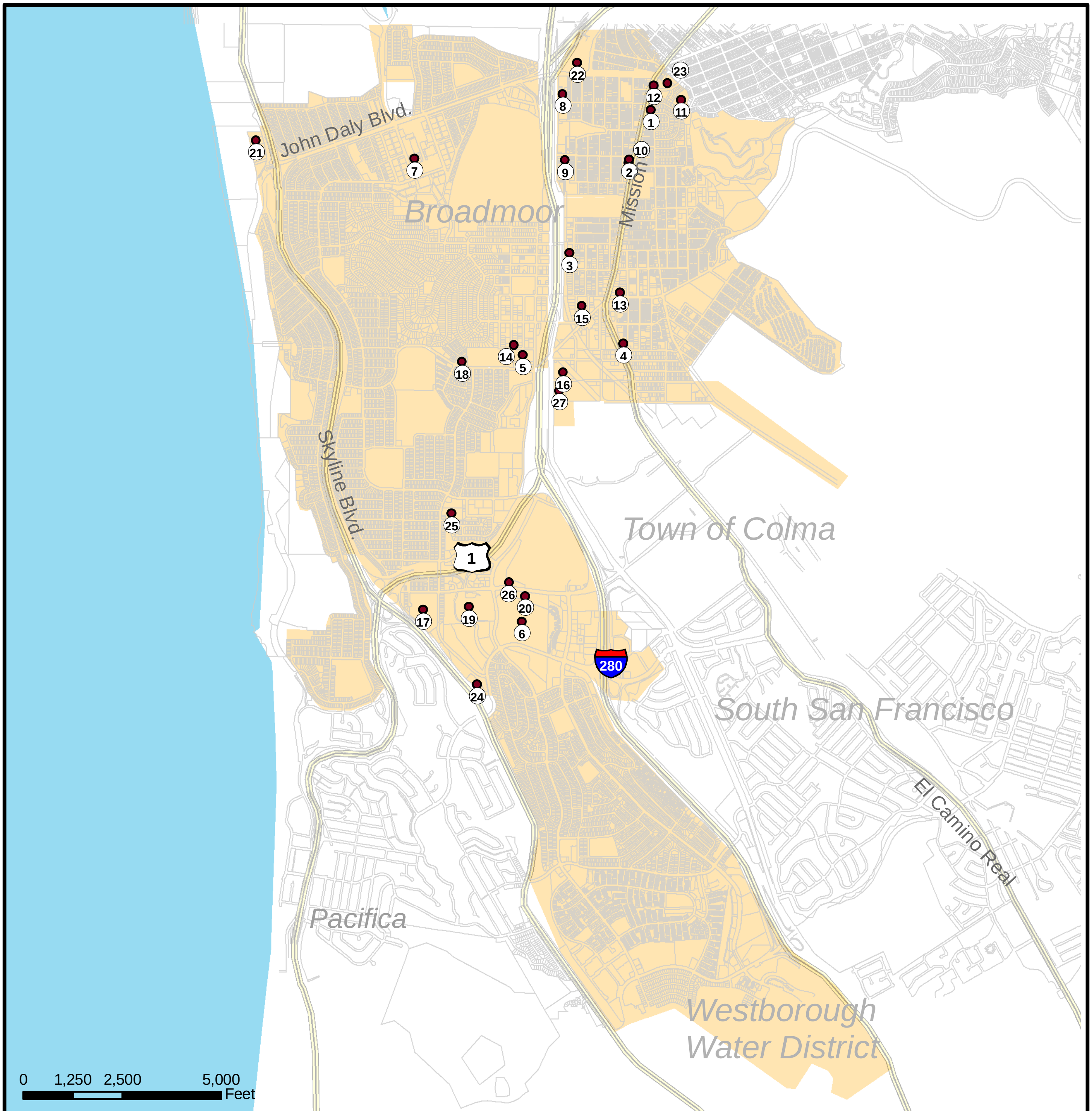

Figure 3: Location of Potential Future Developments

Legend

- Potential Future Development ID (see Table 3 for more information)
- Study Area

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3 Hydraulic Model

A hydraulic model of the District's service area and major sewers was the primary analytical tool used in this study to estimate flows and to assess sewer and lift station capacities. This section describes the modeling software and methodologies. Topics include the selection of the major sewers included in the model, the development and validation of the required data on those major sewers, and the delineation of sewersheds (areas tributary to the model system) used to define load inputs into the model.

3.1 Model Development Overview

RMC utilized InfoWorks™ from Wallingford Software for this Collection System Capacity Evaluation. Several steps are involved in the model-building and application process in order to ensure that the model will accurately predict existing and future flows and capacity limitations. This methodology is summarized below. Additional details are presented in subsequent sections of this TM.

- Define the model network, extract the data on each modeled pipe segment and manhole required for modeling (spatial coordinates; manhole identifiers, rim and invert elevations; pipe invert elevations, diameters, and lengths), validate all data (i.e., check for and correct missing or erroneous data values), and further develop data as necessary based on the validation.
- Divide the service area into sewersheds, consisting of areas of unmodeled pipes tributary to modeled manholes.
- Compile information on land uses, population, and water consumption for both existing and future conditions using Census data, water billing data, and information provided by the City's planning department, as described in 2.2 and Section 4.
- Develop unit flow factors and diurnal patterns to estimate base wastewater flow loads into the modeled system from each sewershed.
- Select representative dry weather days from flow monitoring periods and compare model results to metered flows. Calibrate the model by refining unit flow factors and 24-hour diurnal flow patterns to match the observed flow volumes and peaks at each meter.
- Select wet weather events for use in wet weather flow calibration. For the selected events, compare model results to meter data to develop and calibrate model wet weather parameters governing the volume and response pattern of rainfall-dependent infiltration/inflow (RDI/I) into the modeled system.
- Identify design storm used for analyzing and determining the required capacity of sewer system facilities.
- Determine appropriate dry and wet weather design flow and hydraulic capacity analysis criteria.
- Run the calibrated model with the design storm to identify sewer reaches having capacity deficiencies under existing and future design flows.
- Develop and test alternative solutions consisting of flow diversions, parallel pipes, and/or larger replacement pipes.

3.2 Modeled Sewers and Sewersheds

The modeled collection system consists of links and nodes, which represent the major pipes, manholes, pumps, and lift station wet wells. The service area is divided into subareas (sewersheds), each of which defines the tributary area to a node on the modeled system. The following subsections discuss the modeled system and sewersheds.

3.2.1 Modeled Sewers and Lift stations

The modeled system represents the trunk sewer system, the network of larger diameter pipes that comprises the "backbone" of the system. The modeled network includes 35 miles of pipelines (29 percent of the portion of the NSMCSD collection system that is tributary to the NSMCSD WWTP), five NSMCSD sewer lift stations (Belcrest, Colma, El Portal, Hickey, and Skyline) and one in WWD (Rowntree). This modeled network includes nearly all of the District's 10-inch and larger pipes as well as some 6- and 8-inch pipes. Private sewer systems and systems outside of NSMCSD (e.g., WWD) were not modeled. **Figure 4** shows the entire modeled system.

The District's sewer system contains numerous potential flow splits (manholes with multiple outlet pipes), creating several possible alternative flow paths. Understanding these flow splits was important in delineating a trunk network for the model that would accurately portray actual field conditions. Sufficient data was typically given by the sewer atlas in the form of invert elevations and preferential flow arrows to correctly route critical flow splits. When there was not, manhole surveys were conducted by District field crews. These field survey sheets are included in **Appendix A**.

The six lift stations included in the model were modeled based on information provided by the District. This information included pump curves, wet well dimensions, on/off levels, and other pertinent pump information. Manufacturer's pump curves provided by the District are attached in **Appendix B**. Constant speed pumps were modeled using a head-discharge relationship based on the pump curves and the District's wet well set-points. Most of the modeled pumps are constant speed pumps, with the exception of two: 1) Rowntree and 2) El Portal. El Portal was modeled as a constant speed pump to more accurately model its maximum flow capacity. Understanding the maximum flow capacity of this pump was important because the modeling analysis described in Section 5 revealed that under wet weather flows, this pump operates at near or at maximum capacity. This is contrast to the other VFD pump, Rowntree, which has sufficient capacity during wet weather flows. Therefore, Rowntree was modeled to gradually pump out more flow as wet well levels increase, up to the estimated capacity of the lift station. In effect, this more accurately mimics a variable frequency motor by increasing the pump rate as flows increase and This also dampens fluctuations in wet well levels. Total lift station capacities for Rowntree were estimated outside of the model based on the manufacturer's pump curves and approximate system curve.

Lift station specifications are given in **Table 4**, including the theoretical firm capacity of each lift station (firm capacity is the capacity with one pump not in use). These capacities were estimated for El Portal, Colma, Belcrest, and Skyline lift stations as part of the 1989 Infiltration/Inflow Evaluation in the 1993 Master Plan. The capacities for these pump stations, with the addition of Rowntree, have been re-evaluated in Table 4 using the same methodology but with pump curves and design points provided by the District for this analysis. This information is discussed in more detail in TM 2: "Review of 1993 Collection System Master Plan".

The "modeled" firm capacity of each lift station was calculated by the hydraulic model and is a function of pump curves, head loss through the force main, the water surface elevation in the wet well, and the number of pumps in operation. Maximum capacities (all pumps in operation) as calculated by the hydraulic model are described in Section 5.4.

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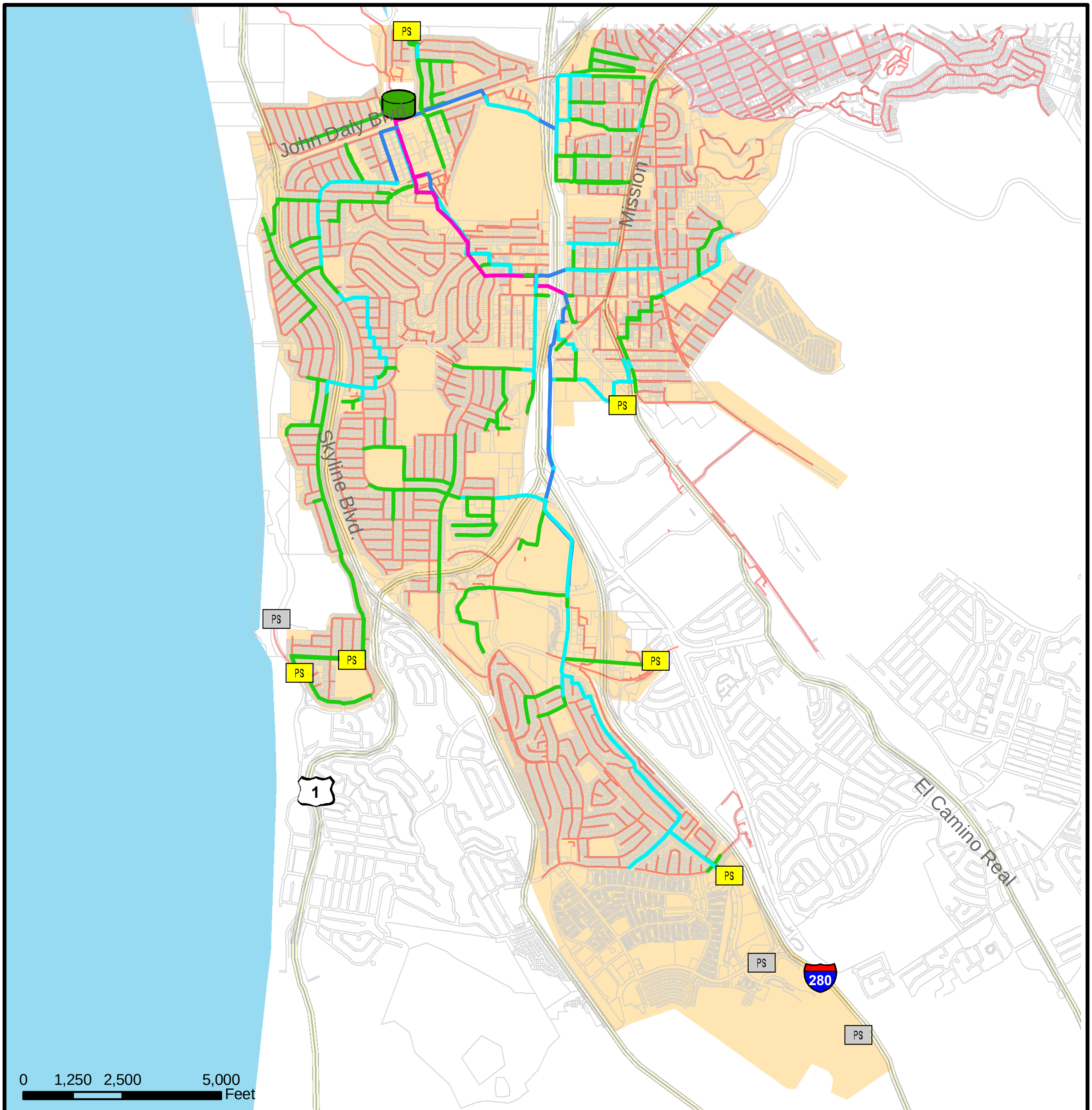

Figure 4: Modeled Sewer System

Legend

PS Unmodeled Lift Stations

PS Modeled Lift Stations

WastewaterTreatment Plant

Study Area

Modeled System (Diameter):

6"-10"

12"-18"

21"-24"

>24"

Unmodeled Pipes

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Table 4: Theoretical Lift station Firm Capacities

Lift station	No. of Pumps	Design Capacity of each Pump (mgd)	Efficiency Factor[1]	Calculated Firm Capacity (mgd)[2]	Modeled Firm Capacity (mgd)	Speed Type
Belcrest	2[3]	0.43[3]	70%	0.30	0.36	Constant
Colma	3	1.30	75%	1.94	2.45	Constant
El Portal	2	0.58	80%	0.46	0.56	Variable
Hickey	2	0.94	80%	0.75	1.10	Constant
Rowntree	3	1.87	75%	2.81	2.81[4]	Variable
Skyline	2[3]	0.43[3]	70%	0.30	0.38	Constant

1. Efficiency Factor is estimated in accordance with the 1989 Infiltration/Inflow Evaluation based on the number of active pumps.
2. Firm capacities have been calculated here using the same methodology as used in the 1993 Master Plan. "Calculated firm capacity" is the product of firm capacity (design flow with one pump out of service) and the Efficiency Factor. Note that the firm capacities shown in this table for Belcrest and Skyline lift stations are lower than what is given in the 1993 Master Plan. This is because the 1993 master plan assumed 4 pumps total, but there are effectively only 2 pumps trains in operation – firm capacity should be calculated with one train offline (2 pumps out of service).
3. Belcrest and Skyline consist of two sets of two pumps in series, for a total of 4 pumps at each lift station. Each set is configured for 0.43 MGD.
4. The firm capacity of Rowntree was assumed equal to the calculated firm capacity.

3.2.2 Modeled Sewersheds

Sewersheds are used in the model to define loads to the modeled system, and represent flow from unmodeled sewers tributary to a modeled manhole. Sewershed parameters define flows entering the collection system, including population-driven sanitary flow, commercial and industrial flow, and infiltration/inflow (I/I). Sewersheds were defined by tracing unmodeled sewers upstream from a modeled node and delineating the drainage area around these unmodeled sewers. As with the selection of pipes to include in the model, the sewershed delineation process takes into account potential flow splits in the sewer system.

The model includes 147 sewersheds ranging in size from just over two acres to nearly 170 acres, with a median size of about 20 acres, not including the areas that drain into the NSMCSD sewer system from WWD. Parts of the WWD that are tributary to NSMCSD's sewer system were represented by three, relatively large sewersheds, ranging from 58 to 424 acres. **Figure 5** shows the modeled sewersheds.

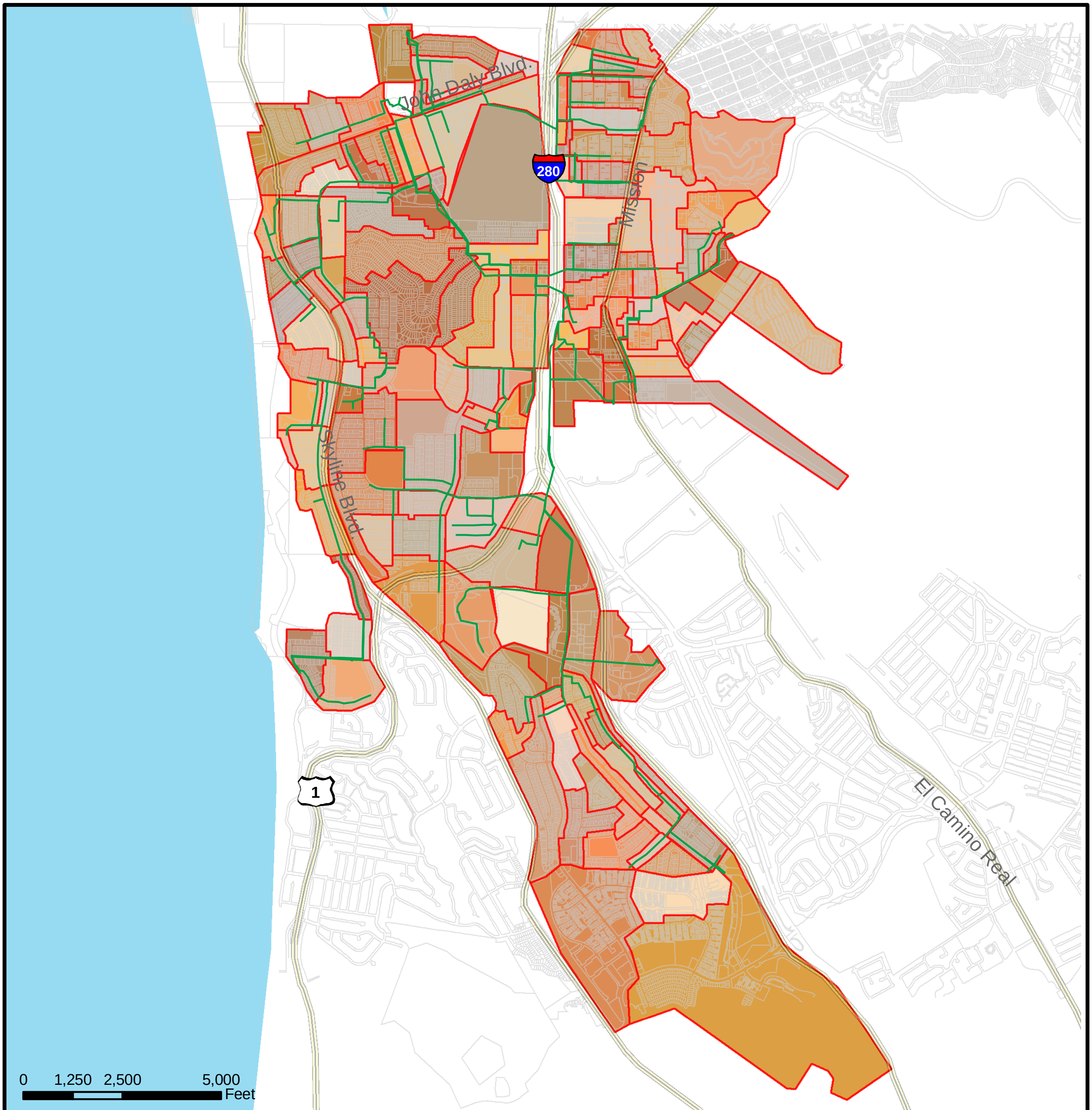

Figure 5: Sewershed Boundaries

Legend

Sewersheds

Modeled Sewer System

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3.2.3 Model Network Construction and Validation

Network data for the model were provided by the District as GIS files with manhole and pipe attribute data (manhole rim and invert elevations; pipe diameters and lengths). With the exception of the WWD, these GIS files formed the basis for constructing the model network and sewersheds. Information included in the GIS was supplemented with data from record drawings, the District's CAD maps, and survey data where the GIS was incomplete. Information about the sewer network in WWD was derived from WWD sewer maps. The model construction and validation process included the following steps:

- The modeled network connectivity was checked and corrected where necessary (i.e., it was verified that correct upstream/downstream manholes were identified for each pipe and there were no missing links or isolated manholes in the network).
- Manholes and pipes with missing data (diameter, inverts, or rim elevations) were identified.
- Profiles for each series of pipe segments in the modeled network were reviewed to visually check for suspect data. Examples of suspect data include downstream pipes smaller than upstream pipes, pipe crowns above ground level, negative pipe slopes, or abrupt steps up or down in invert elevations.
- Missing and suspect data were interpolated where appropriate, or obtained from as-built drawings or manhole surveys.
- As-built and other information provided by the District was used to manually input details on the modeled lift stations. Information required included wet well elevations and dimensions, number of pumps, pump capacities, on/off levels, and head-discharge curves.

Missing or inconsistent invert elevations, rim elevations, and pipe diameters were the most significant data issue in constructing and validating the model. Initially, about 40 percent of all pipes were associated with erroneous or missing upstream invert elevation, downstream invert elevation, or pipe diameter. Several of these issues stemmed from the original GIS data structure. Invert elevations were associated with manholes with no indication as to which inverts were related to individual pipes. This presented a challenge where three or more pipes intersected at a manhole. Most of these situations were remedied by either referencing the sewer CAD maps or by engineering judgment. Where the relative invert elevations determined an important flow split, field personal measured the actual invert depth. The invert elevation was then calculated based on invert depth and known rim elevations. A copy of pertinent field notes can be found in Appendix A.

Another validation hurdle came as a result of data accuracy. All invert and rim elevations are rounded to the nearest foot in both the GIS data and CAD maps. This relatively coarse accuracy resulted in several flat pipes or pipes with negative slopes. This was a particular problem in pipe segments with mild slope (i.e. where the difference between upstream and downstream invert elevations was less than one foot). Where these situations occurred, pipes slopes shown in the CAD maps were used to refine invert elevations, or inverts were inferred using known inverts coupled with upstream and downstream pipe slopes.

Approximately 10 percent of the manholes were missing rim elevations. Data were obtained from as-built drawings and CAD maps, where available, to fill in the missing information. Where not available, rim elevations were inferred based on surrounding ground elevations.

The source of all rim and invert elevations in the model, whether obtained from the GIS, as-built drawings, manhole survey, or interpolation, are documented in the model using data "flags". The District should supplement this database with field surveys prior to design activities, especially where data has been flagged as "inferred" or "interpolated".

4 Existing and Future Wastewater Flows

This section describes different wastewater flow components and the development of wastewater flow estimates. Population and water consumption data provided the basis for estimating average base wastewater flows. Flow monitoring and rainfall data were then used to calibrate the sewer model for both dry and wet weather flow conditions.

4.1 Wastewater Flow Components

Wastewater flows typically include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I). BWF represents the sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system.

GWI is groundwater that infiltrates into the sewer through defects in pipes and manholes. GWI is typically seasonal in nature and remains relatively constant during specific periods of the year. RDI/I is storm water inflow and infiltration that enter the system in direct response to rainfall events. RDI/I can occur through direct connections such as holes in manhole covers or illegally connected roof leaders or area drains, or through defects in sewer pipes, manholes, and service laterals. RDI/I typically results in short-term peak flows that recede quickly after the rainfall ends. Dry weather flow (DWF) consists of BWF plus GWI, while wet weather flow (WWF) adds the RDI/I component. These three flow components are illustrated conceptually in **Figure 6.**

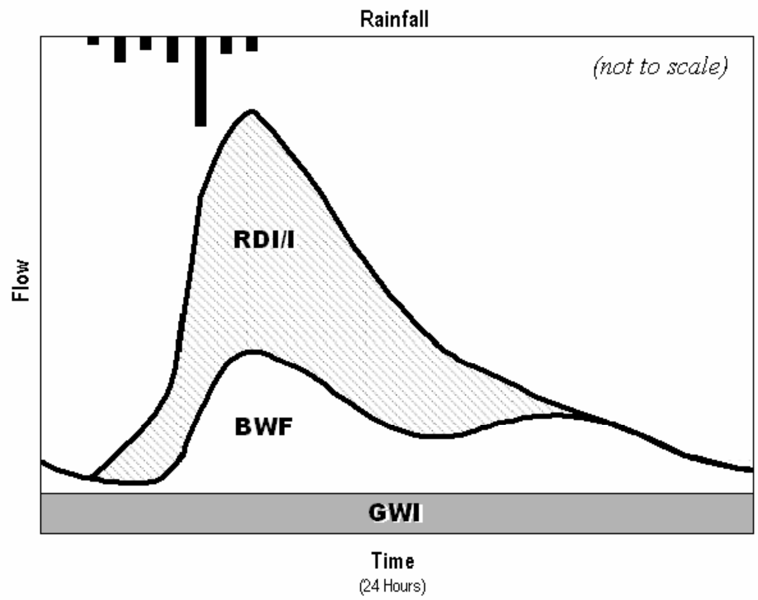

Figure 6: Wastewater Flow Components

4.2 Flow Estimating Methodology

As discussed in Section 3.2, sewersheds are areas that represent flow from unmodeled sewers tributary to a modeled manhole. All model loads, including BWF, GWI, and RDI/I are estimated at the sewershed level for input into the model. Flow monitoring data collected throughout the system are then compared to model flows at certain points throughout the system to refine the magnitude and timing of model loads. The following sub-sections describe the development and assumptions for each load component.

Subsequent sections describe the flow monitoring and model calibration, which contributed to the development and refinement of each flow component described below.

4.2.1 Base Wastewater Flow

Existing base wastewater flow (BWF) entering the modeled system was estimated based on population and water use data. A per-capita unit flow rate was applied to the population within each sewershed to define the average BWF from residential sources, while average winter water consumption was used to define the load from non-residential sources. An exception to this methodology was made for inflows from Westborough Water District. This is discussed in more detail in the following subsections, along with a discussion about how current and future residential and commercial BWF was developed.

Existing Residential Loads

Existing residential loads were determined by applying a per-capita flow rate to sewershed populations. An average per capita flow of 57 gallons per capita per day (gpcd) was used. This per capita flow is similar to wastewater flows observed in other Bay Area cities and is consistent with the per capita water use for Daly City presented in a 2007 Report prepared by the Sierra Club, "Improving Water Conservation: Opportunities for San Francisco Bay Area Water Supply Agencies". This factor was also confirmed to be appropriate for Daly City through comparisons of model flows to dry weather flow meter data, as discussed in Section 4.4, Model Calibration.

Existing Commercial Loads

Existing non-residential flows were based on winter water usage for non-residential customers. Winter water use is considered a reasonable approximation of wastewater flow because outdoor water use during the winter is generally minimal. Water consumption from the District's water billing database was averaged for each non-residential customer for winter months. More specifically, the following billing periods were used to determine the existing wastewater load:

- December through March 2006/2007
- December through March 2007/2008
- November through January 2008/2009

To determine the location of water usage/sewer loads, nonresidential water customers were linked to parcels based on the Assessor Parcel Number (APN). Non-residential water usage was then aggregated to sewersheds.

Future Residential Loads

As discussed in Section 2, future residential developments were indentified and defined in terms of the type of development (multi-family, mixed-used, apartment, etc.) and the number of dwelling units (DUs). Typical flow factors, as used by other agencies in the Bay Area, were then applied to the number of dwelling units. These factors are shown in **Table 5**.

Several future residential and mixed-use developments were identified to be redevelopment projects which will replace existing commercial uses. For these types of projects, the calculated new sewer loads replaced the existing sewer loads in the model wherever a specific APN could be identified for the redevelopment project. If a specific APN could not be identified, new loads were added to existing commercial loads; however, there were very few of these occurrences and the impact on sewershed flows is minimal.

Table 5: Flow Factors for Future Residential Developments

Type of Future Residential Development	Factor	Units
Apartment[1]	170	gpd/DU
Multi-Family Residential[2]	220	gpd/DU
Mixed Use (residential portion)	170	gpd/DU
Single Family Residential	240	gpd/DU

1. More than five dwelling units per building
2. Town-homes, condominiums, duplexes, typically two to four dwelling units per building

Future Commercial Loads

As discussed in Section 2, future non-residential developments were indentified and defined in terms of the anticipated building size for commercial, light industrial, and office developments and the number of rooms for hotel developments. Typical loads were then applied based on these characteristics, as shown in **Table 6**.

Similar to how redevelopment projects were handled for residential projects, some future commercial loads will replace existing loads. Most of these projects are in the form of mixed-use developments, which have commercial and residential components. Existing loads were replaced only if a specific APN had been identified for a commercial redevelopment project.

Table 6: Flow Factors for Future Commercial Developments

Type of Future Non-Residential Development	Factor (gpd)	Unit
Commercial/Retail/Office	0.10	bldg. sq. ft.
Hotel	100	room

Inflows from Westborough Water District

All flows from WWD enter the NSMCSD system through the Rowntree lift station. Most of the flow into Rowntree comes from the Westborough lift station force main, which discharges directly to the Rowntree lift station wet well. The average dry weather flow from the Westborough lift station was modeled as 0.15 MGD, based on flow monitoring on the Westborough force main conducted by the District in September of 2008.

Besides the Westborough force main, some Rowntree lift station inflows originate within a relatively small tributary sewer network, most of which lies within WWD boundaries. Water billing data was not available for this area, and based on inspection of aerial photographs, there did not appear to be significant commercial development. For residential wastewater loads, the same methodology was used here as elsewhere in the model, that is, census data was used in conjunction with a per capita flow factor to estimate flows. An important addition to residential population in this area was the San Francisco County Jail (#5 and #7). The inmate and staff population of this facility (estimated to average 1,000 people[1]) was added to census population to estimate wastewater flows.

[1] Based on conversations with the San Francisco Sheriff's Department Public Information Officer (Susan Fahey) on March 16th, 2009.

4.2.2 Diurnal Profiles

The diurnal profile, or change in the BWF throughout the day, must also be defined to accurately portray wastewater flows. Separate sets of diurnal profiles were developed for weekdays and weekends. Profiles were based on monitored flows throughout the District and are similar to those observed in other cities. Different diurnal profiles were developed to represent residential and non-residential areas, and these profiles are shown in **Figure 7** and **Figure 8**, respectively. The diurnal flow factor, as shown on the graphs, represents the ratio of the flow at a particular hour of the day to the average daily flow. As shown in Figure 7, peak flows in residential areas typically occur in the morning hours, with the morning peak occurring later on weekends than on weekdays. In non-residential areas, the peak flow generally occurs over an extended period during the middle of the day.

A unique diurnal profile was used to model inflow from the Westborough lift station, based on the District's dry weather flow monitoring data. This profile is shown in **Figure 9**.

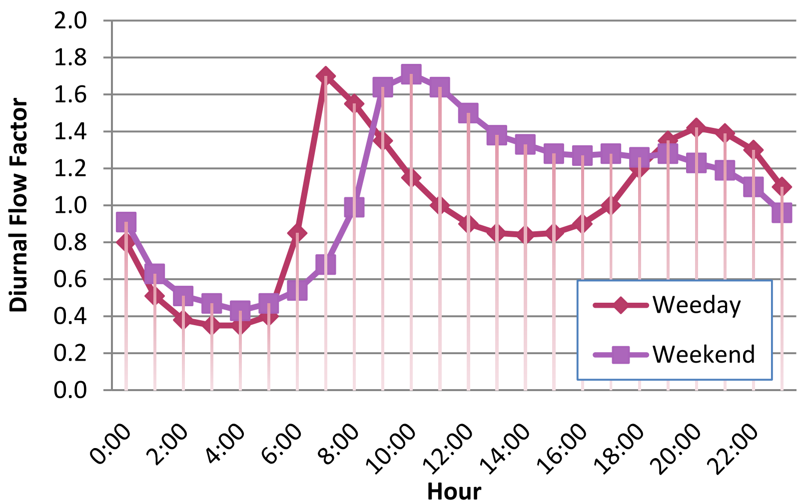

Figure 7: Residential Diurnal Profile

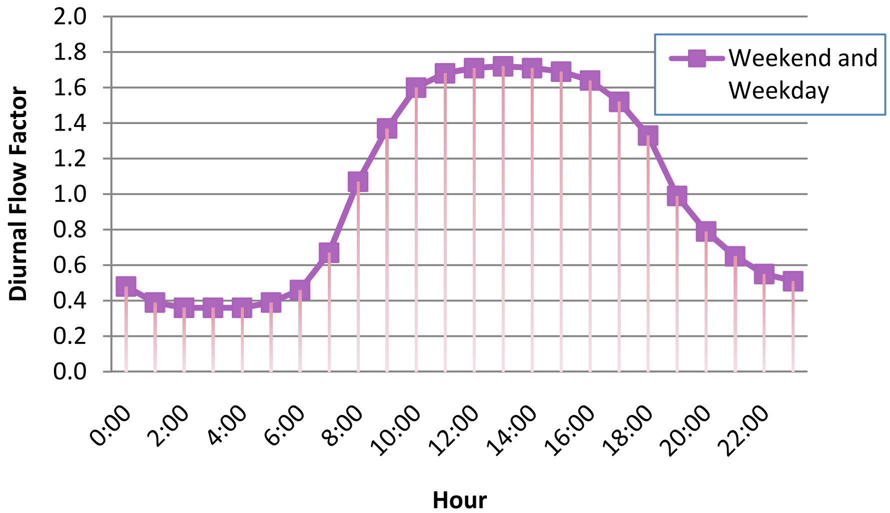

Figure 8: Non-Residential Diurnal Profile

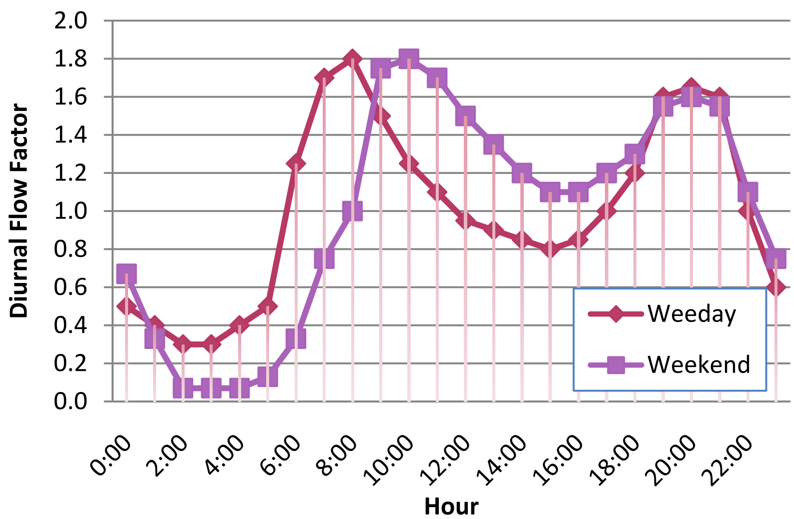

Figure 9: Westborough Diurnal Profile

4.2.3 Groundwater Infiltration

Groundwater Infiltration (GWI) represents infiltration into the sewer system during non-rainfall periods and is typically applied as a constant load in addition to the BWF. The amount of GWI to add in any particular area is determined during model calibration by comparing the modeled base flows to actual observed flows at points in the system where flow meter data are available. It can be difficult to accurately assess the absolute amount of GWI, as other model assumptions such as per capita flow rate will affect the amount of flow attributed to GWI. The 1989 Infiltration/Inflow Evaluation noted high groundwater in the northern part of the District, but based on comparison of modeled dry weather (non-rainfall period) flow and flow monitoring data, groundwater infiltration was not determined to be a significant source of flow in the NSMCSD system. Therefore, no GWI load was applied to the model.

4.2.4 Rainfall-Dependent Infiltration and Inflow

RDI/I sewer loads are defined by the magnitude, shape, and timing of the RDI/I response. The magnitude of the RDI/I response is typically described by the percentage of the rainfall volume (the "R value") that enters the sewer system over a specified drainage area. The RDI/I hydrograph shape is defined by separating the total RDI/I hydrograph volume into components, representing different response times to rainfall (fast, medium, and slow). Each of the components has a specific time to peak (T) and recession coefficient (K). The R component percentages and T and K values are applied to each hour of rainfall to generate a "synthetic hydrograph" that approximates the volume and shape of the hydrograph from an actual observed event. These parameters, when applied to a different rainfall pattern (e.g., a "design storm", as discussed later in Section 5), can be used to estimate the RDI/I response to that particular rainfall event. **Figure 10** illustrates the three RDI/I components and the resulting total RDI/I hydrograph.

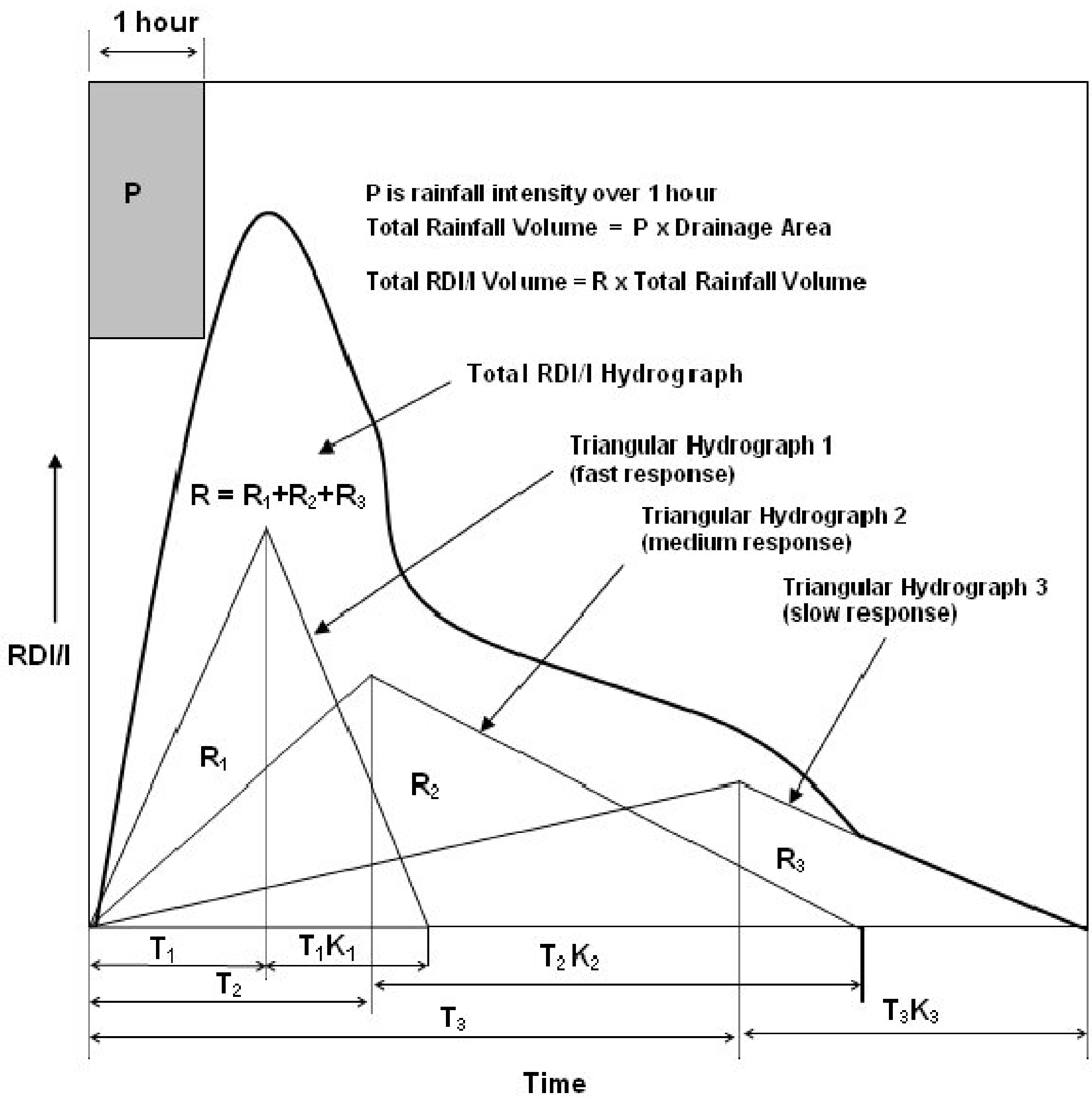

Figure 10: RDI/I Hydrograph Components

RDI/I loads to the model were determined based on comparison of flow monitoring data to model flows during rainfall events, as discussed in later sections of this TM. Although RDI/I unit hydrographs are applied for each sewershed, parameters governing the magnitude, shape, and timing of the RDI/I load are defined by areas draining to flow meters, usually made up of many sewersheds. Exceptions were made to this rule if the age of pipes within a sewershed were known to be the markedly different from other sewersheds that drained to the same flow meter. For example, the Point Pacific development drains to flow meter 2 (see Figure 11 and a discussion of flow monitoring program in Section 4.3), but because this area has a newer sewer system compared to most other sewersheds that drain to flow meter 2, it was assigned different RTK parameters. RDI/I parameters for unmetered areas are based on those for nearby metered areas with similar characteristics.

No distinction was made for areas tributary to flow meters located on parallel pipes when assigning RTK parameters, as their contributing upstream networks are intertwined. This was true for flow meters 4 and 5 and flow meters 8 and 9.

4.3 Flow Monitoring Program

Flow monitoring data provided the basis for comparing and refining model loads, described in Section 4.2, to better match actual flows measured in the field. Under contract to RMC, V&A Engineers conducted flow monitoring at eleven sites between December 2007 and March 2008. The District also contracted with V&A for additional flow monitoring of Avalon, Westborough, and Rowntree Lift stations in September of 2008. **Table 7** shows the flow monitoring locations for the sites monitored by V&A during the 2007/08 wet weather program. **Figure 11** shows the meter locations and incremental tributary areas. **Figure 12** provides a schematic of flow meters in the model. For more detailed information about the flow monitoring program and its results, refer to the TM 3A: “Flow Monitoring Results”.

Rainfall was also recorded at three locations by V&A during the winter of 2007/2008. These rain gauges were strategically located in the northern, central and southern part of the District. Rain gauges were assigned to most sewersheds using Thiessen Polygons (which is a method for assigning the closest gauge to each sewershed), however some assignments were modified during model calibration (described in Section 4.4) to more accurately reflect orographic effects. Rain gage locations and sewershed assignments are shown in **Figure 13**.

Table 7: Flow Monitoring Locations

Flow Meter	Location	Manhole No.	Nominal Pipe Dia. (in.)
FM1	John Daly Blvd. at WWTP	MH-C04-119	24
FM2	Mayfair Dr. at S. Mayfair Ave.	MH-D03-064	18
FM3	S. Mayfair Ave. betw. Crestwood & Lake Merced Blvd.	MH-C04-134	21
FM4	Park Plaza Drive, south of Coronado	MH-C05-154	30
FM5	Park Plaza Drive, south of Coronado	MH-C05-098	18
FM6	Upstream of F Street Lift Station	MH-E07-089	15
FM7	Southgate Ave. at Sullivan Ave.	MH-D08-014	15
FM8	Serramonte Center	MH-D09-007	21
FM9	Serramonte Center	MH-D09-034	15
FM10	Verducci Court, northeast of Gellert Blvd.	MH-E13-053	15
FM11	90th Street, east of Sullivan Ave	MH-D06-133	24

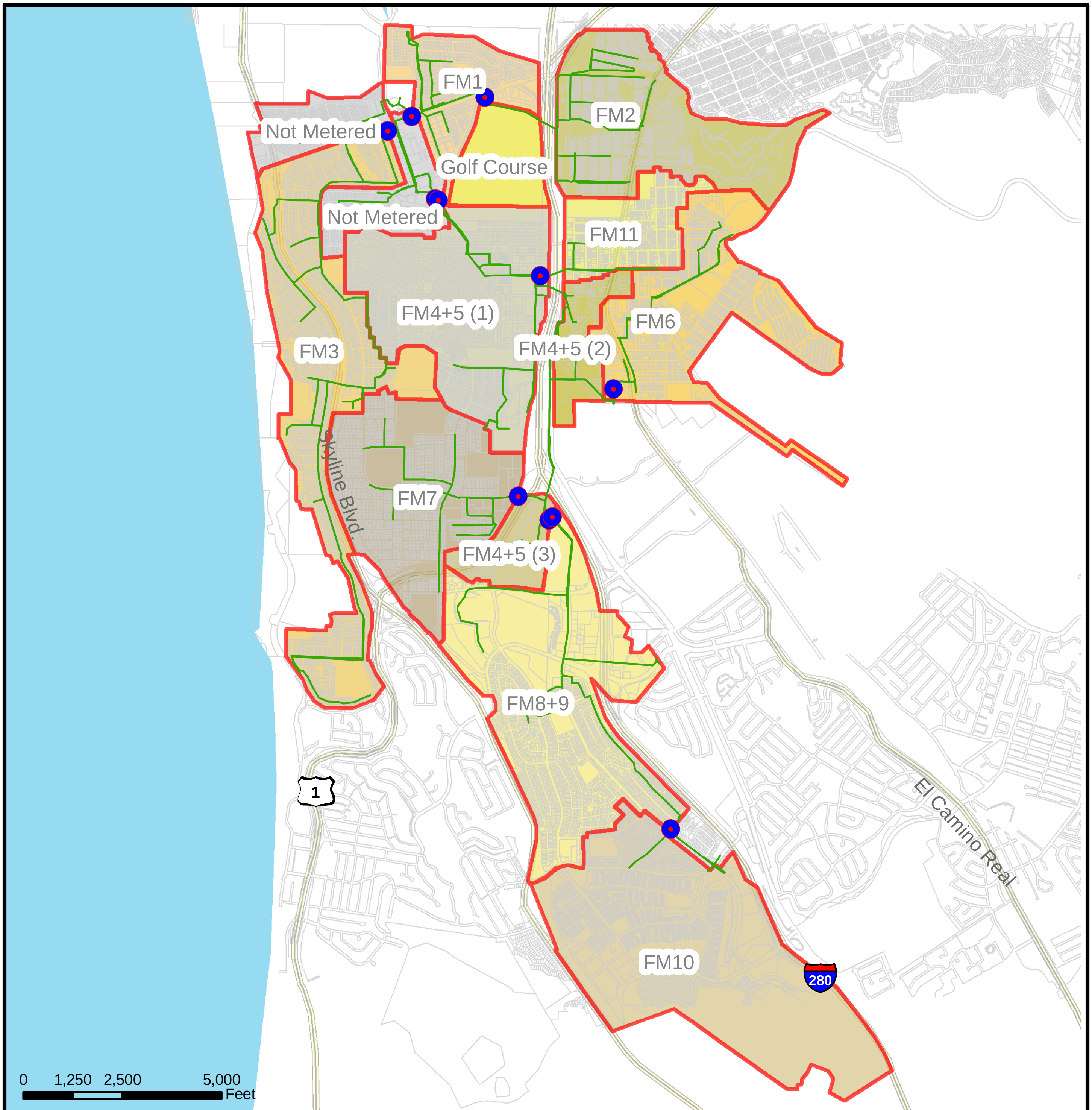

Figure 11: Flow Meter Locations and Tributary Locations

Legend

- 2007/2008 Flow Meters
- Modeled Sewer System
- Flow Meter Tributary Areas

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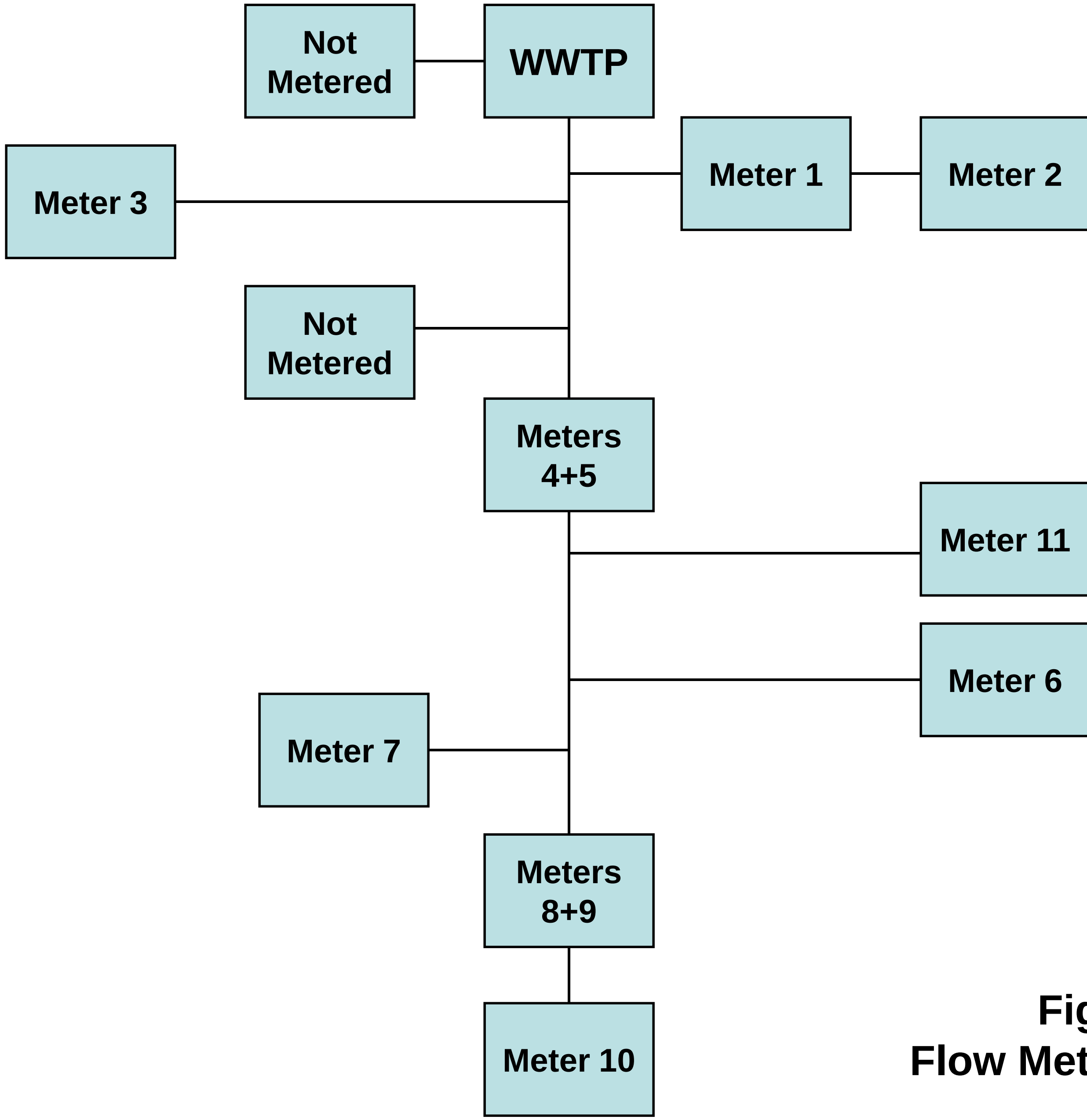

Figure 12
Flow Meter Schematic

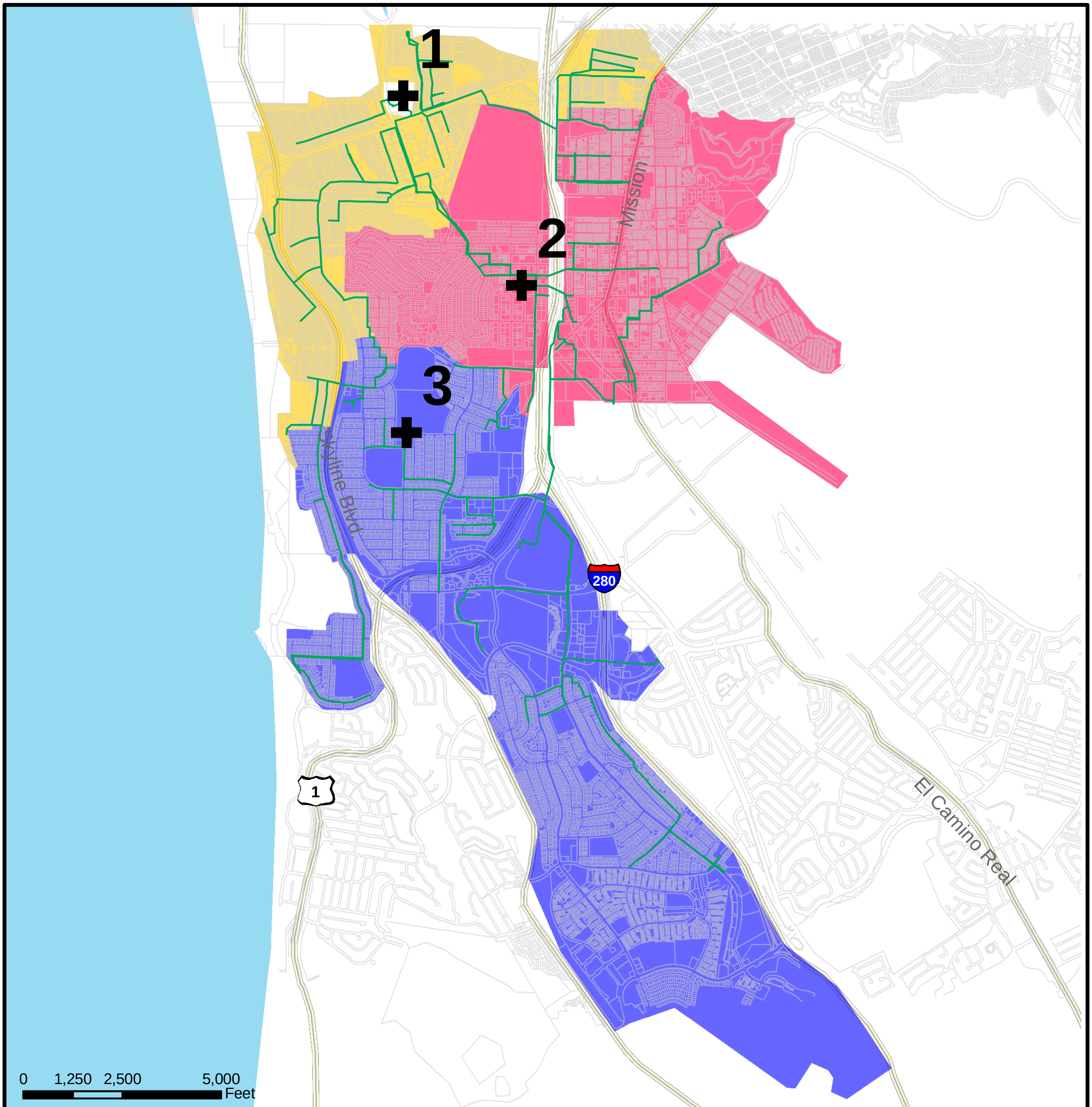

Figure 13: Rain Gauge Locations and Influence Areas

Legend

Rain Gauge Area 1

Rain Gauge Area 2

Rain Gauge Area 3

2007/2008 Rain Gauges

Modeled System

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4.4 Model Flow Calibration

Dry and wet weather model flows were calibrated by comparing model results to flow monitoring data for selected days during the 2007/08 flow monitoring period The following sub-sections describe the calibration of dry and wet weather flows.

4.4.1 Dry Weather Flow Calibration

Dry weather flow (DWF) calibration involves confirming the per-capita flow rates and 24-hour diurnal flow profiles in the model that result in the best match of modeled flows to monitored flows for a typical dry period.

Dry weather periods used for calibration are selected to have minimal preceding rainfall and to avoid major holidays. A dry weather flow calibration period was selected from the available flow monitoring period, between February 11th and 18th, 2008. This period included both weekdays and a weekend.

Overall, the dry weather calibration yielded reasonably good results. **Figure 14** shows an example comparison between model results and meter data for flow meter 9, a primarily residential area. In this figure, the red line shows the meter data, and the green line shows the model results. DWF calibration results for all meters are attached in **Appendix C**.

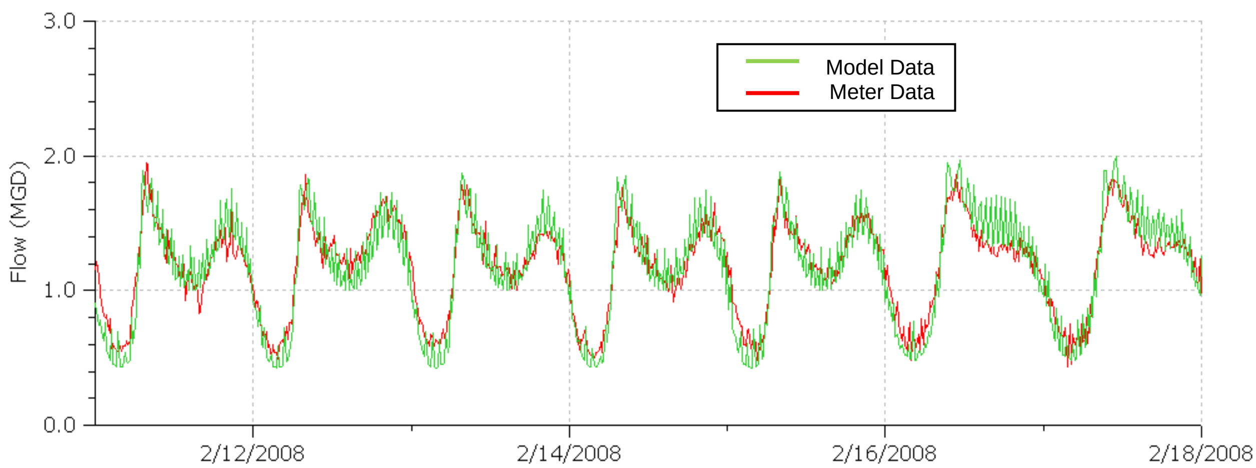

Figure 14: Example Dry Weather Flow Calibration Results, FM 9

Although DWF calibration was generally successful for most of the meters overall, modeled flows at flow meter 6 were higher than measured flows by about 33 percent. Potential reasons for this discrepancy were investigated, such as possible upstream flow splits or incorrect sewershed boundaries, or different types of land use (and therefore flow generation) characteristics. However, no obvious explanation was found. Since there is no justification for using lower unit flow rates in this area, and it is also possible that the meter may have read low, no adjustments to BWF unit flow factors for this area were made. For all of the other meters, the modeled flows matched the metered flows to within 10 percent or less.

4.4.2 Wet Weather Flow Calibration

Following completion of the dry weather calibration, the model was calibrated for wet weather conditions. Wet weather flows were calibrated first based on upstream meters. RDI/I parameters for the area draining to each upstream meter were determined, then calibration to downstream meters resulted in

RDI/I parameters for the area draining to the system between the upstream and downstream meters. The areas used for WWF calibration are shown in Figure 11 with the discussion of RDI/I loads under Section 4.2.4.

The January 25th, 2008 storm event was used as the primary calibration event for most flow meter areas, and the January 4th event was used for verification. **Figure 15** shows an example comparison between the measured and model response to the January 25th, 2008 storm for flow meter 9. In this figure, the red line shows the meter data, the green line shows the model results, and blue represents rainfall data. Not all meters exhibited this good a calibration for wet weather events, but the overall calibration is considered to be reasonably good. WWF calibration results for all meters are attached in **Appendix D**.

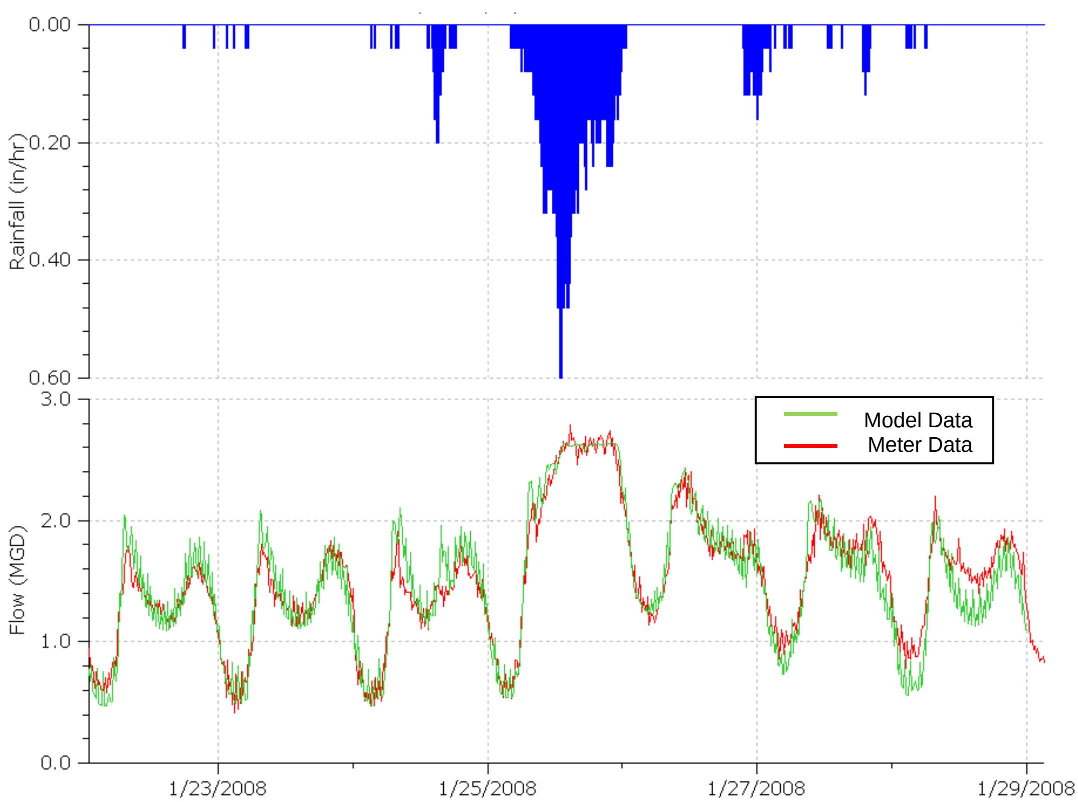

Figure 15: Example Wet Weather Calibration, January 25th, 2008 Storm, FM 9

5 Sewer System Capacity Analysis

The hydraulic capacity of sewers must be sufficient to convey peak wastewater flows. Peak flows are defined for a "design" condition based on criteria established by the agency, and the capacity of the system under those design conditions are evaluated against specified capacity criteria. The design flow and capacity criteria used for the NSMCSD capacity evaluation are discussed below. The calibrated hydraulic model was used to generate design flows and to identify capacity deficiencies. This section presents the results of the capacity analysis performed for both dry and wet weather flows under existing and future conditions.

5.1 Design Flow Criteria

The model calibration determined dry and wet weather flow parameters that represent existing flow conditions. These parameters were reviewed to determine their applicability for use in identifying future capacity deficiencies and for sizing future sewers. Based on this review, the following design criteria were adopted for use in the capacity analysis:

- The calibrated per-capita DWF rates for residential populations were used for both existing and future conditions. This assumes that there will be no significant reductions (e.g., from water conservation) or increases (e.g., from more intense water use) in the rates in the future.
- The same calibrated wet weather flow parameters were applied to generate flows throughout the system under both existing and future conditions. This assumes that there will be no significant reductions (e.g., from rehabilitation or replacement of sewers and private sewer laterals), or increases (e.g., from sewer deterioration) in I/I in the future. This basic assumption means that over the long-term, NSMCSD will perform sufficient sewer rehabilitation to maintain I/I flows at or below current levels. This assumption should be verified periodically at the time of future updates to the District's collection system capacity evaluation.
- A design rainfall event was applied in the model to the calibrated wet weather parameters to determine design peak wet weather flows. The 1993 Master Plan used a 4-hour duration design storm with uniform intensity of 0.45 inches per hour to evaluate hydraulic capacity. This event was considered to represent a 5-year frequency storm based on historical rainfall statistics. As discussed in previous TMs, use of a uniform intensity design storm is a non-conservative approach. Most agencies utilize a design rainfall event that includes a higher intensity peak hour rainfall (e.g., equivalent to a 1-hour, 5-year frequency rainfall intensity), resulting in a more conservative peak flow than a uniform intensity design storm. Therefore, for this capacity analysis, a more accepted varying-intensity storm with a peak intensity of 0.93 inches per hour was used to evaluate hydraulic capacity of the sewer system. The uniform and varying intensity design storms are shown in **Figure 16** and **Figure 17**, respectively.
- The timing of the design storm also affects the resulting peak wastewater flows. If the design storm is timed to cause peak RDI/I at the same time as peak base wastewater flow ("peak-on-peak"), the total peak wet weather flow will be higher than if the design storm occurs during the minimum base wastewater flow[1]. To be conservative, peak-on-peak timing using a weekend BWF diurnal curve (since higher morning peak flows typically occur on weekends) was assumed for this analysis for purposes of identifying potential capacity deficiencies and required improvement projects. However, model simulations for a less conservative peak-on-average

[1] Timing of the storm to produce peak-on-peak results is generally thought to create a return period in the peak wastewater flow that is greater than the return period of the design rainfall event itself (i.e., a 5-year storm event occurring at the same time as peak base wastewater flow would occur less often than a 5-year storm occurring at any other time during the day).

timing, as well as for a uniform intensity design storm, were also conducted to assess the relative risk of sewer system overflows to aid in prioritizing potential capacity improvement projects.

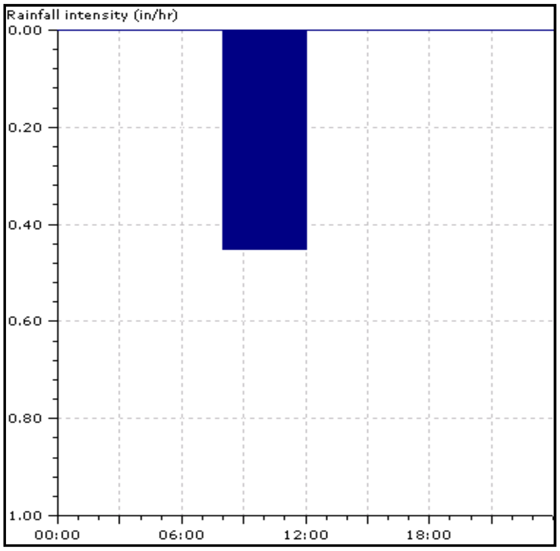

Figure 16: 5-Year Uniform Intensity Design Storm

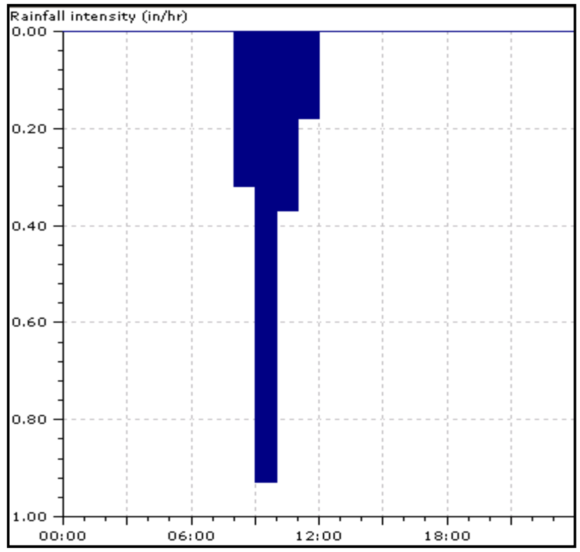

Figure 17: 5-Year Varying Intensity Design Storm

5.2 Capacity Analysis Criteria

Capacity deficiencies requiring improvement projects were identified based on surcharge conditions (water level above the crown of the pipe) under peak wet weather flows. Capacity restrictions causing surcharging within 5 feet of ground elevation were identified as improvement projects.

Note that surcharging does not necessarily indicate a capacity restriction at that particular location, as flows can back up into upstream pipes due to a "throttle" condition (i.e., capacity deficiency) in a downstream pipe section and cause extensive surcharging upstream due to backwater effects ("backup surcharge").

5.3 Dry Weather Flow Capacity Results

The calibrated model was run for dry weather flow under both existing and future scenarios. No pipes were found to be surcharged under peak dry weather flow conditions, under either existing or future scenarios.

5.4 Wet Weather Flow Capacity Results

The calibrated model was run with the design storm for both the existing and future scenarios. **Figure 18** presents the surcharge results for future peak wet weather flow conditions (results for existing conditions are virtually the same). As seen in the figure, surcharging is predicted to occur in several areas throughout the system. In some cases, the surcharge level may reach the ground surface, resulting in predicted overflows in the model. Profile plots of the most significant segments of surcharged pipes identified in the model are included in **Appendix E**.

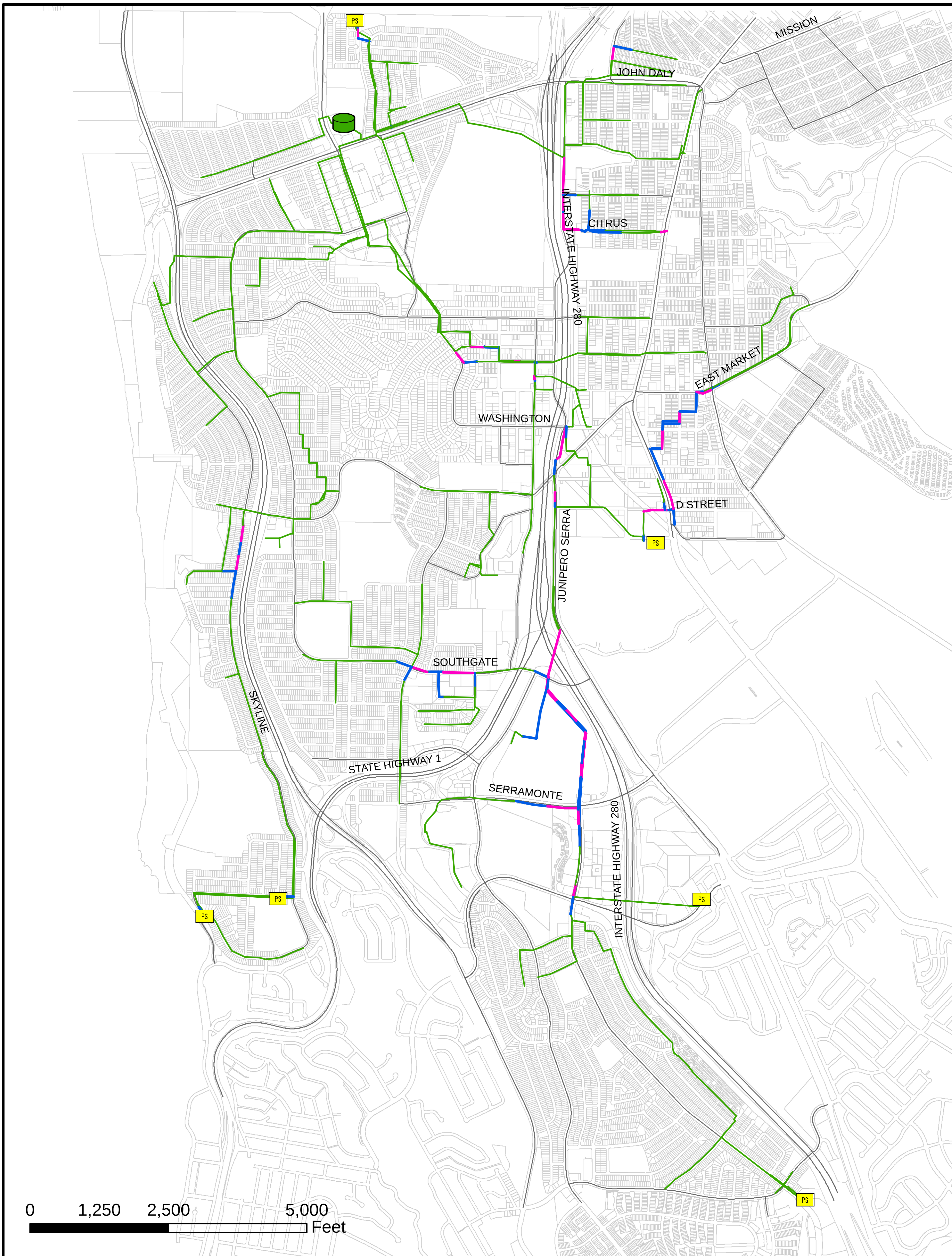

Figure 18: Model Predicted Capacity Deficiencies under Future Flows + Varying Intensity Design Storm

Legend

- No Surcharge
- Backup Surcharge
- Throttle Surcharge
- Lift Station
- WWTP

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As noted previously, surcharging does not necessarily indicate a capacity limitation at that particular location, as flows can back up due to a capacity deficient area and cause extensive surcharging due to backwater. Therefore, capacity improvement projects were not defined for pipes that were surcharged only due to backwater conditions. However, relieving upstream capacity deficiencies can often create additional capacity limitations downstream of the relieved pipe, and these effects were considered during development of improvement projects.

Table 8a shows the comparison of pump capacity to predicted existing peak wet weather design flows. Maximum wet weather inflows into these lift stations is virtually unchanged under future conditions; therefore, the magnitude of capacity deficiencies is not expected to increase with currently planned developments.

Comparing the maximum[1] and firm[2] lift station capacities to predicted peak wet weather design inflows shows that Belcrest, Skyline, and El Portal lift stations do not have sufficient capacity. Based on model results, Belcrest, Skyline, and El Portal lift stations are operating inefficiently; this is largely in part to undersized force mains, resulting in high head losses. To help remedy these deficiencies, three improvements projects have been identified to increase force mains capacity (theses projects are described in Section 6). Even with larger force mains, however, these lift stations will need all pumps in operation in order to prevent significant backup during the varying design storm event. **Table 8b** shows how firm and maximum capacities can increase by providing sufficient force main capacity. Under ideal conditions, each lift station should be able to pump maximum inflows using only firm capacity. In order for this to hold true for these three pump stations, more pump capacity (i.e., additional pumps) would be required in addition to the identified improvement projects.

Table 8a: Existing Lift station Capacities

Lift station	No. of Pumps	Motor Speed	Max Design Inflows: Varying Storm (mgd)	Max Design Inflows: Uniform Storm (mgd)	Modeled Firm[1] Capacity (mgd)	Modeled Max[2] Capacity (mgd)
Belcrest	2[3]	constant	0.76[4]	0.62[4]	0.36	0.45
Colma	3	constant	2.38	2.05	2.45	2.75
El Portal	2	variable	1.31	0.71	0.56	0.63
Hickey	2	constant	0.65	0.48	1.10	--[5]
Rowntree	3	variable	1.17	1.07	--[6]	--[6]
Skyline	2[3]	constant	0.68[4]	0.53[4]	0.38	0.45

1. Assumes one pump is offline
2. Assumes all pumps in operation
3. Belcrest and Skyline consist of two sets of two pumps in series, for a total of 4 pump at each lift station.
4. Note that during peak storm events, flow backs up in the Belcrest wet well and flows back to Skyline lift station. This situation was accounted for in the model, however, maximum inflows shown in this table ignore flow circulation. The firm capacity of Rowntree was assumed equal to the calculated firm capacity.
5. Hickey lift station requires use of only one pump under modeled peak flows.
6. To mimic variable frequency controls, this pump was modeled to gradually pump out more flow as wet well levels increase, up to the estimated capacity of the lift station. Modeling the pump this way will limit lift station throughput to only what is needed (thus the lift station has more actual capacity than the modeled maximum flow shows).

[1] Maximum capacity is the capacity of the lift station will all pumps in operation
[2] Firm capacity is the capacity of the lift station will one pump out

Table 8b: Lift Station Capacities with Force Main Improvements

Lift station	No. of Pumps	Motor Speed	Max Design Inflows		Modeled Firm[1] Capacity (mgd)	Modeled Max[2] Capacity (mgd)
			Varying Storm (mgd)	Uniform Storm (mgd)		
Belcrest	2	constant	0.76	0.62	0.61	0.75
El Portal	2	variable	1.31	0.71	0.72	1.35
Skyline	2	constant	0.68	0.53	0.53	0.66

1. Assumes one pump is offline
2. Assumes all pumps are operating

6 Recommended Capacity Improvement Program

This section presents the specific sewer projects that are recommended for inclusion in the District's capital improvement program (CIP), based on the findings of the capacity analysis. Each project is documented with a general description, project details, planning-level capital cost estimates, and relative priority rating. Recommendations for project implementation are provided.

6.1 Project Identification

Capital improvement projects were identified to proactively address potential problems identified in the capacity analysis. For each capacity limitation identified, a project was developed to replace the existing pipe with a larger pipe or parallel the existing pipe with another pipe. **Figure 19** shows an overview of the project locations, and **Table 9** summarizes all of the identified projects, including location, a brief description (length and diameter of pipe), relative priority, and estimated planning-level cost. Explanations of project priorities and cost estimates follow in subsequent sections. A detailed description of each project is included in **Appendix F**.

As noted in Section 5, the need for a project was identified based on the extent of surcharge and depth of water level below ground surface. Specifically, a project was developed for any reach of pipe which the model determined both to be under capacity and to cause surcharging within 5 feet of ground level under a 5-year design storm with varying intensity. These surcharge criteria allow the District to focus capital spending on areas with the highest risk of sanitary sewer overflows (SSOs).

Projects were sized to not surcharge under future peak wet weather conditions, although a minimal amount of remaining surcharge was considered acceptable if installation of a parallel or replacement pipe would be very difficult to construct. Projects were sized assuming that the existing pipe was replaced with a larger pipe at the same slope as the existing pipe, or the pipe was paralleled with another pipe such that the combined capacity of both pipes would provide sufficient capacity to convey the peak design flows.

Some potential alternatives to pipe replacement, such as diverting or re-routing flows, or using alternative alignments, were identified and considered as part of this evaluation. However, these types of alternatives were not analyzed extensively. The District should evaluate such possibilities during the project pre-design phase, or as part of further planning prior to constructing these projects. Potential for such alternatives on a project-by-project basis are included on the project detail tables at the end of this section.

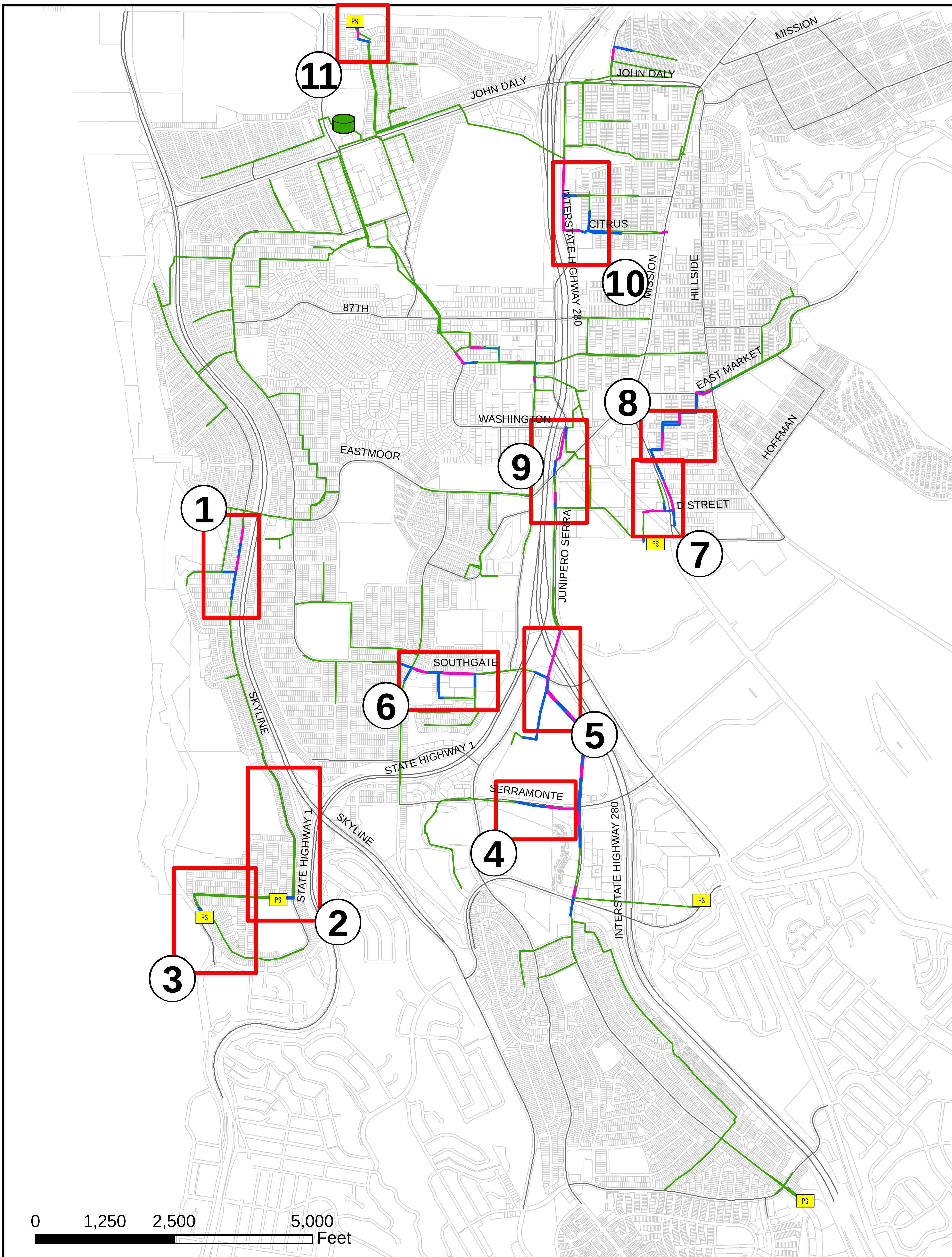

Figure 19: Capacity Improvement Project Locations

Legend

- No Surcharge
- Backup Surcharge
- Throttle Surcharge
- PS Lift Station
- WWTP

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Table 9: Recommended Capacity Improvement Projects, Ordered by Project ID

Project ID	Project Name	Description & Location	Priority	Estimated Capital Cost (2008 Dollars)
C-1	Skyline Blvd.	Upsize 1,080 ft of 8" pipe to 10" along Skyline Dr. from Carmel Ave. to south of Westmoor Ave	4	$377,000
C-2	Belcrest LS/force main	New parallel 6" FM (3,082 ft) from Belcrest Lift Station to Oceanside Dr.	1	$851,000
C-3	Skyline LS/force main	New parallel 6" FM (1,925 ft) from Skyline Lift Station to Belcrest Lift Station	1	$532,000
C-4	Serramonte Blvd.	Upsize 575 ft of 8" pipe to 10" along Serramonte Blvd. from Gellert Blvd. to Serramonte Center Driveway	3	$201,000
C-5	I-280 Crossing	New 30" I-280 crossing, from Southgate Ave to Junipero Serra (750 ft of 48" casing and 275 feet open cut pipe)	3	$2,927,000
C-6	Southgate Ave.	Upsize 578 ft of 10" pipe to 15" along Southgate Ave from Escuela Dr. to Cerro Dr	5	$225,000
C-7	D Street/Mission St.	Upsize 1,083 ft of 10" and 12" pipe to 15" and 285 ft of 10" pipe to 12" along Mission Street and D St	2	$524,000
C-8	A Street to E. Market	Upsize 339 ft of 10" pipe to 12" and 178 ft of 8" pipe to 10" in 1st Ave. and 2nd Ave. between A Street and East Market	5	$183,000
C-9	Junipero Serra/San Pedro	New parallel 24" pipe (516 ft) parallel to Junipero Serra Blvd. between San Pedro and Washington	4	$323,000
C-10	Junipero Serra/Citrus	Upsize 116 ft of 8" pipe to 10" and 593 ft of 8" and 10" pipe to 12" and 977 ft of 10" and 12" pipe to 15" along Junipero Serra Blvd and Citrus Avenue	2	$634,000
C-11	El Portal LS/force main	New 8" parallel force main (1,219 ft) at El Portal along Cliffside St	2	$357,000
		Total Estimated Capital Cost		**$7,134,000**

7 Project Cost Estimates

Capital costs were estimated based on cost data and prior experience from similar projects in other Bay Area communities. These cost estimates are planning or conceptual level estimates, and are considered to have an estimated accuracy range of -30 to +50 percent. This level of accuracy corresponds to an "order of magnitude" or "Class 5" cost estimate as defined by the American Association of Cost Estimators. These estimates are suitable for use for budget forecasting, CIP development, and project evaluations, with the understanding that refinements to the project details and costs would be necessary as projects proceed into the design and construction phases. Estimates were prepared using late-2008 U.S. dollars. The cost index used for escalation of past bid prices is the Engineering News Record Construction Cost Index (ENR-CCI) for San Francisco Bay Area. The December 2008 index value is 9781.

Cost criteria include baseline construction costs for gravity trunk sewers using open-cut and trenchless (e.g., pipe bursting, microtunneling) methods. Unit costs for gravity trunk sewers vary with pipe diameter and depth, and are assumed to include manholes, lateral connections, mobilization/demobilization, bypass pumping, pavement restoration, and traffic control. A 30 percent allowance for contingencies for unknown conditions was also included for all projects, as well as an allowance of 25 percent of construction cost for engineering, administration, and legal costs. **Table 10** summarizes the general cost criteria. The itemized cost estimate for each project is detailed on the individual project information sheets included at the end of this section.

Table 10: Cost Criteria

Item	Description/Assumptions	Unit Cost[1]
Baseline Pipeline Construction Costs		
Pipe Burst gravity sewers	10-inch to 15-inch diameter	215 – 240 $/lf
Open-cut gravity sewers	10-inch to 24-inch diameter	290 – 385 $/lf
Open-cut force main	6-inch to 8-inch diameter	170 - 180 $/lf
Trenchless construction	48" casing installed with tunnel boring machine	2,200 $/lf
Cost Allowances		
Construction Contingency	% of Baseline Construction Subtotal	30%
Engineering, Administration, and Legal Costs	% of Construction Subtotal	25%

1. Engineering New Record Construction Cost Index (ENR-CCI) 9781. San Francisco Bay Area, December 2008.

7.1 Project Priorities

Each project was given one of five overall priority ratings, based on the severity of capacity limitations, conservativeness of model results, and other considerations. The process used to develop these priorities is described in the paragraphs below.

Capacity limitations were defined in terms of level of surcharge, expressed as the distance from the ground surface to the maximum water level under peak flow conditions. A higher level of surcharge (smaller distance between the ground and the water level) is indicative of a more critical capacity limitation.

Level of surcharge was also calculated for several different modeling scenarios consisting of both existing and future conditions, type of design storm, and timing of the storm versus the diurnal BWF. As described in Section 5.1, the two rainfall events considered are the uniform intensity storm (less conservative) and the varying intensity storm (more conservative). The timing of the design storm also affects the resulting peak wastewater flows. If the design storm is timed to cause peak RDI/I at the same time as peak BWF, the total peak wet weather flow will be higher than if the design storm occurs during the average BWF. For the purposes of this analysis, scenarios were defined for the varying design rainfall event applied to both average and peak BWF. **Table 11** shows how the deficient pipe segments performed under these various scenarios.

Table 11: Surcharge Depth under Various Scenarios

Project ID	Project Name	Pipe Surcharge (depth of water below ground, feet)			
	Model Scenario>>	*A*	*B*	*C*	*D*
	Planning Period>>	*Future*	*Existing*	*Future*	*Future*
	Design Storm>>	*Varying*	*Varying*	*Varying*	*Uniform*
	Timing of Storm>>	*Peak BWF*	*Peak BWF*	*Average BWF*	*Peak BWF*
C-1	Skyline Blvd.	2.7	2.7	2.8	2.8
C-2	Belcrest LS/force main	0	0	0	0
C-3	Skyline LS/force main	0	0	0	0
C-4	Serramonte Blvd.	0	2.1	2.5	4.9
C-5	I-280 Crossing	1.1	1.4	5.2	NS
C-6	Southgate Ave.	1.8	2.5	NS	NS
C-7	D Street/Mission St.	0	0	0	2.6
C-8	A Street to E. Market	3.3	3.3	5.9	NS
C-9	Junipero Serra/San Pedro	3.8	4	6.1	NS
C-10	Junipero Serra/Citrus	0	0	0	0.3
C-11	El Portal LS/force main	0	0	0	0

Note: NS = No surcharge. “0” depth of water means potential overflow.

Each project was given a "surcharge score" for each scenario based on the levels of surcharge shown in Table 11 and illustrated in **Figure 20**. The highest score (5) was given for potential overflow condition, and the lowest score (1) for a no surcharge condition.

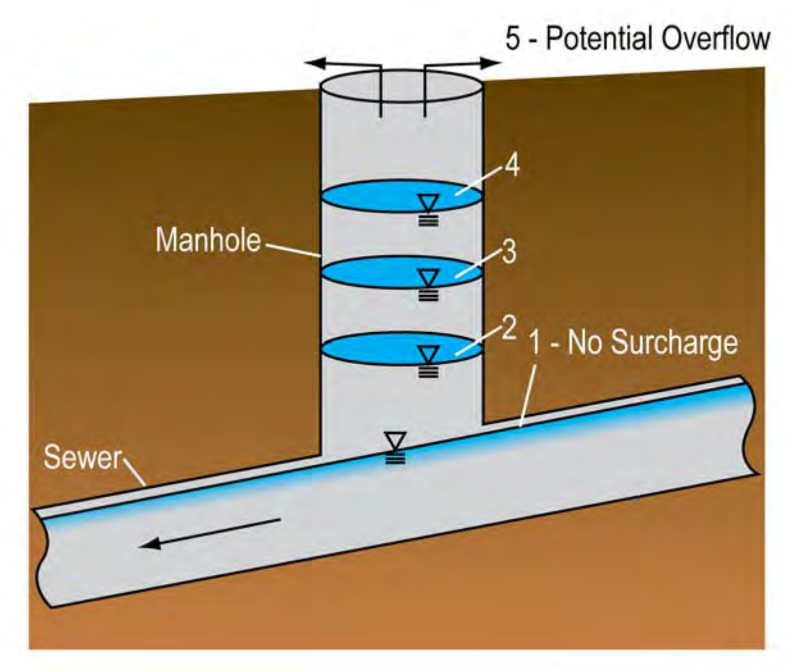


Figure 20: Surcharge Scores

The resulting surcharge that occurs under the various scenarios was weighted based on the criticality of the scenario or, in other words, the potential for a sanitary sewer overflow. For example, a pipe that surcharges under existing conditions is a more critical situation than one that surcharges only under future conditions. Similarly, a surcharge condition which occurs under both a uniform and varying design storm event is more critical than one that occurs only during the larger, more conservative varying rainfall event. The weights assigned for each modeling scenario are shown in **Table 12**.

Table 12: Relative Criticality of Modeling Scenarios

Modeling Scenario	Description	Weight* (Relative Criticality)
A	Future conditions, varying intensity design storm at peak BWF	1
B	Existing conditions, varying intensity design storm at peak BWF	2
C	Future conditions, varying intensity design storm at average BWF	3
D	Future conditions, uniform intensity design storm at peak BWF	4

* Higher weight = greater criticality (surcharge under that scenario means higher potential for overflows)

Based on the surcharge scores and weights, a total surcharge score was calculated for each project, as follows:

$$\text{Total Surcharge Score} = 1 \times SS_A + 2 \times SS_B + 3 \times SS_C + 4 \times SS_D \text{ (highest possible score = 50)}$$

Where SS_A is the surcharge score for model scenario A, etc.

Lastly, other factors were taken into account when assigning project priority. For example, Project C-5, the I-280 Crossing, was given a slightly higher priority rating than other projects with similar weighted surcharge scores because it is a single pipe in a very critical location. Similarly, Belcrest and Skyline lift stations (Projects C-2 and C-3) were assigned the highest priority not only because the model predicted sewer overflows for this area, but also because actual sewer overflows were documented in 2008.

Table 13 shows the surcharge scores and resulting priority ratings.

Table 13: Capacity Improvement Project Prioritization

Project ID	Project Name	Surcharge Score				Total Surcharge Score	Other Considerations	Priority
	Model Scenario>>	*A*	*B*	*C*	*D*			
	Weight>>	*1*	*2*	*3*	*4*			
C-1	Skyline Blvd.	3	3	3	3	30		**4**
C-2	Belcrest LS/force main	5	5	5	5	50	Prev. SSOs	**1**
C-3	Skyline LS/force main	5	5	5	5	50	Prev. SSOs, location next to failing bluff	**1**
C-4	Serramonte Blvd.	5	3	3	2	28		**4**
C-5	I-280 Crossing	4	4	2	1	22	Critical crossing	**3**
C-6	Southgate Ave.	4	3	1	1	17		**5**
C-7	D Street/Mission St.	5	5	5	3	42		**2**
C-8	A Street to E. Market	3	3	2	1	19		**5**
C-9	Junipero Serra/San Pedro	3	3	2	1	19	Maintenance problems	**4**
C-10	Junipero Serra/Citrus	5	5	5	5	50		**2**
C-11	El Portal LS/force main	5	5	5	5	50		**2**

Note: See Table 12 for definition of model scenarios and weights.

7.2 Project Implementation

The District should begin implementation of the Capital Improvement Program recommended in this TM, starting with the highest priority projects. This TM does not specify an implementation schedule, as the District will need to balance sewer capacity improvements with the need for other capital projects. The following items should be considered in project scheduling and design, and in future updates of the Collection System Capacity Evaluation.

- Move forward with further planning and design of the Priority 1 projects. Flow monitoring should be considered to verify the modeled flows and capacity deficiencies unless monitoring has already been performed in the vicinity or capacity problems (surcharge or overflows) have been documented in the field. Further planning or pre-design work should include consideration of alternative routes for new sewers, pump station upgrades versus parallel force mains, and other potential options. The potential for combining projects (e.g., capacity and rehabilitation projects) should be reviewed to avoid building multiple projects in the same area and to identify more cost-effective solutions.
- The alignments and sizes of all recommended projects should be verified with detailed predesign analyses, including topographic surveys, geotechnical investigations, utility research, and constructability reviews. This is particularly important because the accuracy of pipe slopes in the model was limited by the accuracy of the GIS data provided (elevations only to nearest whole foot).
- Most projects detailed in this TM are based on pipe replacement. The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.
- The District should assess its sewer rates and fees and evaluate financial alternatives to fund the recommended capacity improvement CIP. The capacity evaluation should be updated whenever there are major changes in planning assumptions or, at a minimum, every five years.

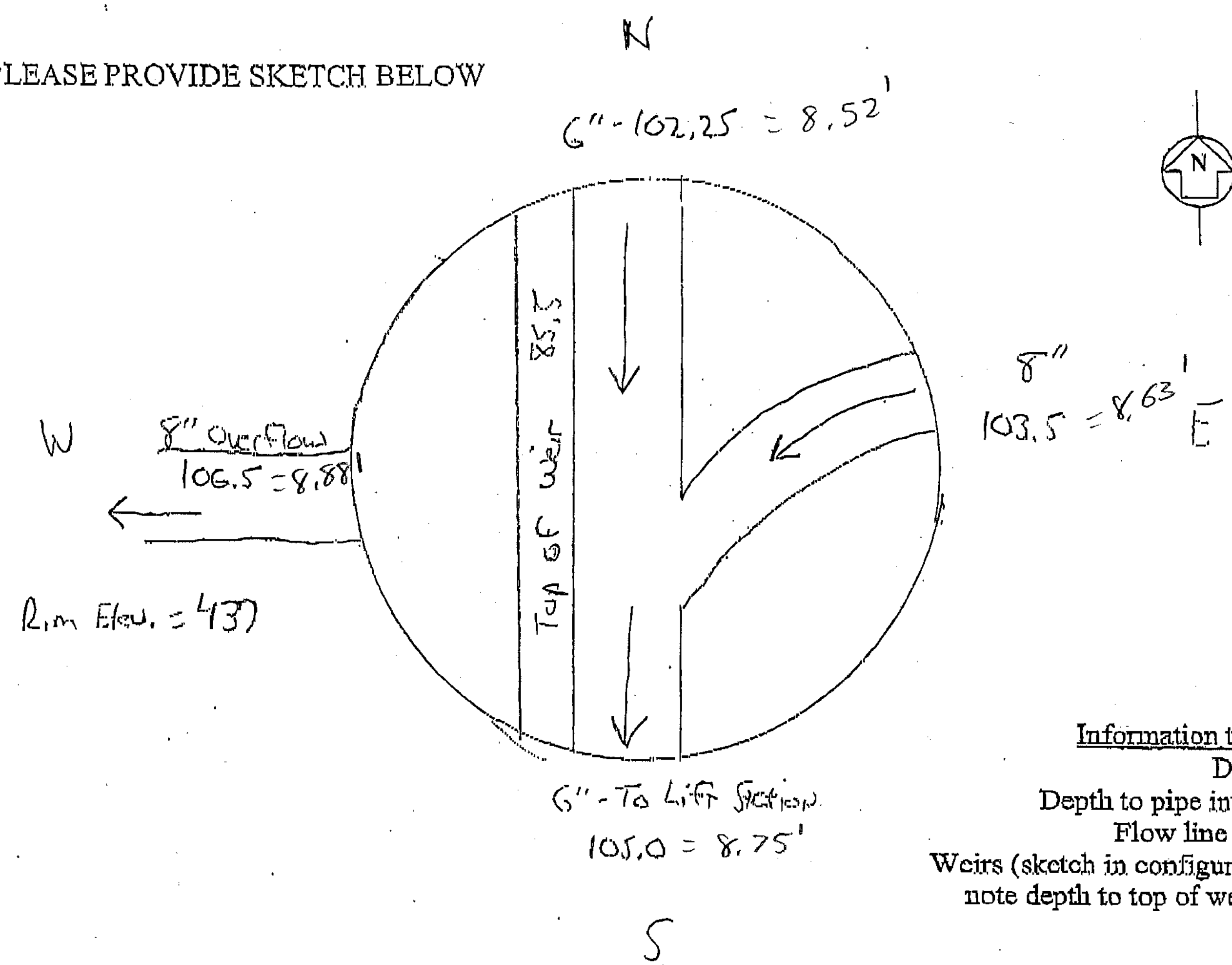
7.3 Detailed Project Descriptions

Detailed descriptions of the recommended projects are included in Appendix F. The descriptions are each contained on a single page and follow a standard format that consists of a summary project description, including project name, priority, brief description (pipe length, diameter, location), and other pertinent information, including key assumptions and potential alternatives. Each description also includes an itemized cost estimate. The projects are presented in numerical order by their project identifier.

Appendix A
Field Notes

Manhole (Node) ID:	MH - #6 - B-10	Survey Date:	3-11-09
Atlas Map:		Survey Party:	Jeff B.
Street:	Belcrest		Kevin M.
Nearest Cross Street:	Clearview		
Closest Street Address:	Intersection		

PLEASE PROVIDE SKETCH BELOW



Information to Note:
Diameter
Depth to pipe invert (ft)
Flow line (arrow)
Weirs (sketch in configuration & note depth to top of weir in ft)

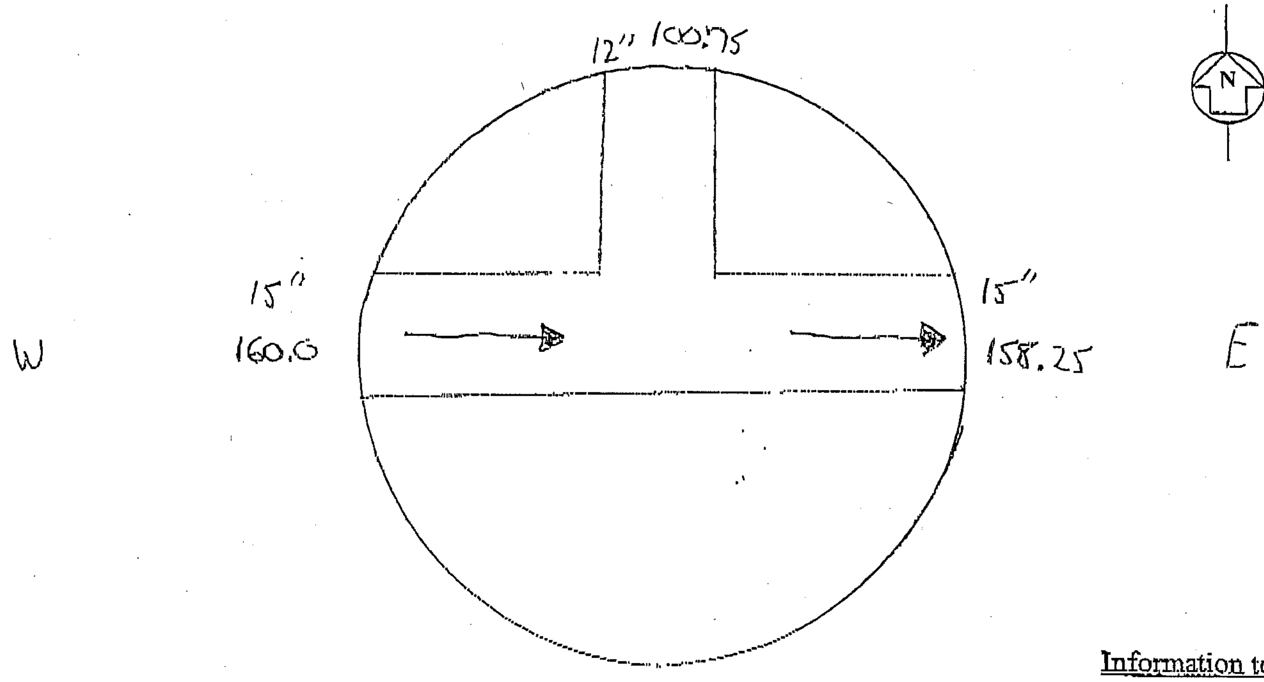
Comments:

RMC Water and Environment		CITY OF Daly City MANHOLE SURVEY

Manhole (Node) ID:	MH - #78 - C-1 (West)	Survey Date:	3-11-09
Atlas Map:		Survey Party:	Jeff B
Street:	Callan		Kevin M
Nearest Cross Street:	Meath		
Closest Street Address:	Intersection		

N

PLEASE PROVIDE SKETCH BELOW



S

Information to Note:
Diameter
Depth to pipe invert (ft)
Flow line (arrow)
Weirs (sketch in configuration & note depth to top of weir in ft)

Comments:

RMC Water and Environment

CITY OF Daly City
MANHOLE SURVEY

Manhole (Node) ID: ______ Survey Date: ______

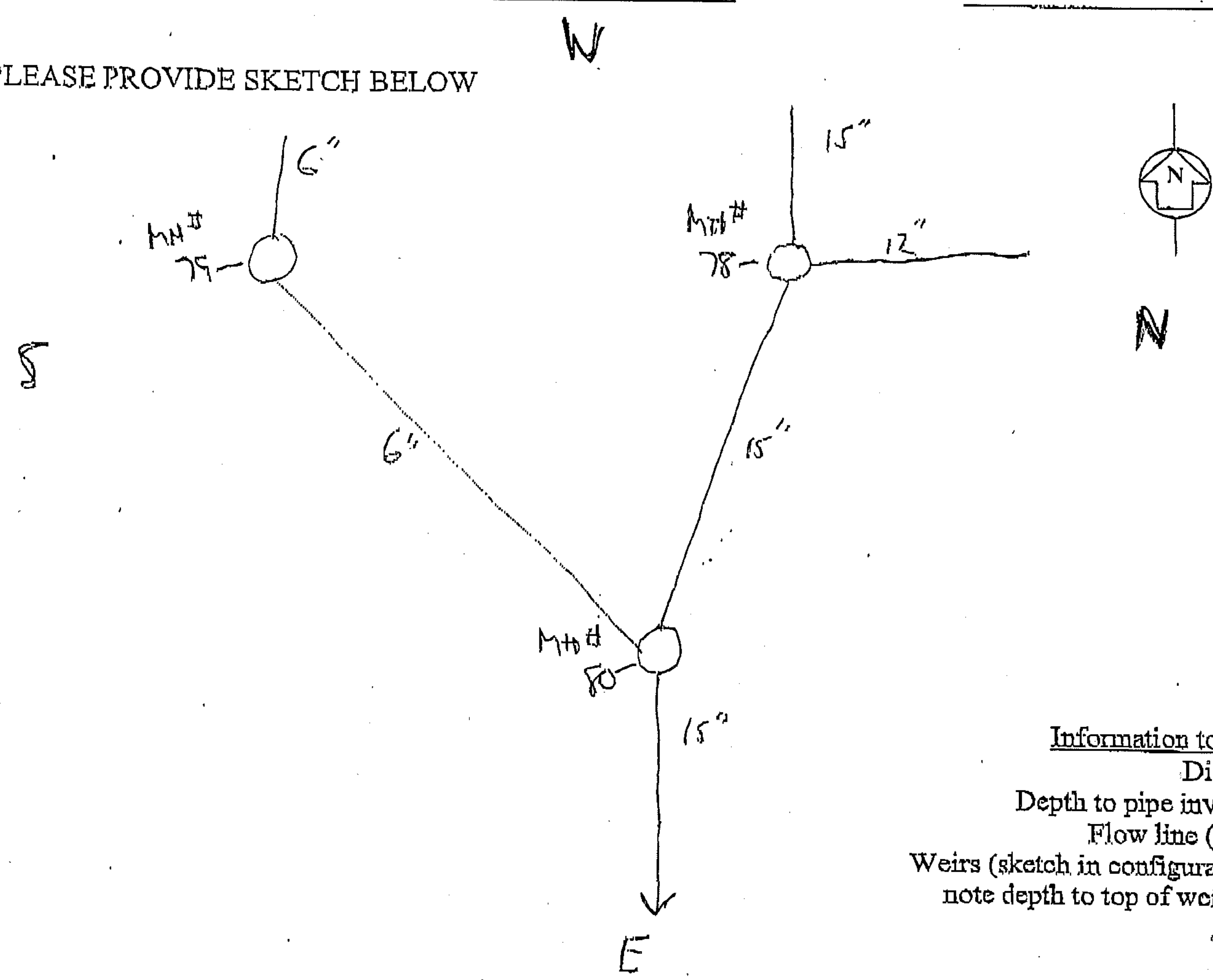
Atlas Map: ______ Survey Party: ______

Street: ______

Nearest Cross Street: ______

Closest Street Address: ______

PLEASE PROVIDE SKETCH BELOW



Information to Note:
Diameter
Depth to pipe invert (ft)
Flow line (arrow)
Weirs (sketch in configuration & note depth to top of weir in ft)

Comments: Actual in Field for Callan/Keith Intersection Different From what is shown on map.

RMC Water and Environment

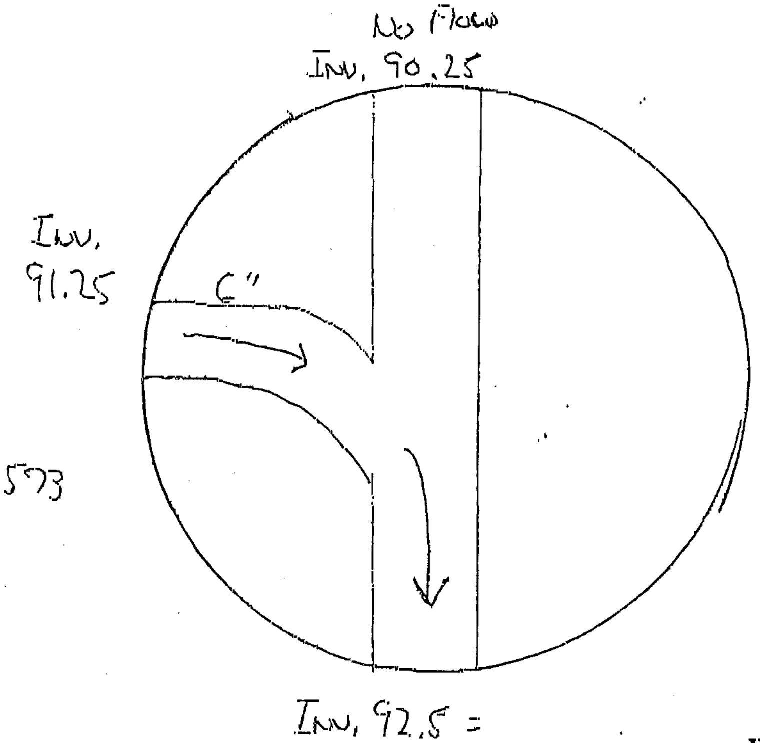
CITY OF Daly City
MANHOLE SURVEY

MH-E13-33

Manhole (Node) ID:	MN - #33 - E-13	Survey Date:	3-11-09
Atlas Map:		Survey Party:	Jeff B/ Kevin M
Street:	Gellert		
Nearest Cross Street:	Warwick		
Closest Street Address:	Intersection		

PLEASE PROVIDE SKETCH BELOW N

W



E

Rim Elev = 573

Information to Note:
Diameter
Depth to pipe invert (ft)
Flow line (arrow)
Weirs (sketch in configuration & note depth to top of weir in ft)

S

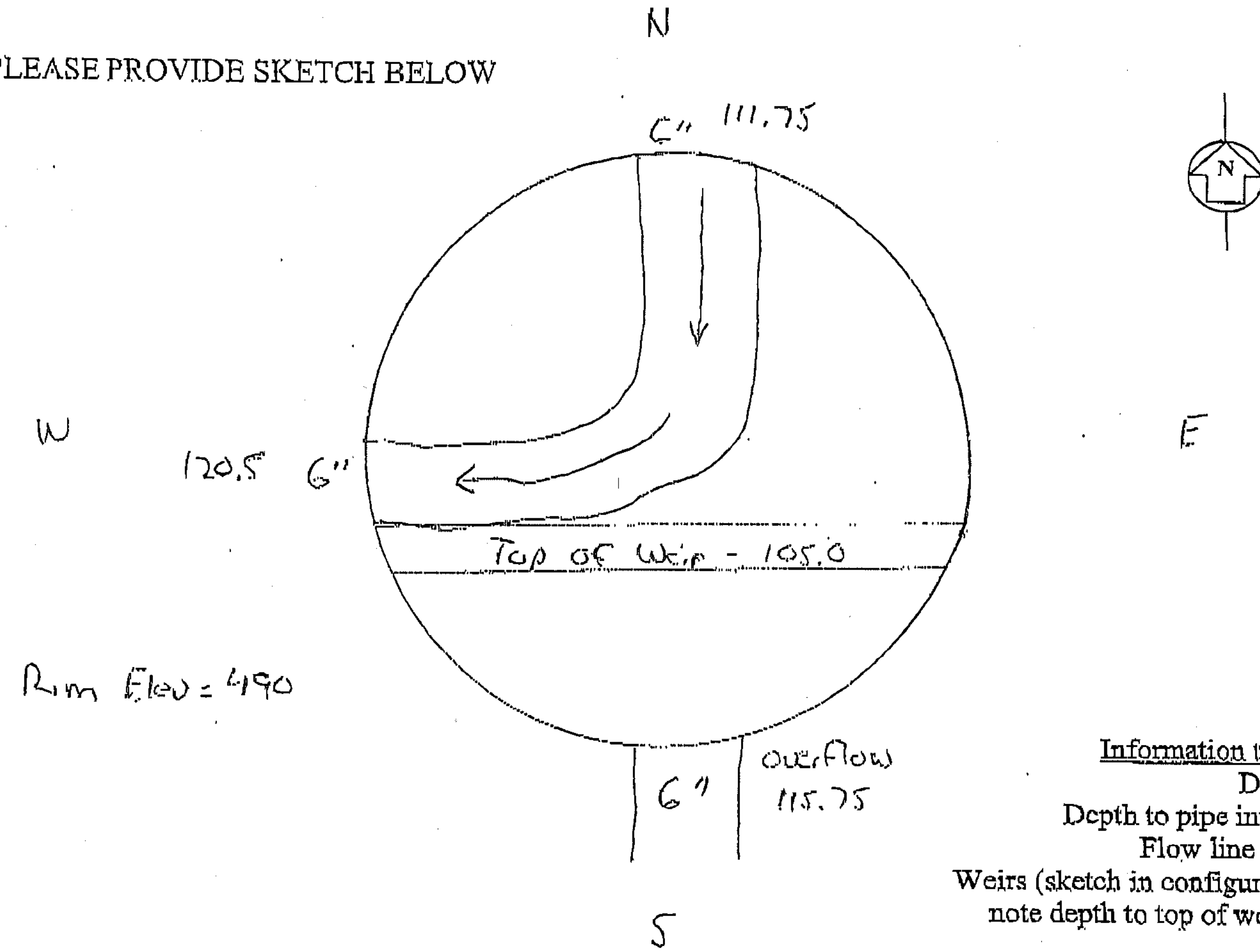
Comments:

RMC Water and Environment

CITY OF Daly City
MANHOLE SURVEY

Manhole (Node) ID:	MH. #135- E-4	Survey Date:	3-11-09
Atlas Map:		Survey Party:	Jeff B
Street:	Florence		Kevin M
Nearest Cross Street:	Theirs		
Closest Street Address:	73 Florence		

PLEASE PROVIDE SKETCH BELOW



Information to Note:
Diameter
Depth to pipe invert (ft)
Flow line (arrow)
Weirs (sketch in configuration & note depth to top of weir in ft)

Comments:

RMC Water and Environment

CITY OF Daly City
MANHOLE SURVEY

DEPARTMENT OF WATER AND WASTEWATER RESOURCES

(650) 515-0263

Phone: ~~(650) 746-8314~~

CITY OF DALY CITY
153 Lake Merced Boulevard
Daly City, CA 94015

(650) 746-8323

Fax: ~~(650) 746-8346~~

FAX TRANSMISSION COVER LETTER

DATE: 1-26-09 **TIME:** 11:30

TO: Aaron Hope

FAX NO.: 415-324-3401

 # OF PAGES: 2 *(including this page)*

Originals to be mailed? Yes ___ No ✓

FROM: Kevin McCarthy

Central Warehouse
FAX No. (650) ~~746-8346~~ 746-8323

MESSAGE: (if any)

Hello Aaron,

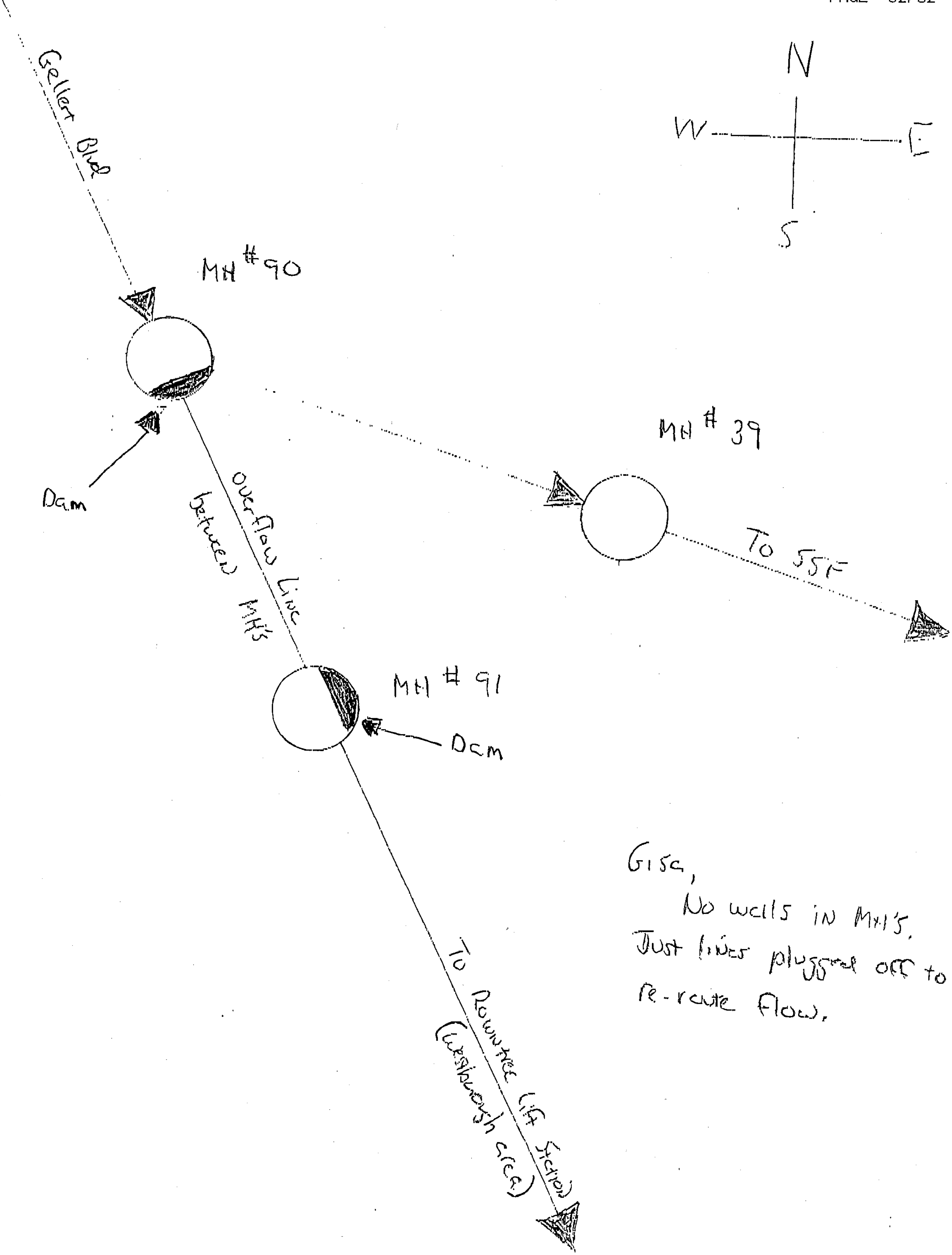
Spoke w/ Gisa & she said to fax you this drawing. Hope it helps. More info. to follow on your other inquiries. Questions 2 Cell # 650. 515-0263

Thanks

Kevin

Warehouse fax.doc

12/4/03



Gisa,

No walls in MH's. Just lines plugged off to re-route flow.

Appendix B
Pump Curves

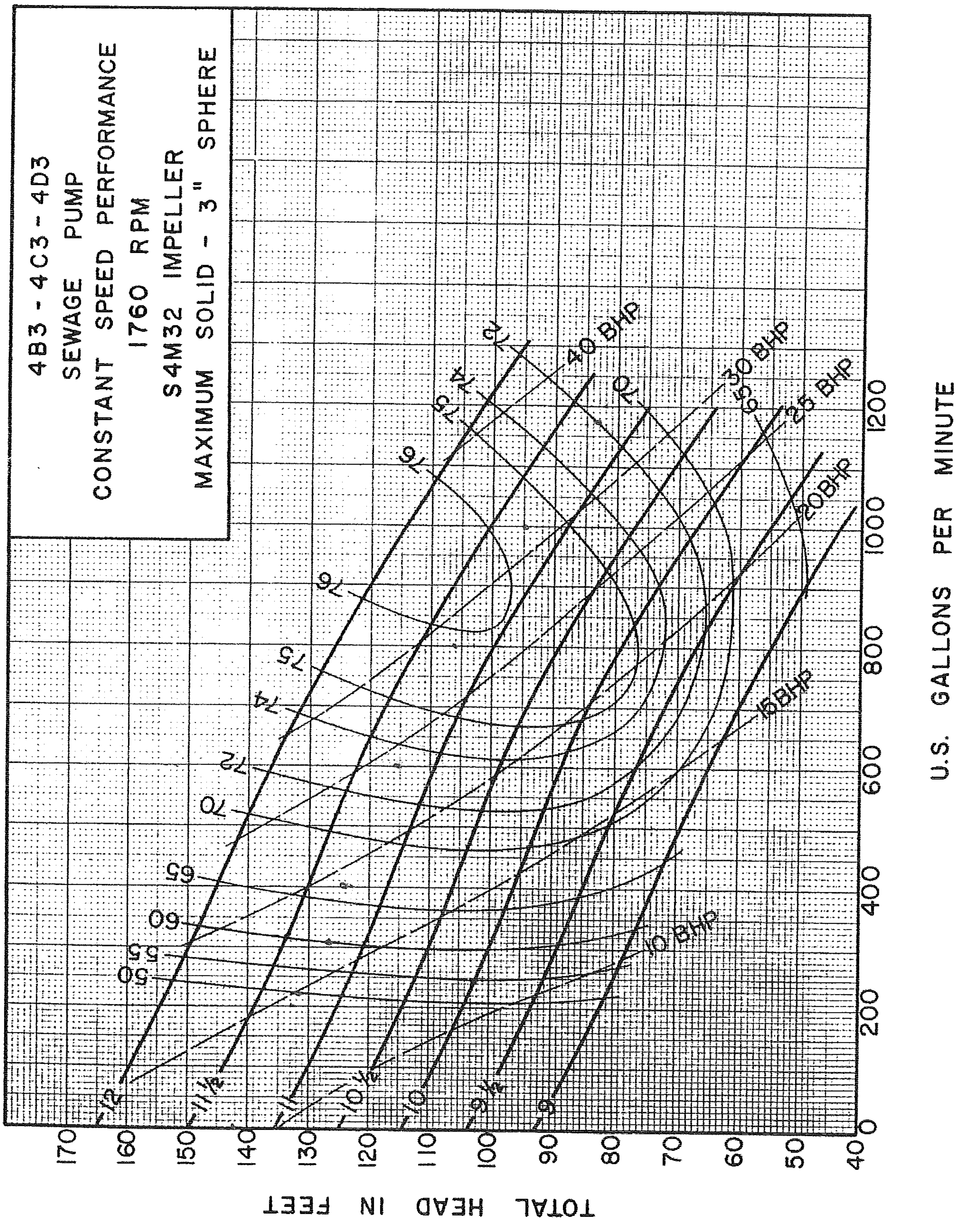
Belcrest

Lift Station

Smith & Loveless Engineering Data

Main Plant: Lenexa, Kansas 66215

Pump Performance Curves
Constant Speed
4B3/4C3/4D3
1760 RPM



Colma

Lift Station

Fairbanks Morse Pump
A Member of Pentair Pump Group

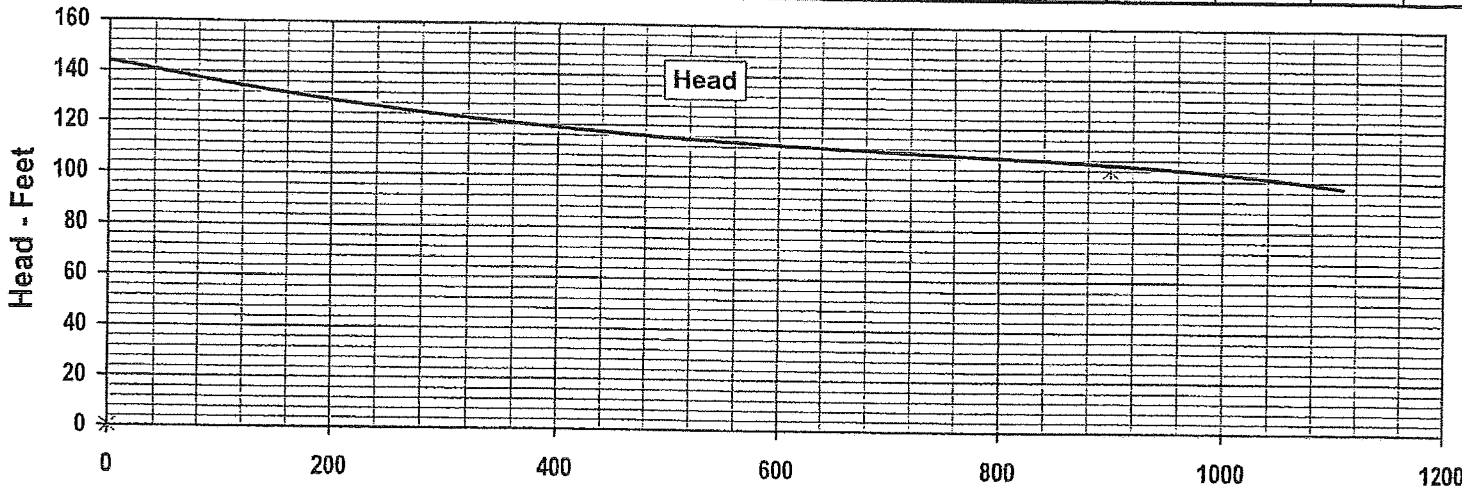
CURVE NO.:	C-085337
REV.	
REFERENCE:	080895

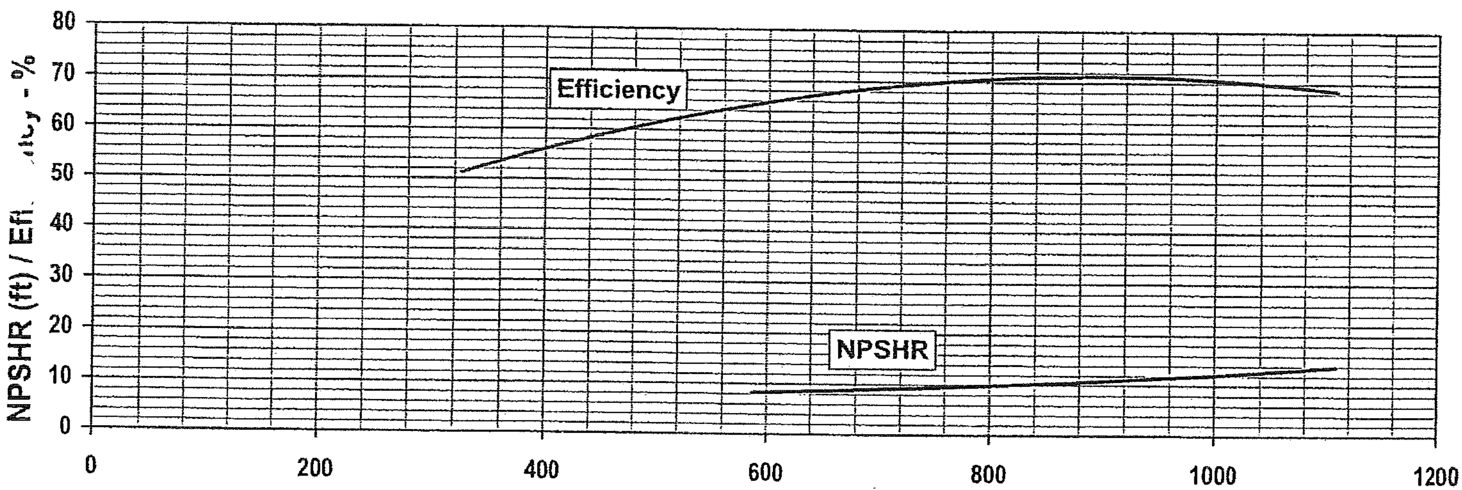
4-B5444L SUBMITTAL CURVE

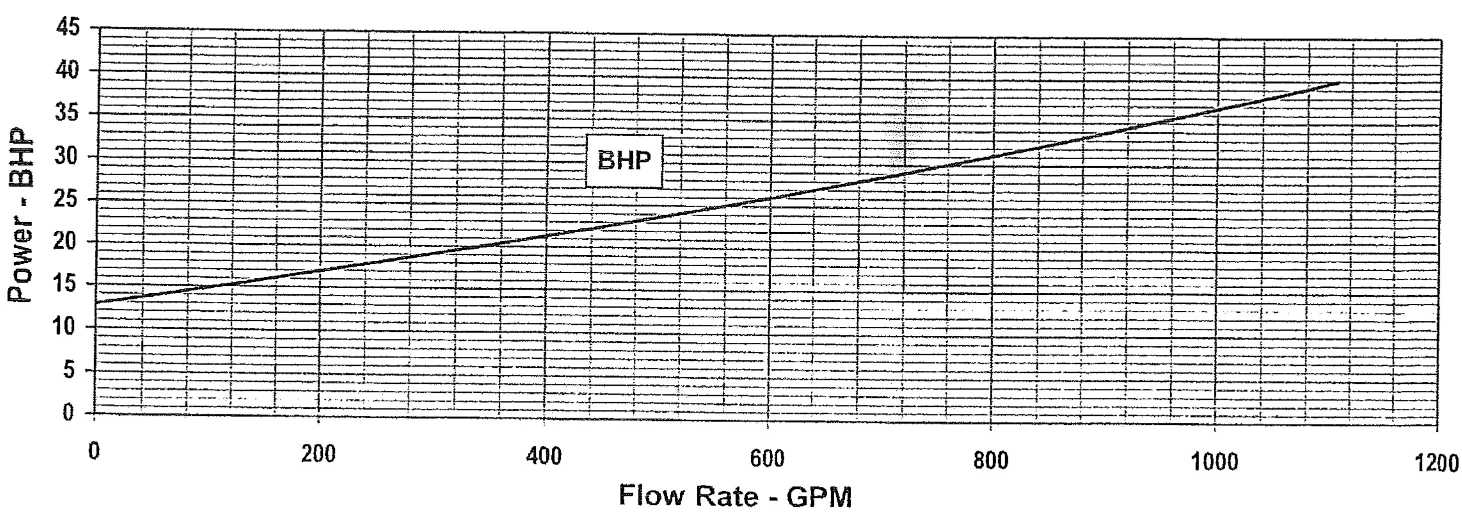
SPEED	IMPELLER	DIAMETER	VANES
1175	T4D1B	17.00	2
SPHERE	**DRIVER**	**DATE**	**BY**
3"	40 HP	7/22/02	BW

THIS CURVE IS BASED ON THE ACTUAL TEST PERFORMANCE OF A SIMILAR PUMP. ONLY THE INDICATED POINT(S) IS GUARANTEED.

GUARANTEED VALUES			
FLOW	HEAD	EFF.	BHP
900	104		---







El Portal

Lift Station

Fairbanks Morse Pump
A Member of Pentair Pump Group

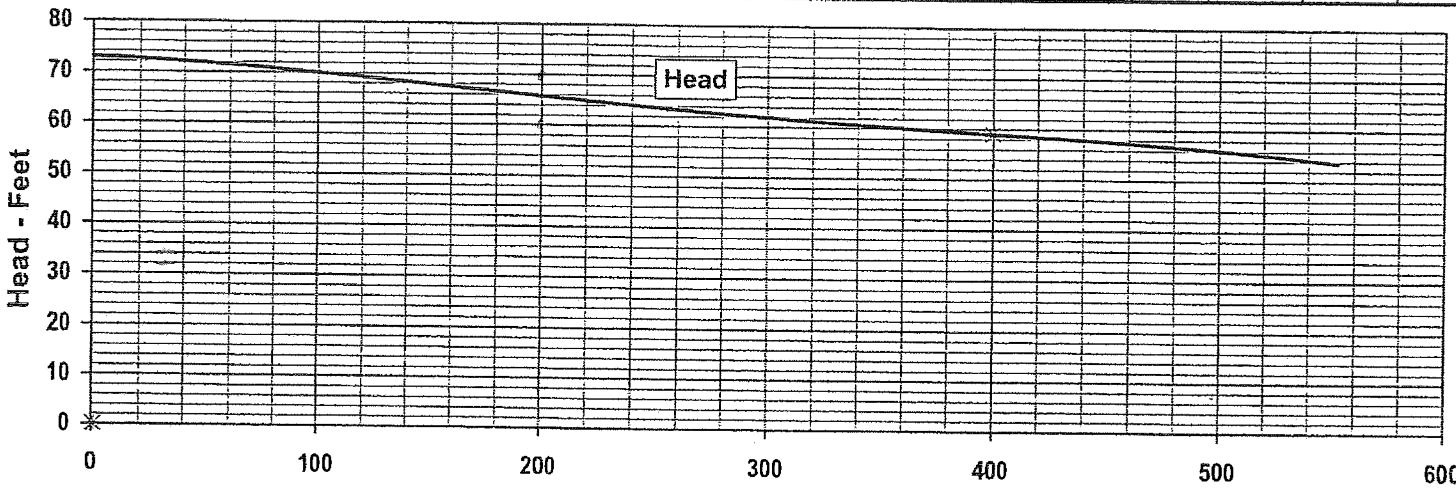
4-B5433 SUBMITTAL CURVE

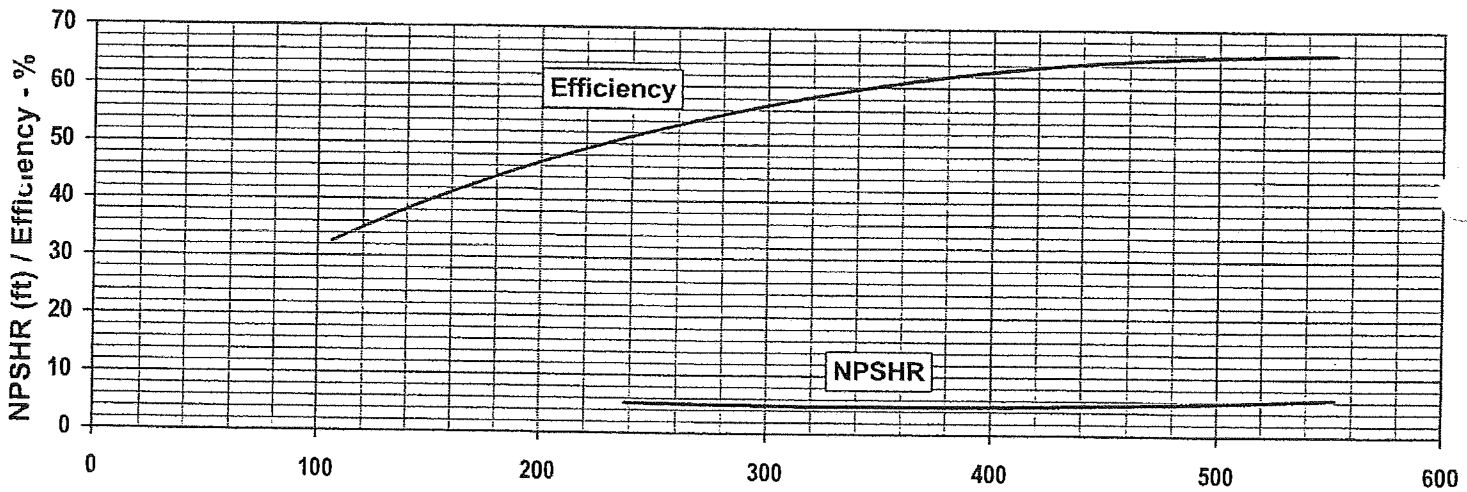
CURVE NO.:	C-085338
REV.	
REFERENCE:	084622

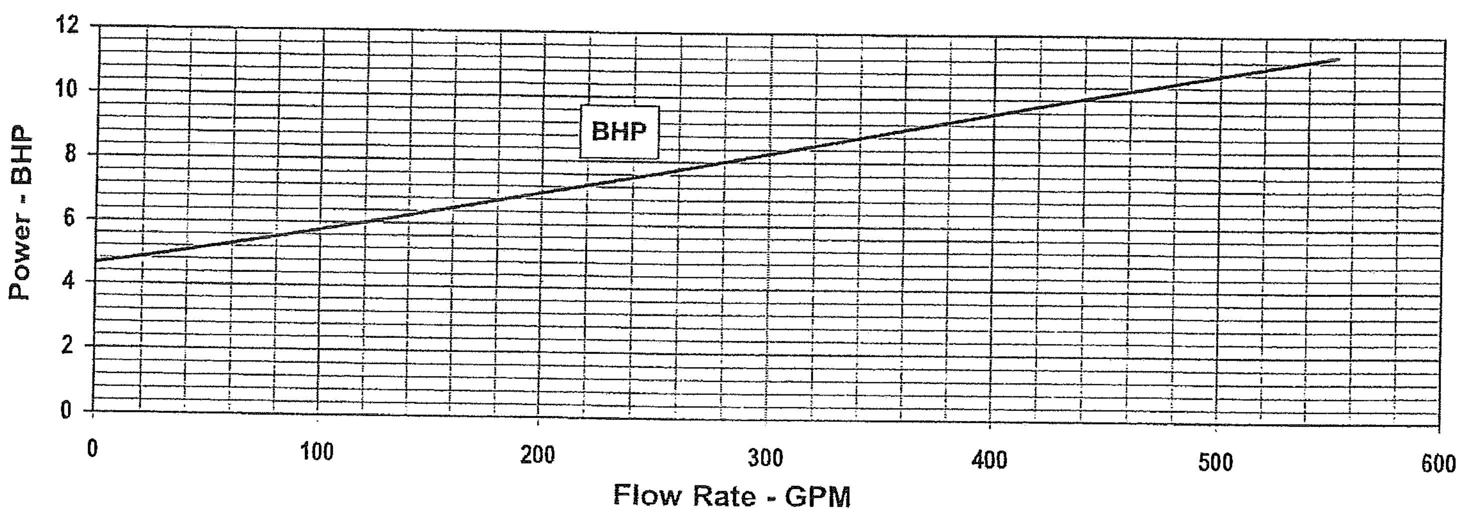
SPEED	IMPELLER	DIAMETER	VANES
1160	T4C1A	17.00	2
SPHERE	**DRIVER**	**DATE**	**BY**
3"	10 HP	7/22/02	BW

THIS CURVE IS BASED ON THE ACTUAL TEST PERFORMANCE OF A SIMILAR PUMP. ONLY THE INDICATED POINT(S) IS GUARANTEED.

GUARANTEED VALUES			
FLOW	HEAD	EFF.	BHP
400	59		-







Hickey

Lift Station

TEST REPORT

PRODUCT

Serial No.		Performance curve No.	Motor module/type	Voltage (V)
3300.091	0320019	63- 464-00-2060	121	460

Base module	Impeller No.	Gear type	Gear ratio	Imp.diam/Blade angle	Water temp °C
005	481 72 13				21

TEST RESULTS

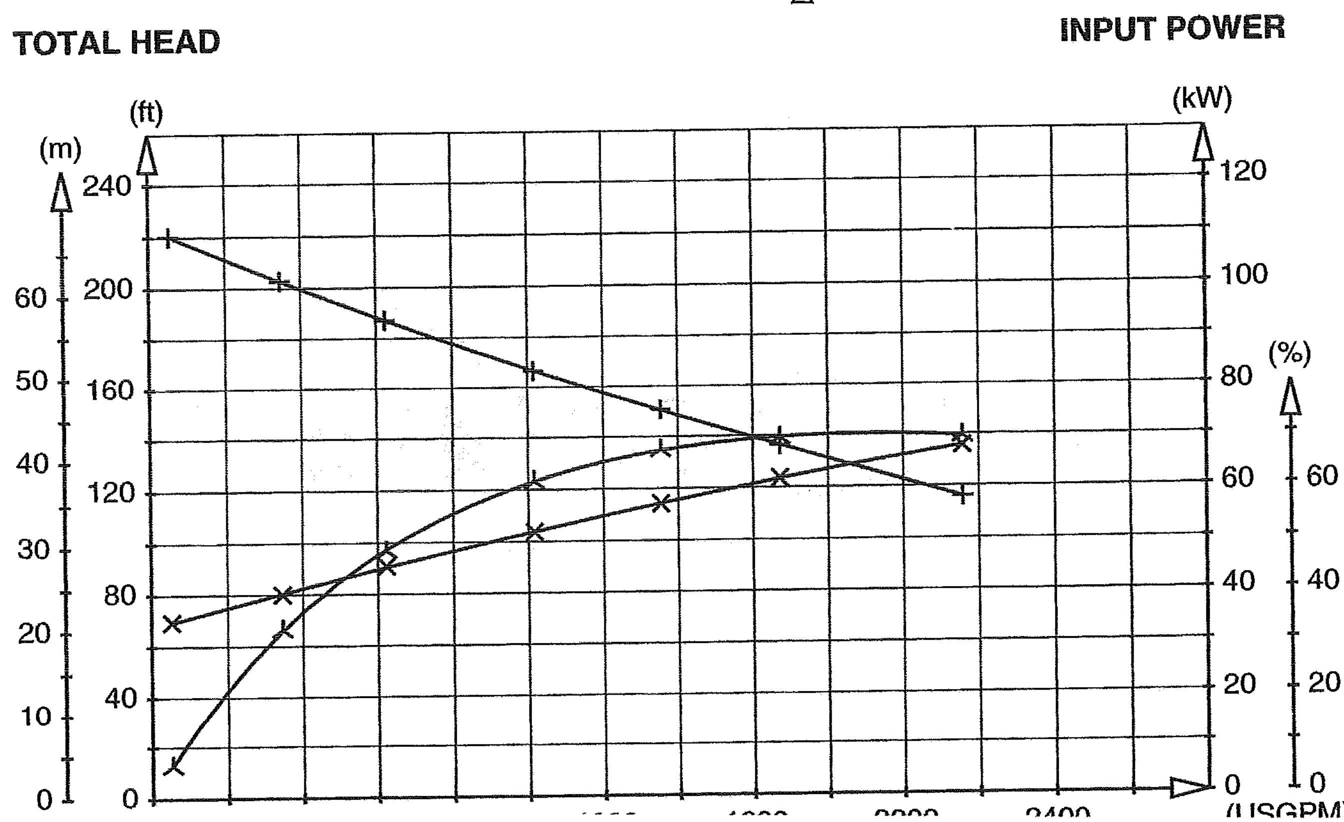
Pump total head H (ft)	Volume rate of flow Q (USGpm)	Motor input power P (kW)	Voltage U (V)	Current I (A)	Overall efficiency η (%)
220.24	52.4	34.66	461	59.7	6.28
202.83	345.9	39.97	461	65.7	33.11
187.05	623.3	45.42	460	72.8	48.42
166.45	1016.7	51.85	460	81.3	61.57
150.69	1353.7	57.08	459	87.7	67.42
136.19	1672.1	61.72	459	93.9	69.60
116.03	2157.5	67.79	458	102.4	69.67

Accepted after	Test facility		Test date	Time	Chief tester
HI	Lindas Sweden	Q1	03-03-27	14:02	2050

482533-1

PLOTTED TEST RESULTS

Measured point : + = Q/H, × = Q/P   Duty point : ◇ = Q/H, □ = Q/P, △ = Q/ETA overall   Calculated point : 人 = Q/ETA overall 1



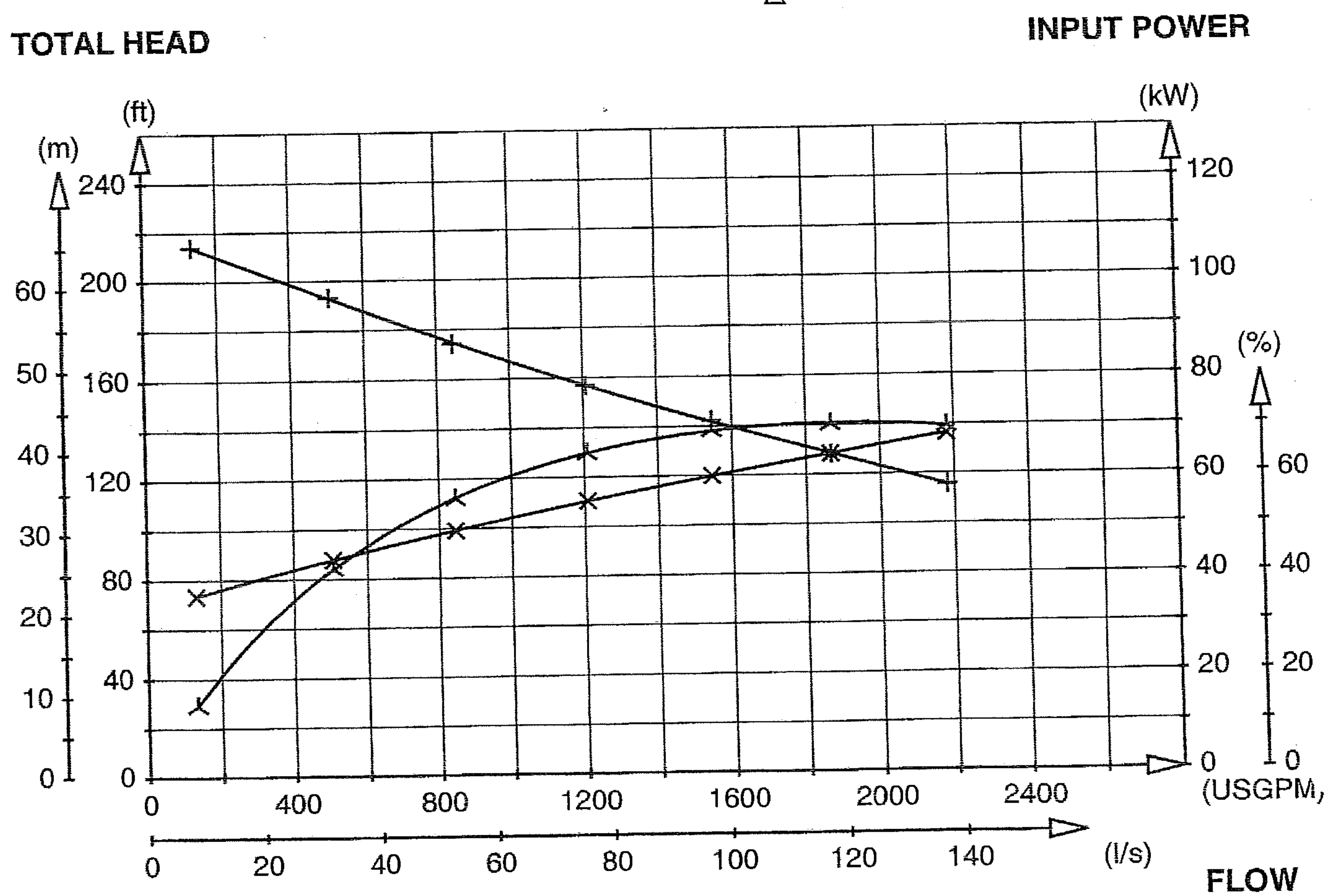
Serial No.					
3300.091	0320018	63- 464-00-2060		121	460
Base module	Impeller No.	Gear type	Gear ratio	Imp.diam/Blade angle	Water temp °C
005	481 72 13				19

TEST RESULTS

Pump total head H (ft)	Volume rate of flow Q (USGpm)	Motor input power P (kW)	Voltage U (V)	Current I (A)	Overall efficiency η (%)
213.92	132.5	36.68	462	62.4	14.58
193.29	507.1	43.94	461	70.6	42.08
174.19	845.2	49.56	462	77.7	56.03
157.07	1208.3	55.17	462	84.9	64.90
142.37	1544.8	59.93	462	91.3	69.23
128.20	1865.2	64.16	462	97.0	70.30
115.60	2180.5	68.18	462	102.6	69.75

Accepted after	Test facility	Test date	Time	Chief tester
HI	Lindas Q2 Sweden	03-03-26	07:03	

PLOTTED TEST RESULTS Measured point : + = Q/H, × = Q/P; Duty point : ◇ = Q/H, □ = Q/P, △ = Q/ETA overall; Calculated point : ⅄ = Q/ETA overall 1



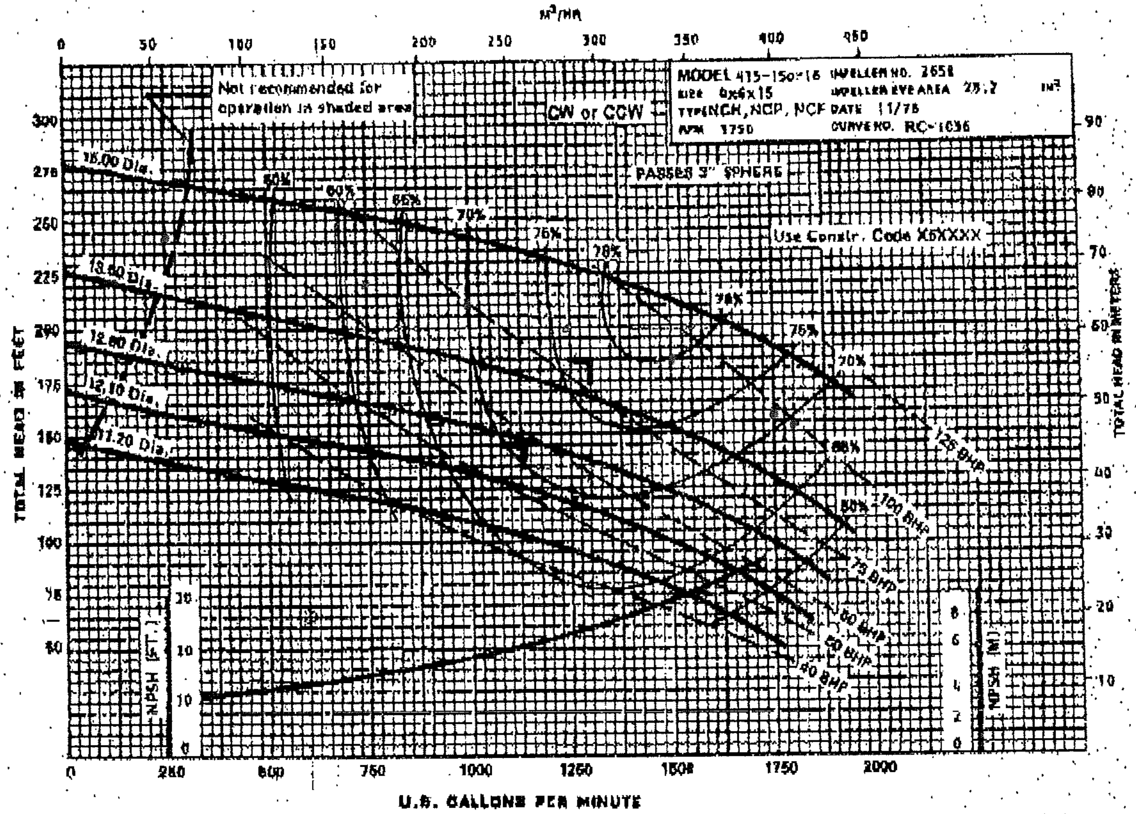
Rowntree

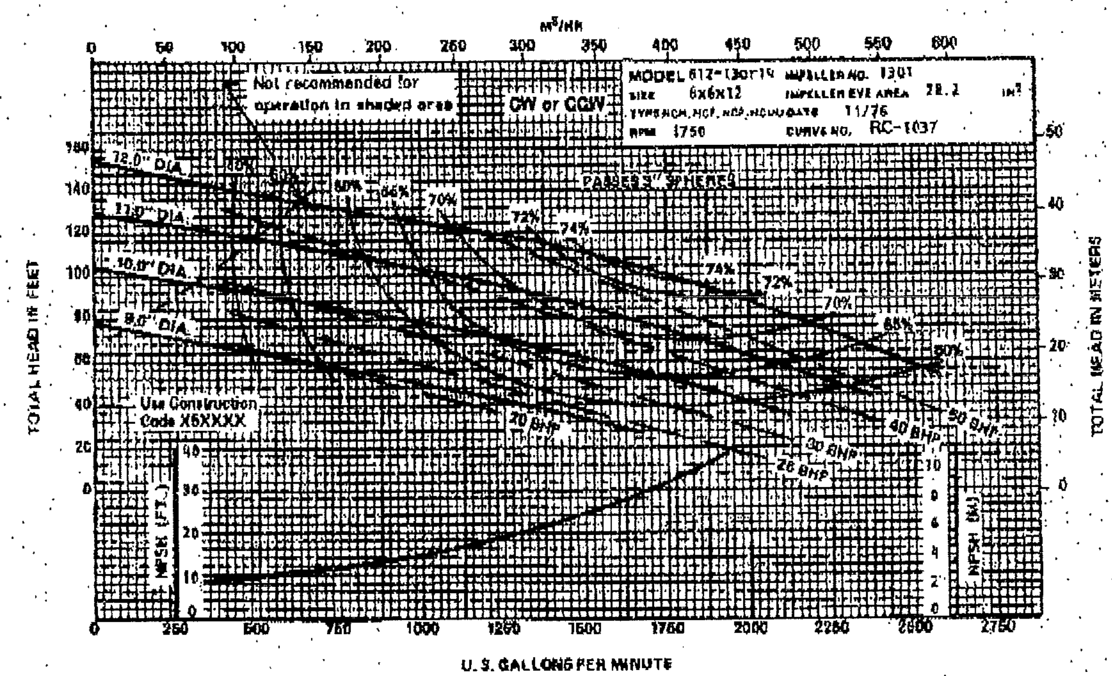
Lift Station

PACO

PERFORMANCE CURVES
PACO® NON-CLOG PUMPS, DRY PIT

1750 RPM





PACO
PACIFIC PUMPING COMPANY
DIVISION OF BALTIMORE AIRCOIL COMPANY, INC. A MERCK & CO., INC. SUBSIDIARY
P.O. Box 12924 • 845-92nd Ave. • Oakland, CA 94604

PACIFIC PUMPING COMPANY OF CANADA
DIVISION OF BALTIMORE AIRCOIL INTERAMERICAN CORPORATION
35 Sinclair Ave. • Georgetown, Ontario, Canada

1/76 Rerun 6/77
Printed in U.S.A.

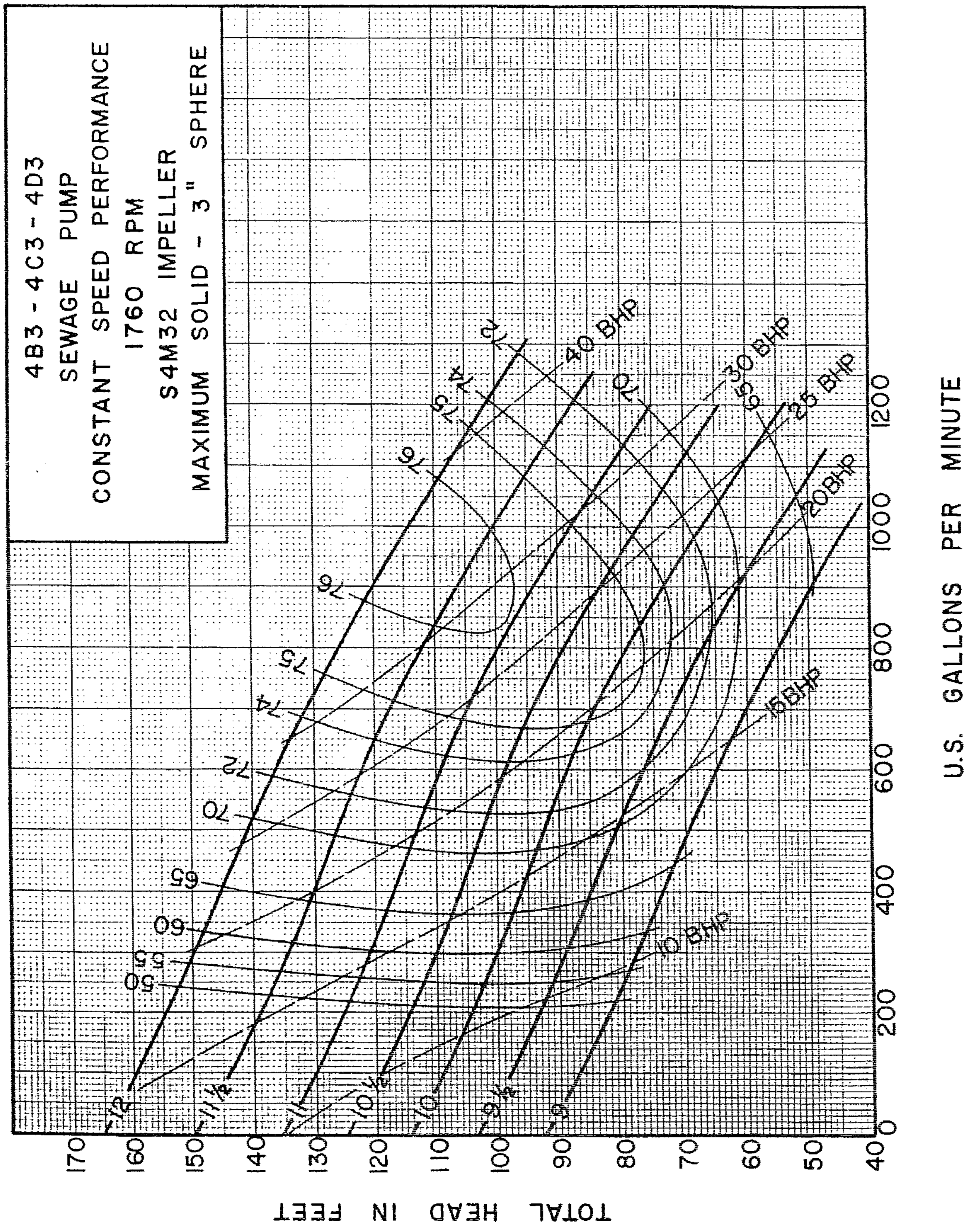
Skyline

Lift Station

Smith & Loveless Engineering Data

Main Plant: Lenexa, Kansas 66215

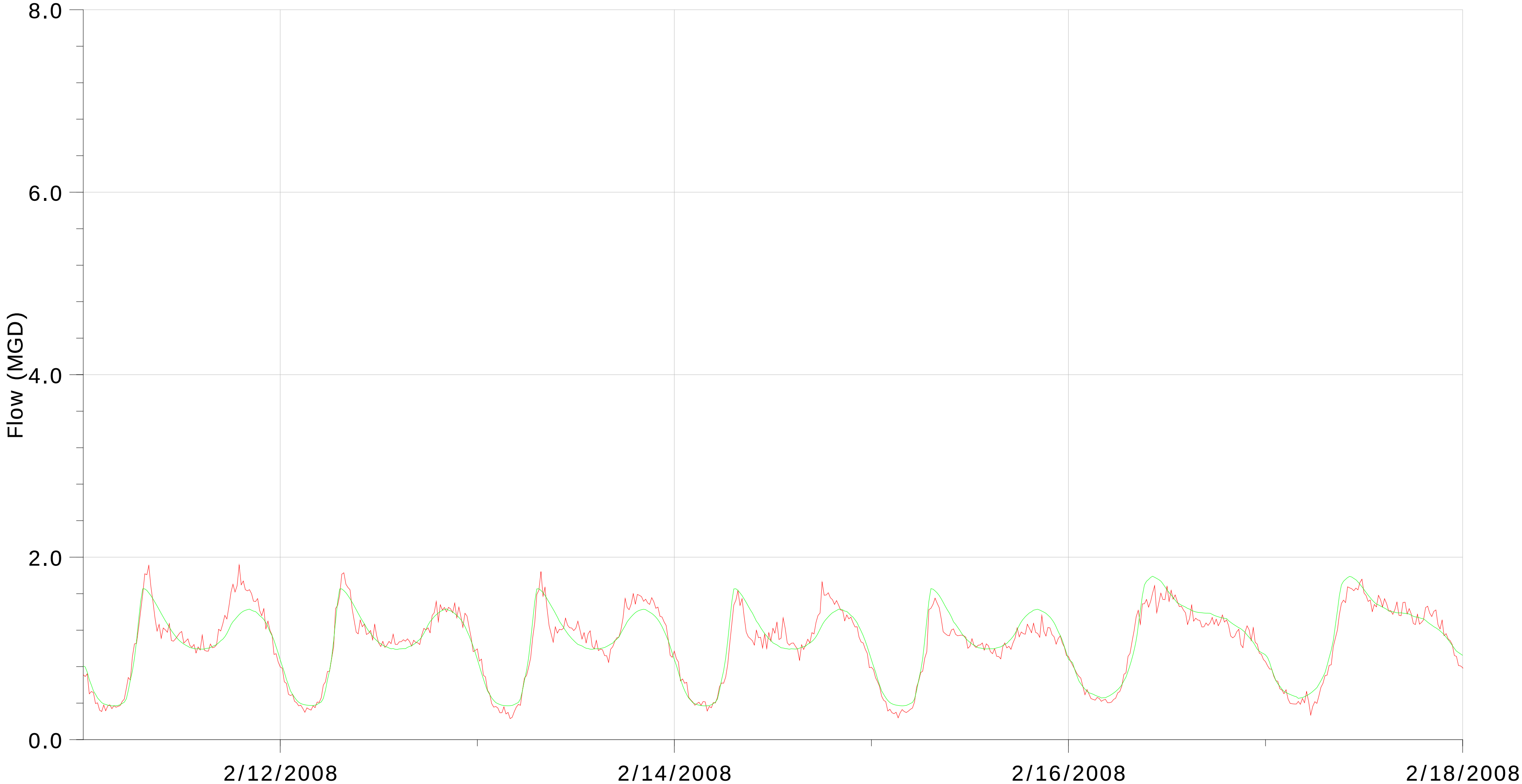
Pump Performance Curves
Constant Speed
4B3/4C3/4D3
1760 RPM



Appendix C
Dry Weather Flow Calibration Results

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 1 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

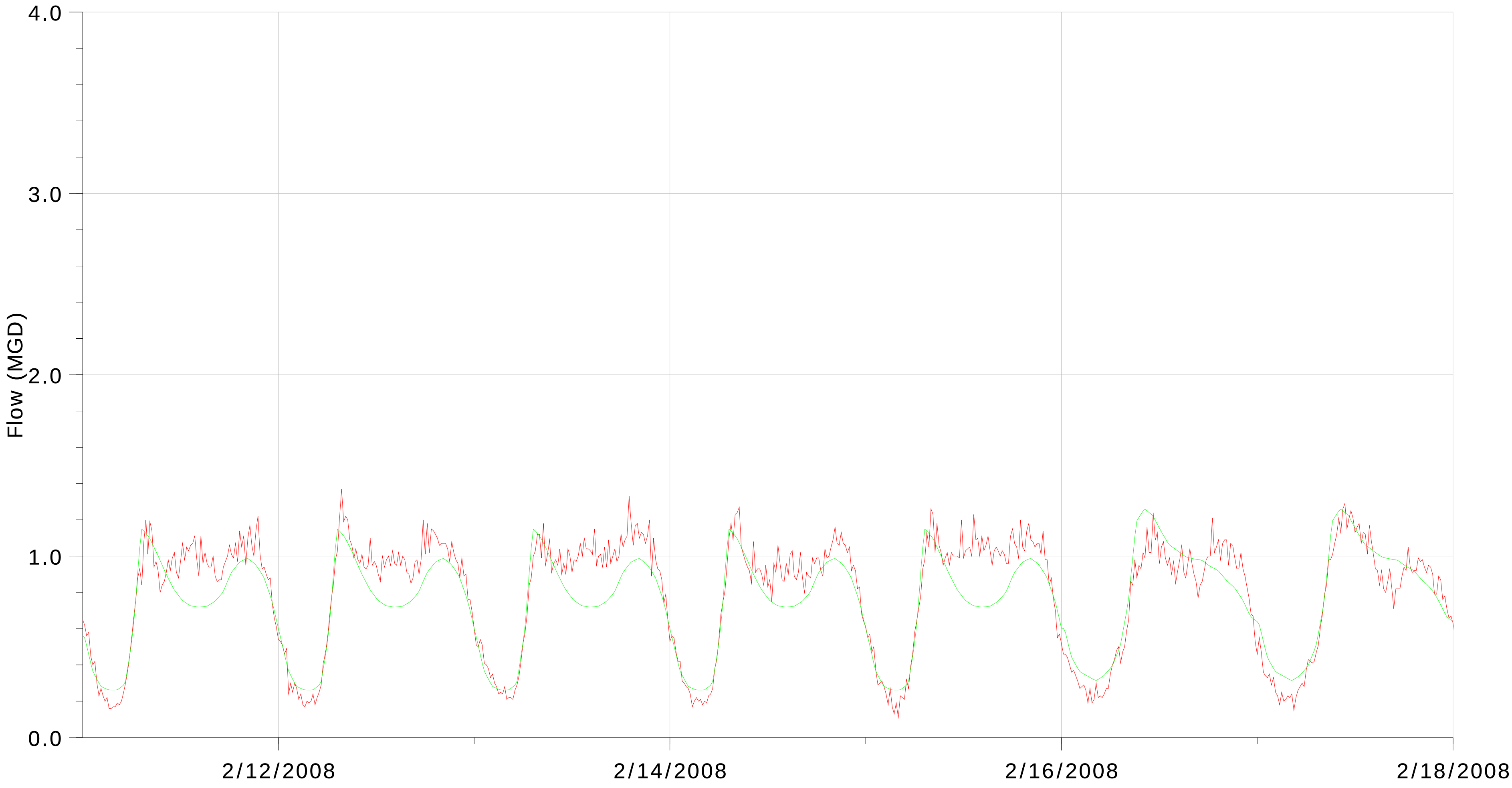
Flow Survey Location (Obs.) FM1 MH-C04-119.1



	Flow (MGD) Min	Max	Volume (US Mgal)
Obs.	0.230	1.913	7.358
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.370	1.790	7.456

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 2 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

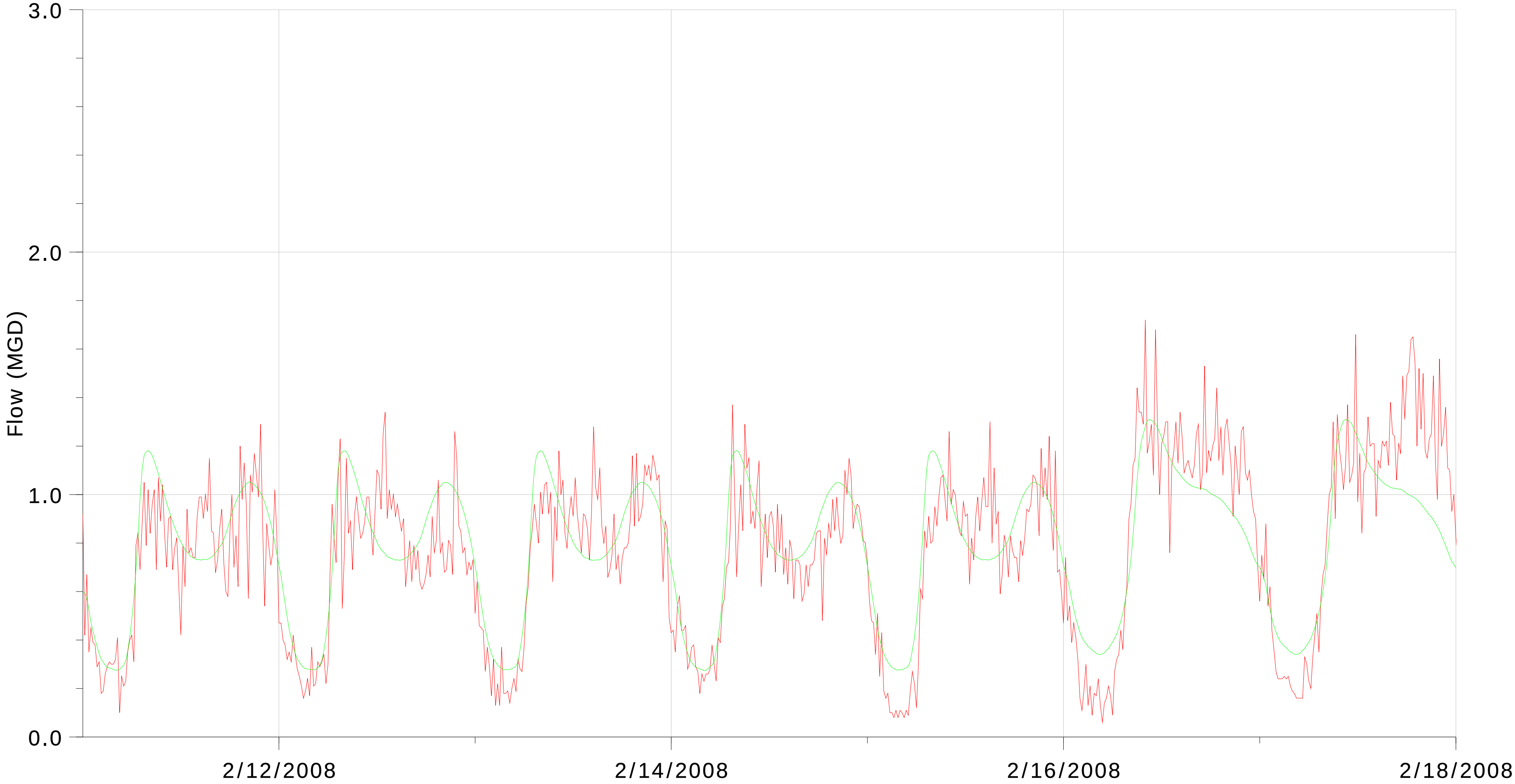
Flow Survey Location (Obs.) FM2 MH-D03-064.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Obs.	0.110	1.370	5.577
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.262	1.258	5.231

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 3 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

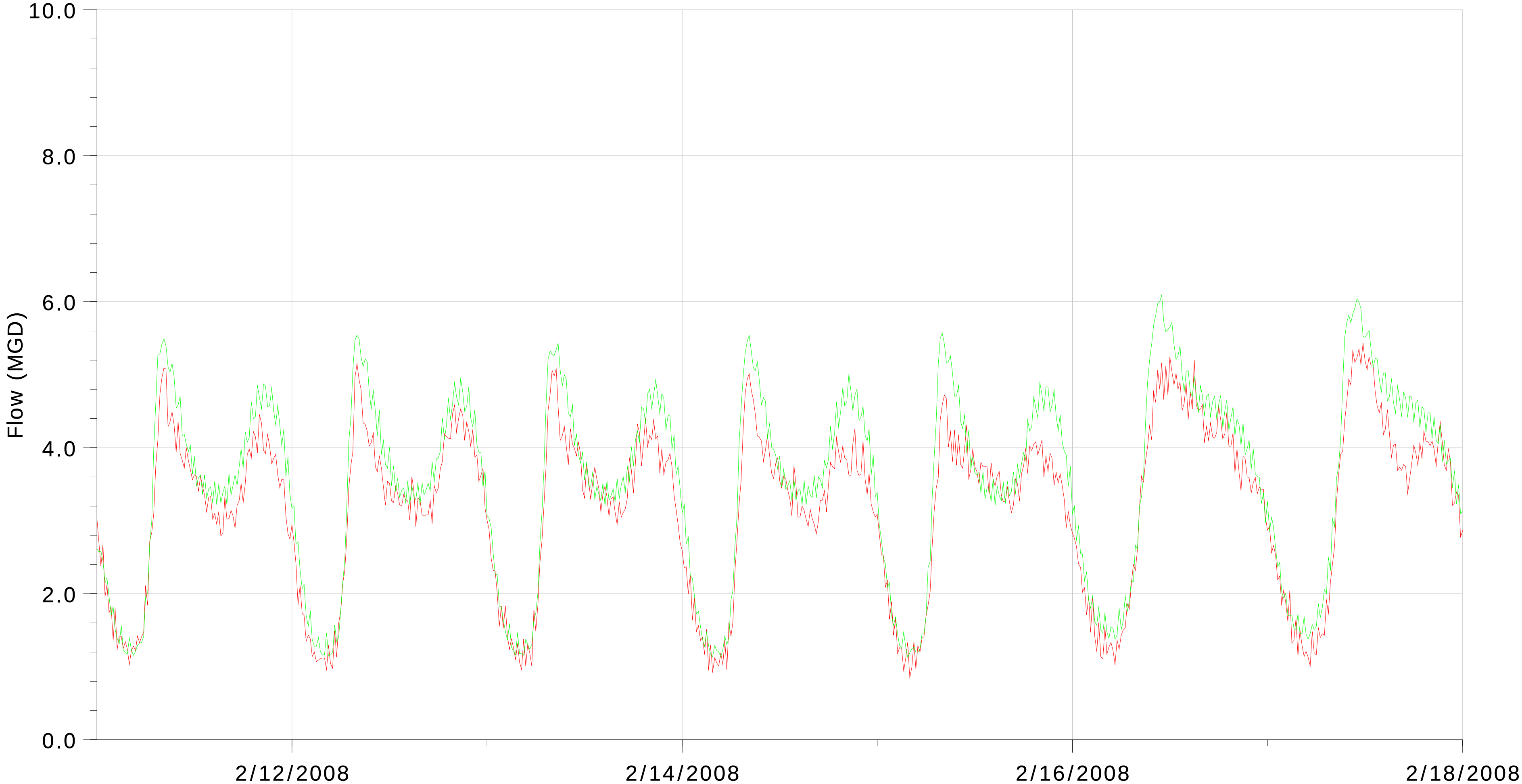
Flow Survey Location (Obs.) FM3 MH-C04-134.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Obs.	0.060	1.720	5.419
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.273	1.310	5.507

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 4 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

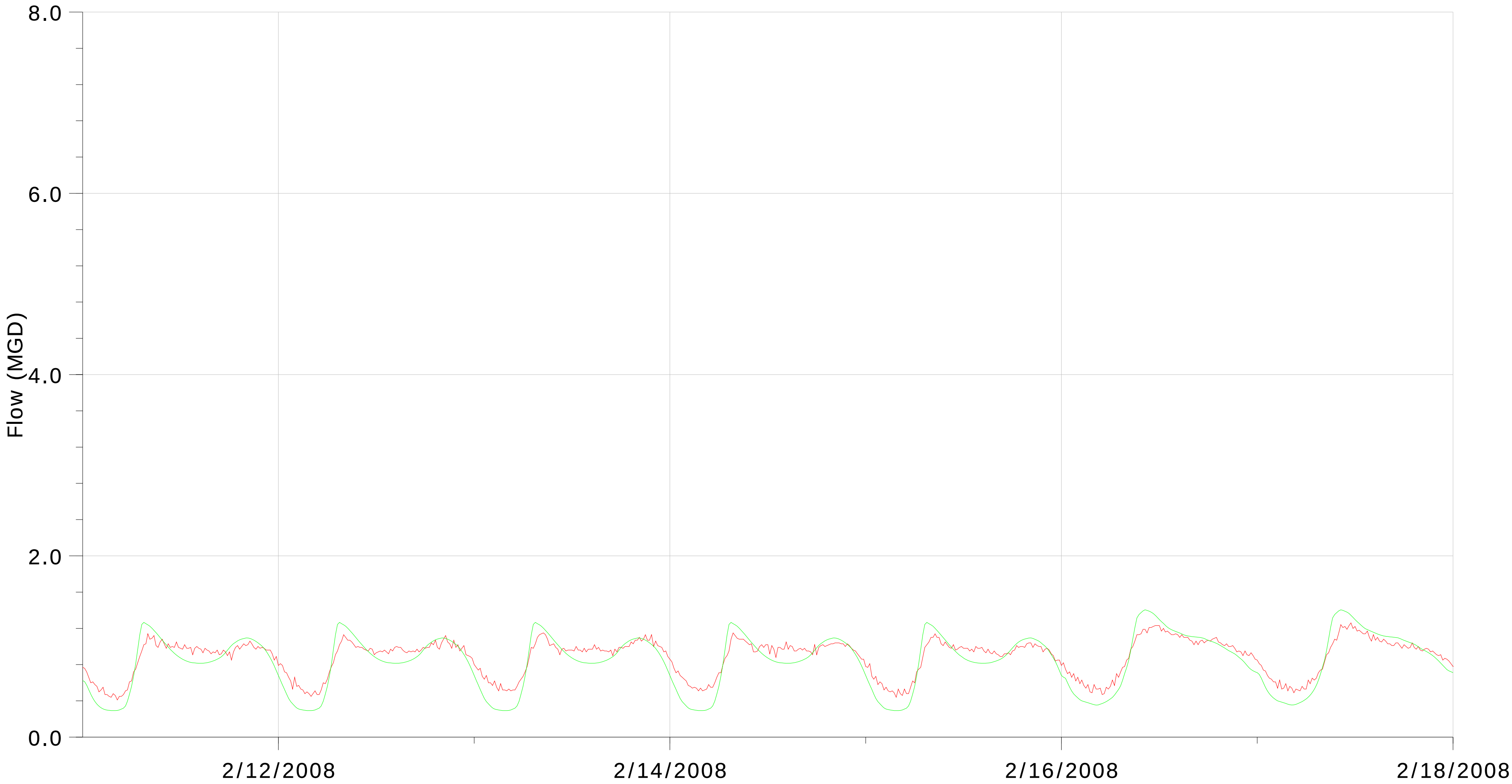
Flow Survey Location (Obs.) FM4 MH-C05-154.1



	Flow (MGD) Min	Max	Volume (US Mgal)
Obs.	0.850	5.430	22.344
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	1.128	6.091	24.750

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 5 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

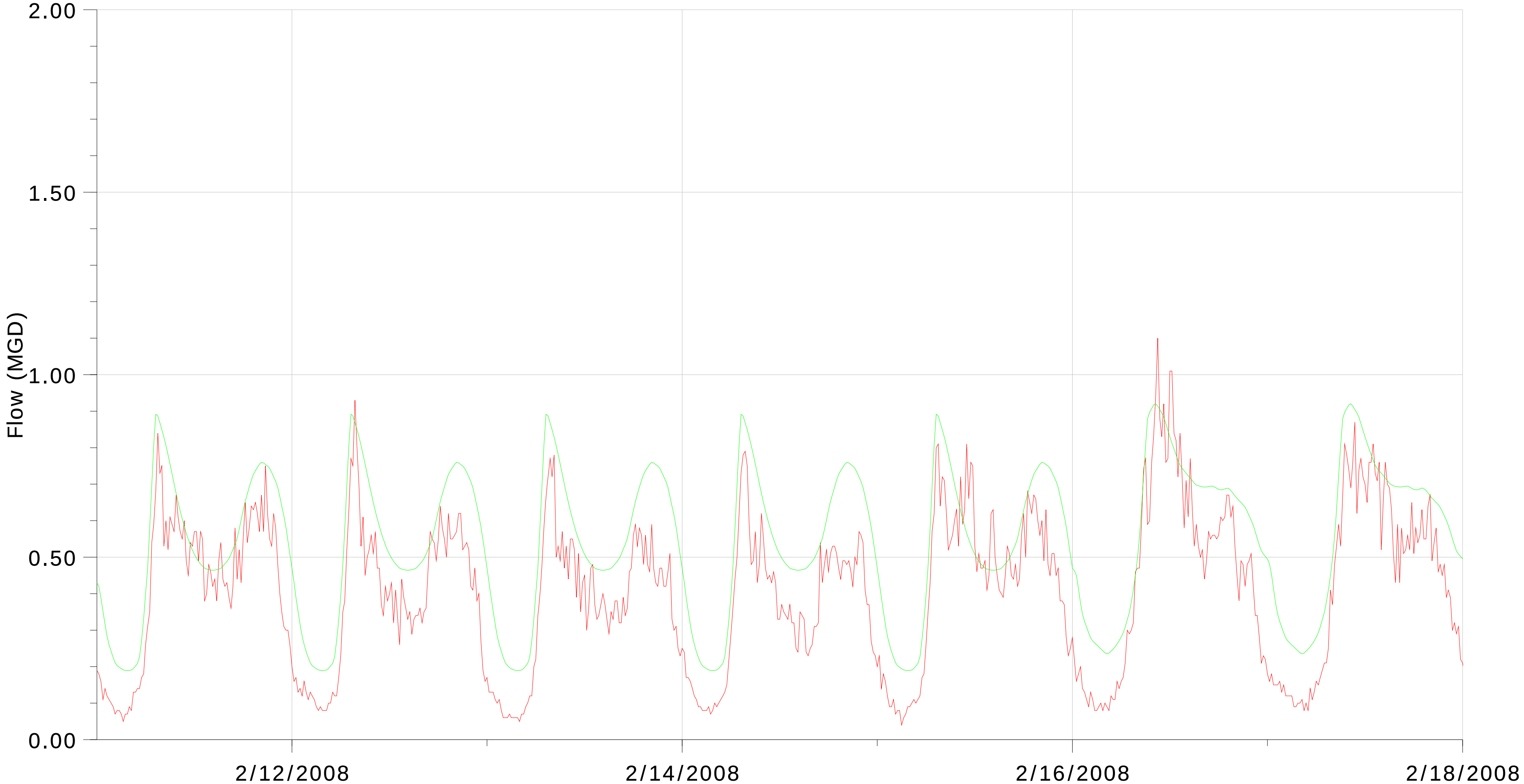
Flow Survey Location (Obs.) FM5 MH-C05-098.1



	Flow (MGD) Min	Max	Volume (US Mgal)
Obs.	0.410	1.260	6.203
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.293	1.404	5.852

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 6 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

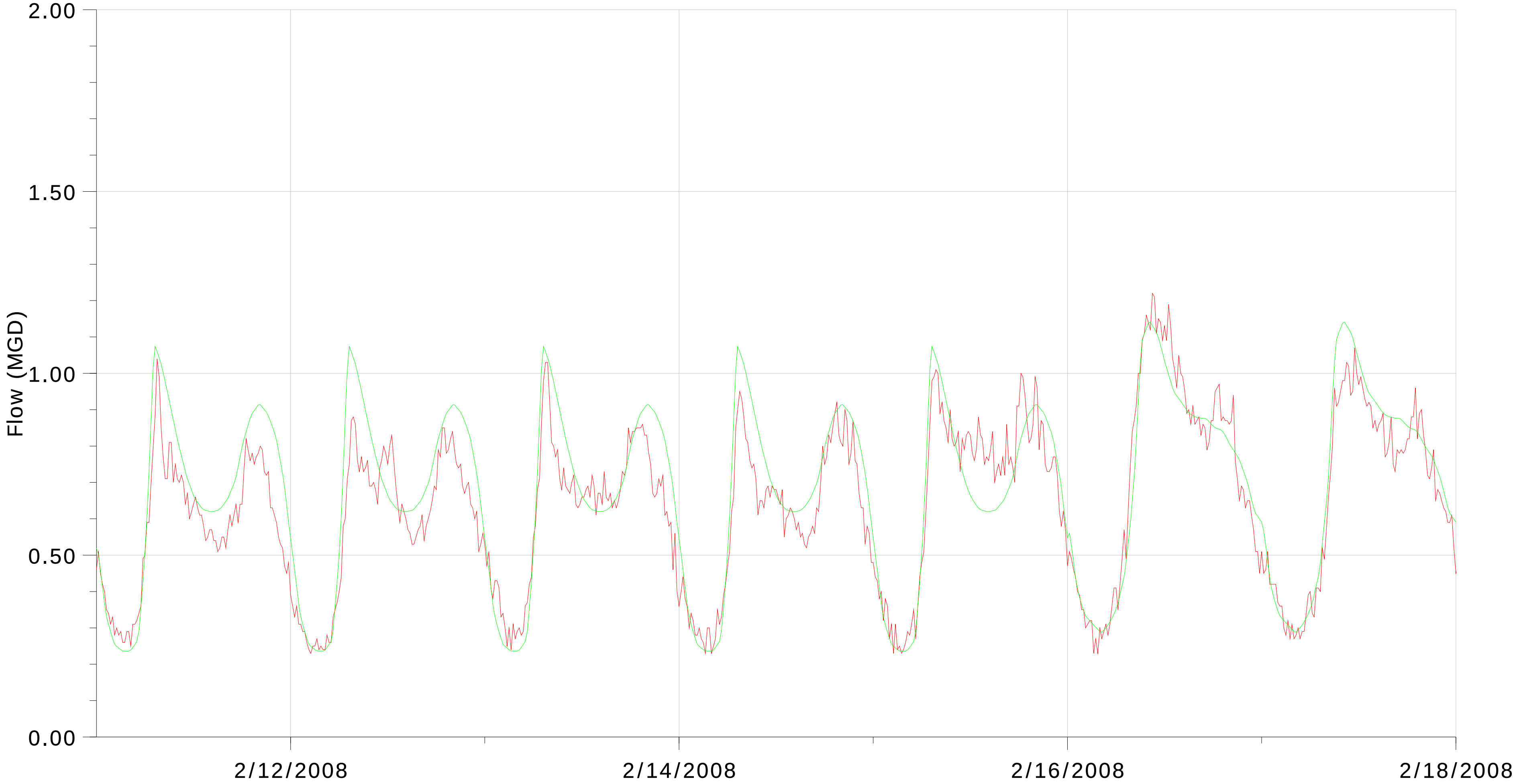
Flow Survey Location (Obs.) FM6 MH-E07-091.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Obs.	0.040	1.100	2.848
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.190	0.921	3.807

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 7 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

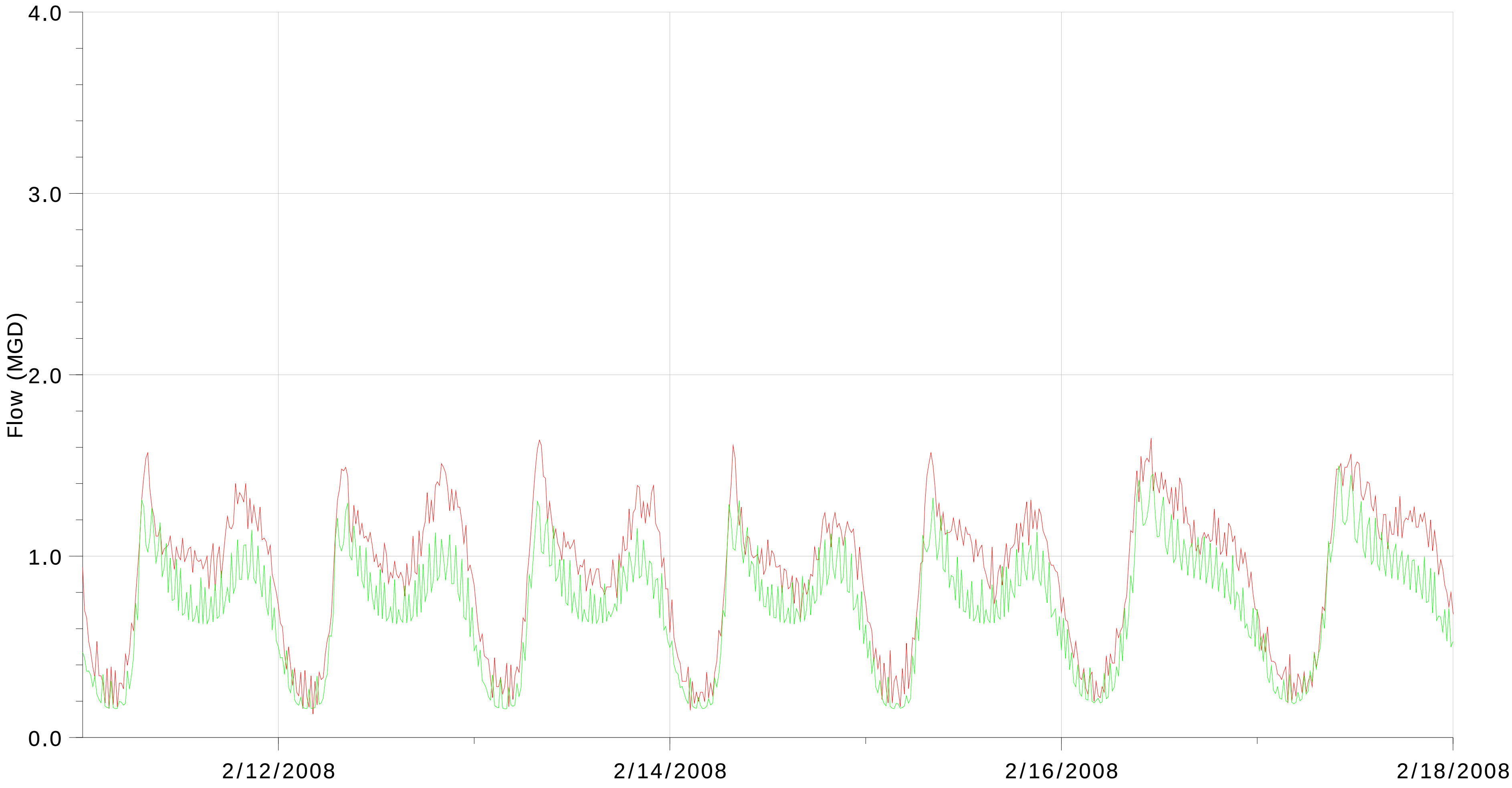
Flow Survey Location (Obs.) FM7 MH-D08-014.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Obs.	0.230	1.220	4.483
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.236	1.141	4.734

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 8 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

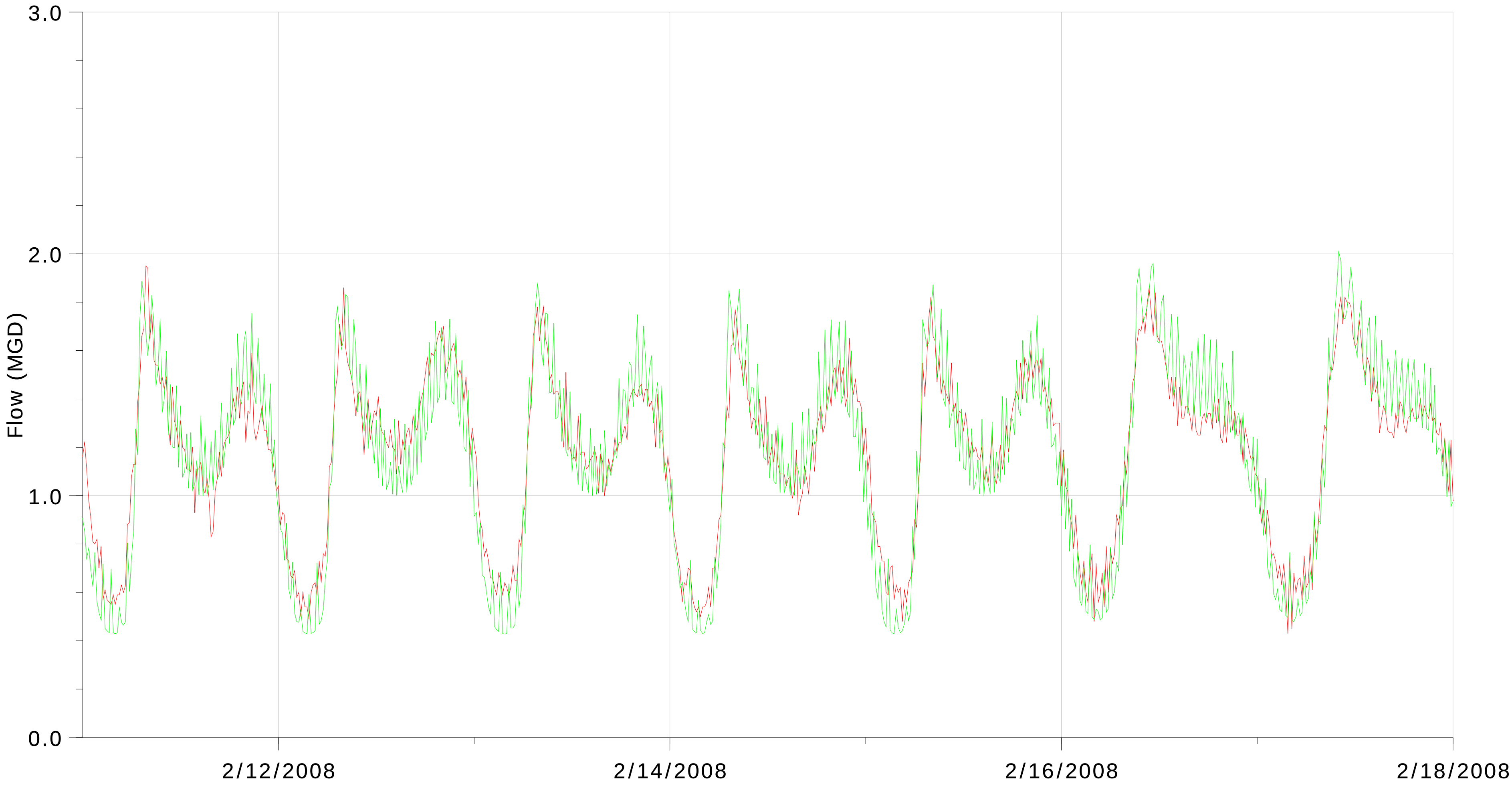
Flow Survey Location (Obs.) FM8 MH-D09-007.1



	Flow (MGD) Min	Max	Volume (US Mgal)
Obs.	0.130	1.650	6.373
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.158	1.494	5.018

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 9 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

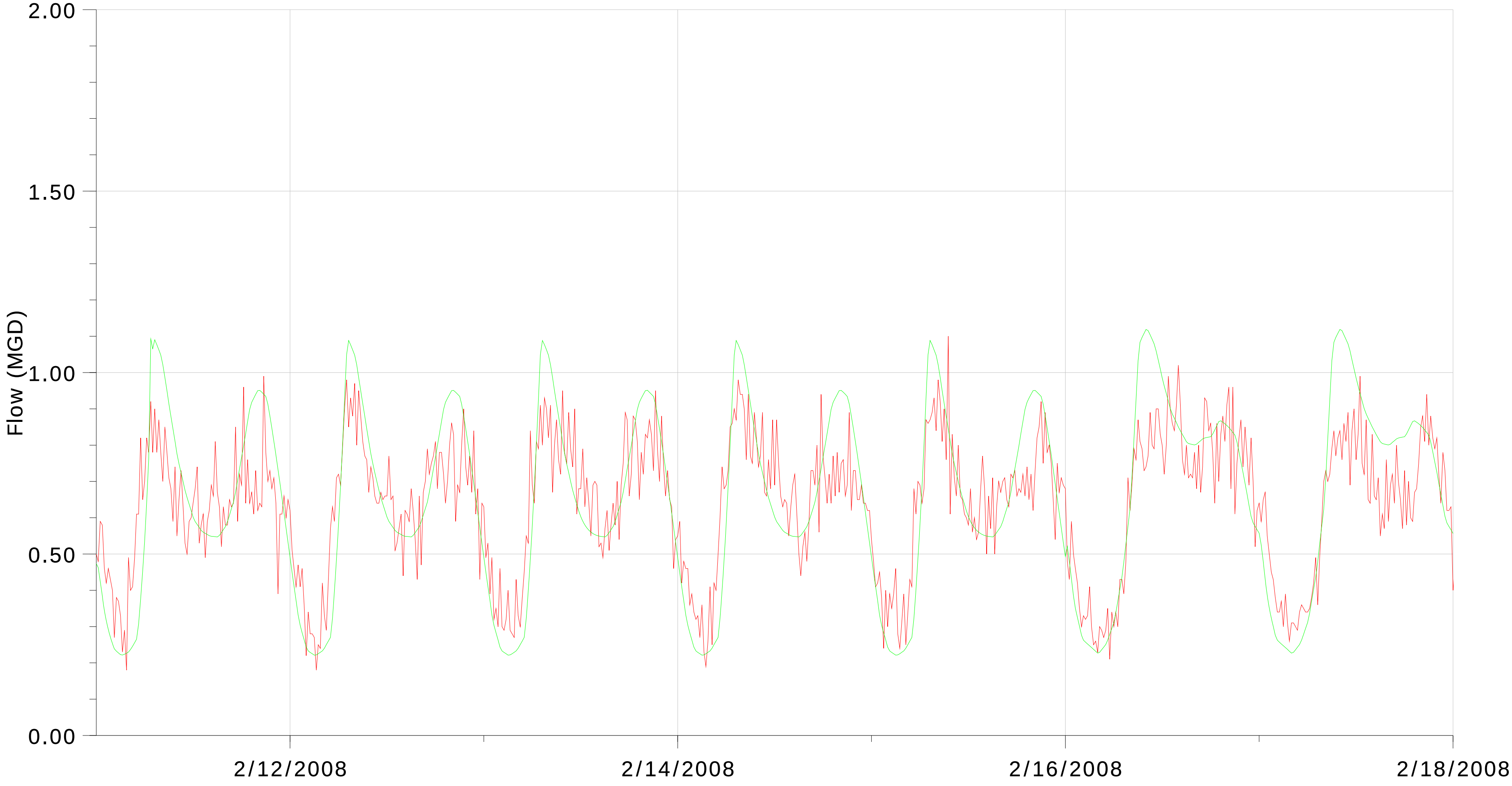
Flow Survey Location (Obs.) FM9 MH-D09-034.1



	Flow (MGD) Min	Max	Volume (US Mgal)
Obs.	0.430	1.950	8.267
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.428	2.008	8.194

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 10 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

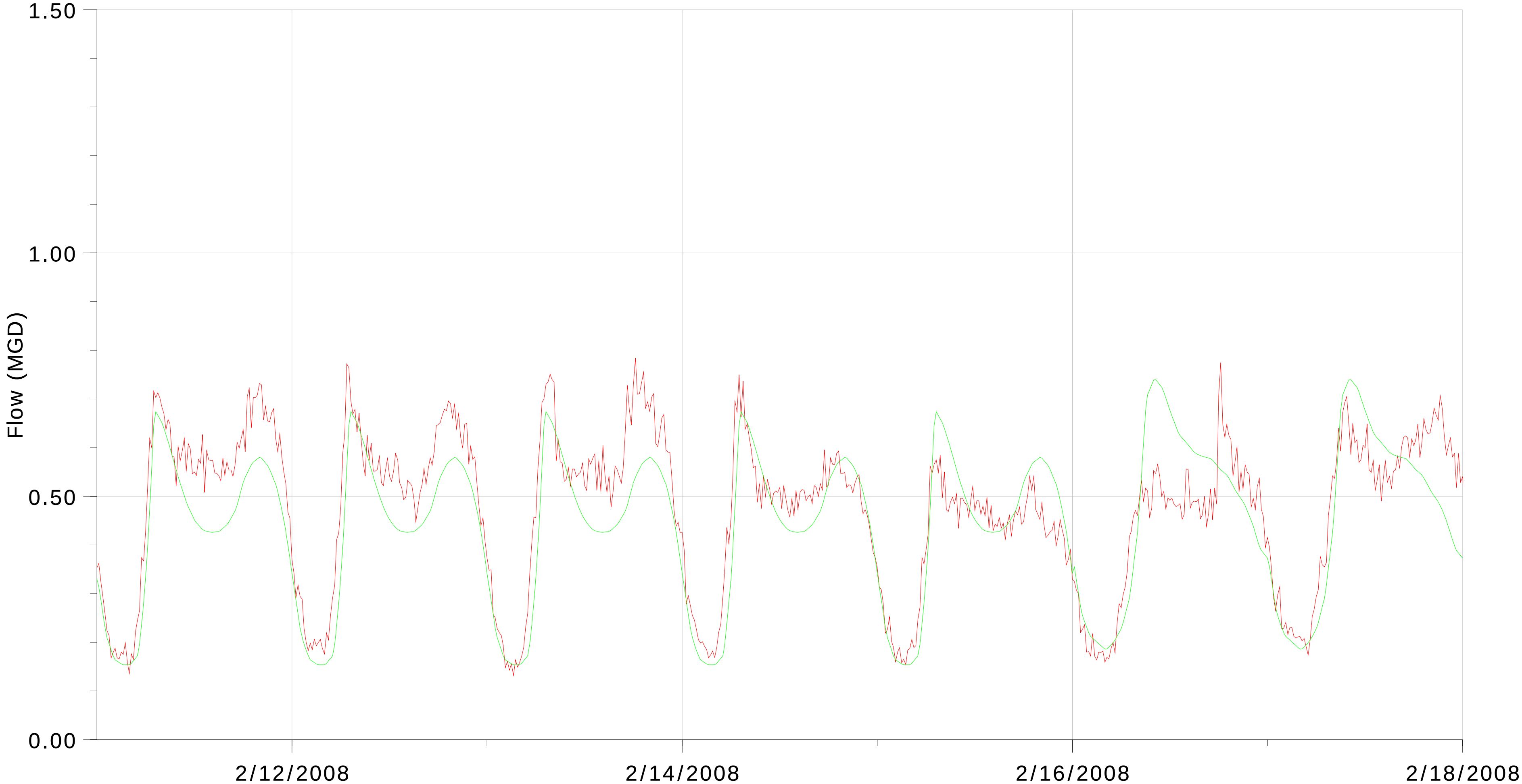
Flow Survey Location (Obs.) FM10 MH-E13-053.1



	Flow (MGD)		
	Min	Max	Volume (US Mgal)
Obs.	0.180	1.100	4.481
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.221	1.119	4.561

Observed / Predicted Plot Produced by ahope (4/9/2009 8:21:42 PM) Page 11 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>DWF Calibration>DWF_Existing!!>DWF (4/8/2009 2:56:12 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/9/2009 8:21:30 PM)

Flow Survey Location (Obs.) FM11 MH-D06-134.1

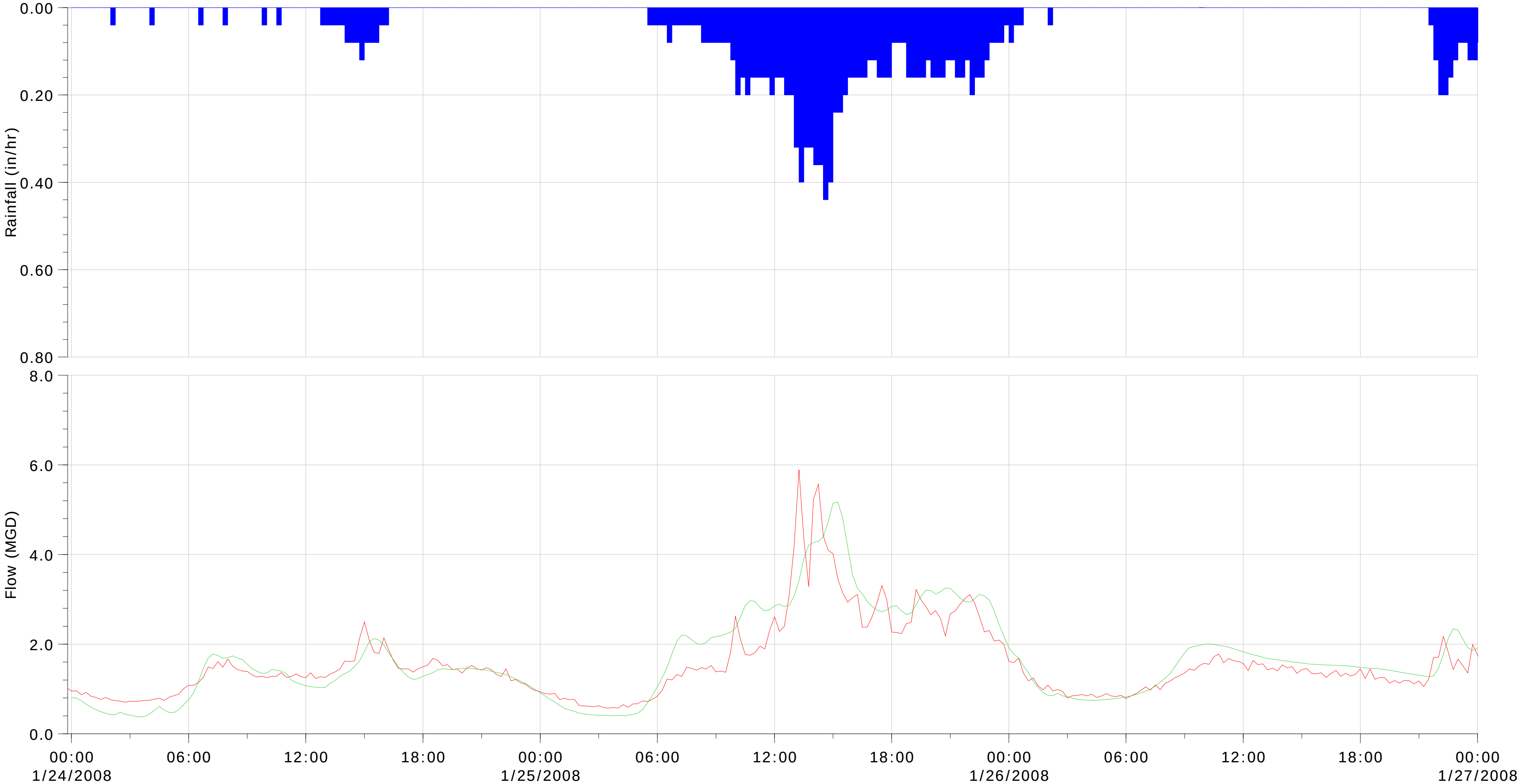


	Flow (MGD) Min	Max	Volume (US Mgal)
Obs.	0.132	0.784	3.324
..._City_Catchment>DWF Calibration>DWF_Existing!!>DWF	0.154	0.740	3.083

Appendix D
Wet Weather Flow Calibration Results

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 1 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

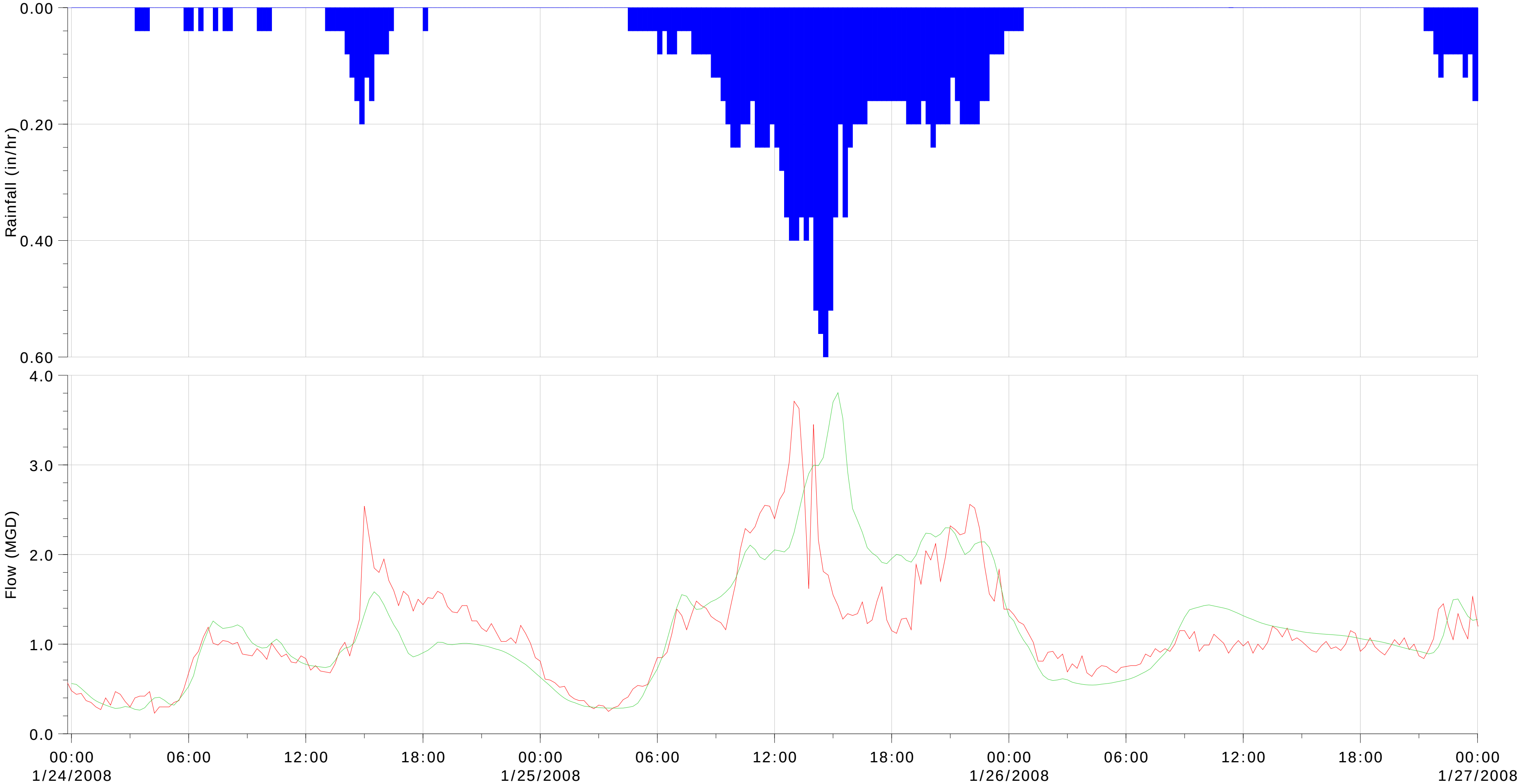
Flow Survey Location (Obs.) FM1 MH-C04-119.1, Rainfall Profile: 1



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	3.452	0.440	0.048			
Obs.				0.574	5.891	4.691
...torm_Existing>2008_Rainfall_inhr				0.376	5.169	4.954

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 2 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

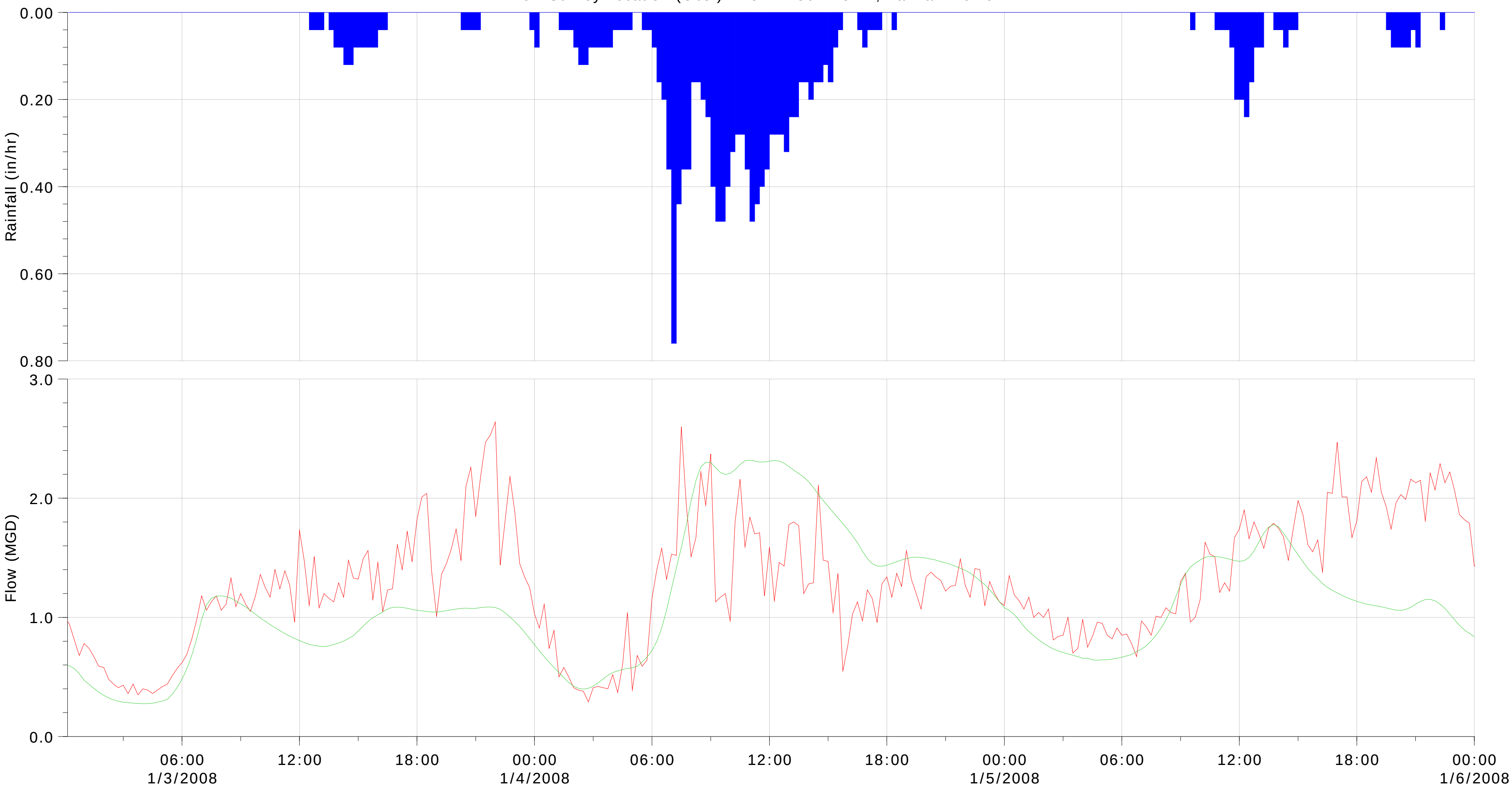
Flow Survey Location (Obs.) FM2 MH-D03-064.1, Rainfall Profile: 2



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	4.483	0.600	0.063			
Obs.				0.230	3.710	3.441
...torm_Existing>2008_Rainfall_inhr				0.264	3.804	3.504

Observed / Predicted Plot Produced by ahope (4/8/2009 5:20:33 PM) Page 3 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan4th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 3:02:41 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

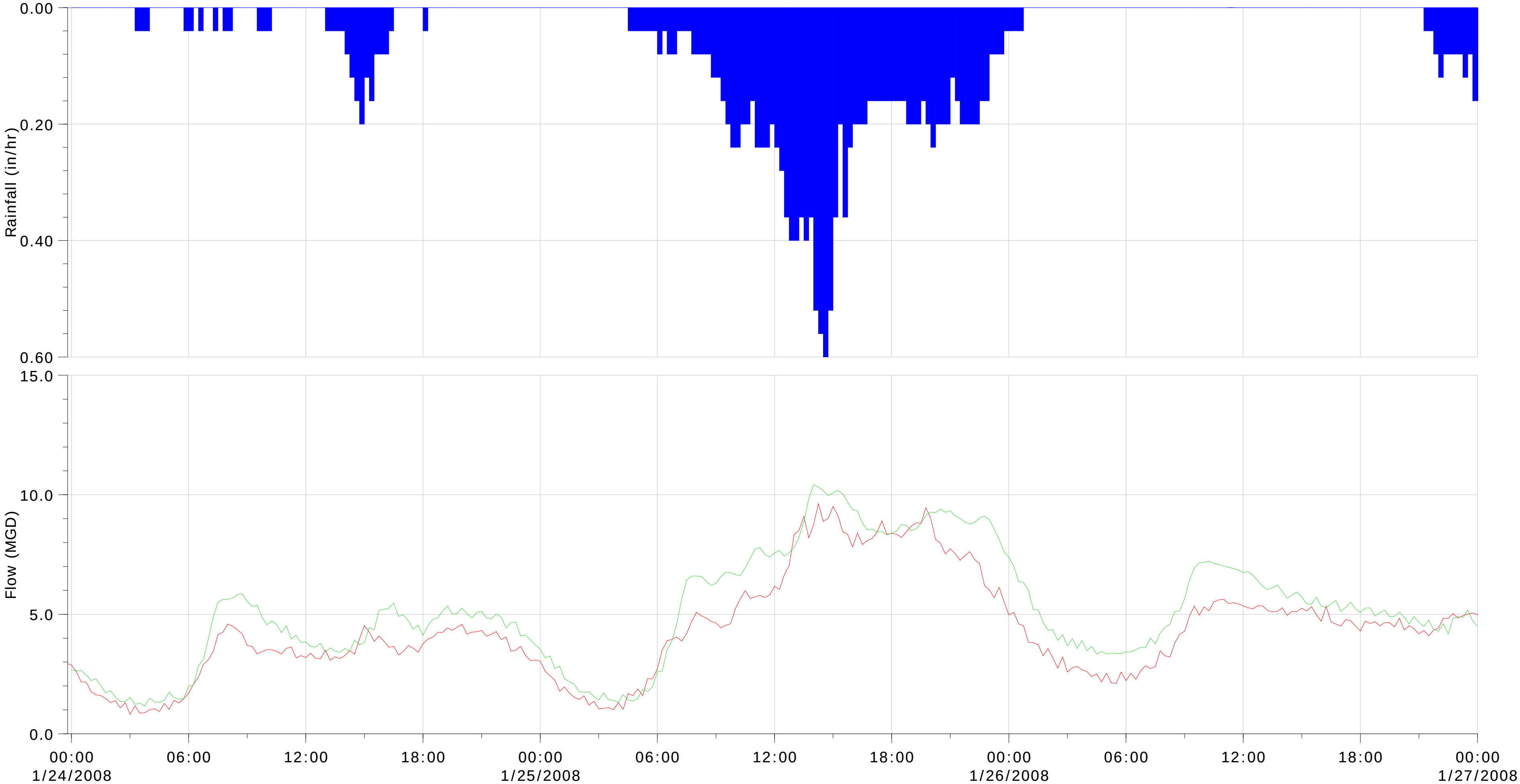
Flow Survey Location (Obs.) FM3 MH-C04-134.1, Rainfall Profile: 1



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	3.920	0.760	0.054			
Obs.				0.290	2.640	3.902
...torm_Existing>2008_Rainfall_inhr				0.276	2.319	3.375

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 4 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

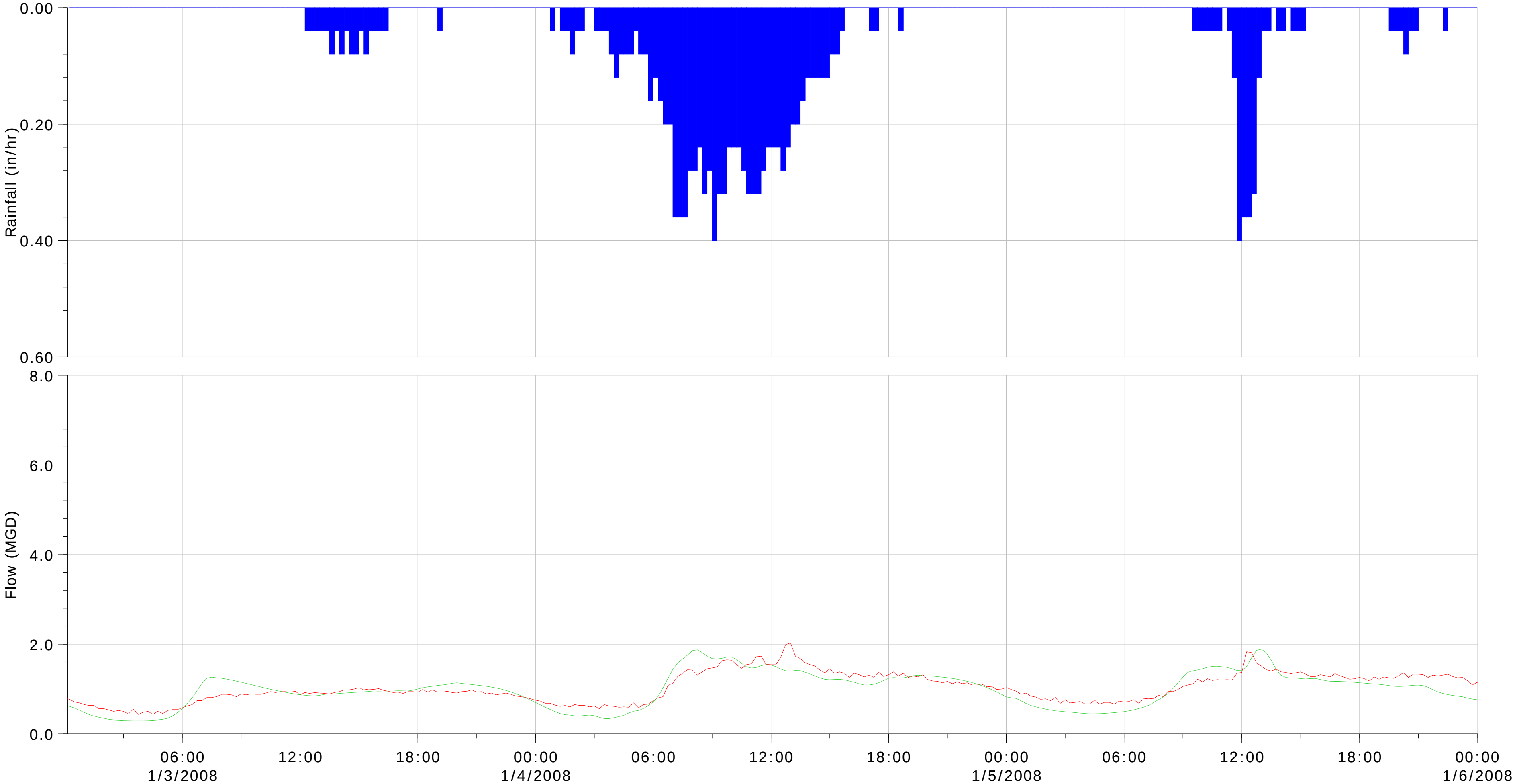
Flow Survey Location (Obs.) FM4 MH-C05-154.1, Rainfall Profile: 2



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	4.483	0.600	0.063			
Obs.				0.820	9.630	13.027
...torm_Existing>2008_Rainfall_inhr				1.166	10.429	15.423

Observed / Predicted Plot Produced by ahope (4/8/2009 5:20:33 PM) Page 5 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan4th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 3:02:41 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

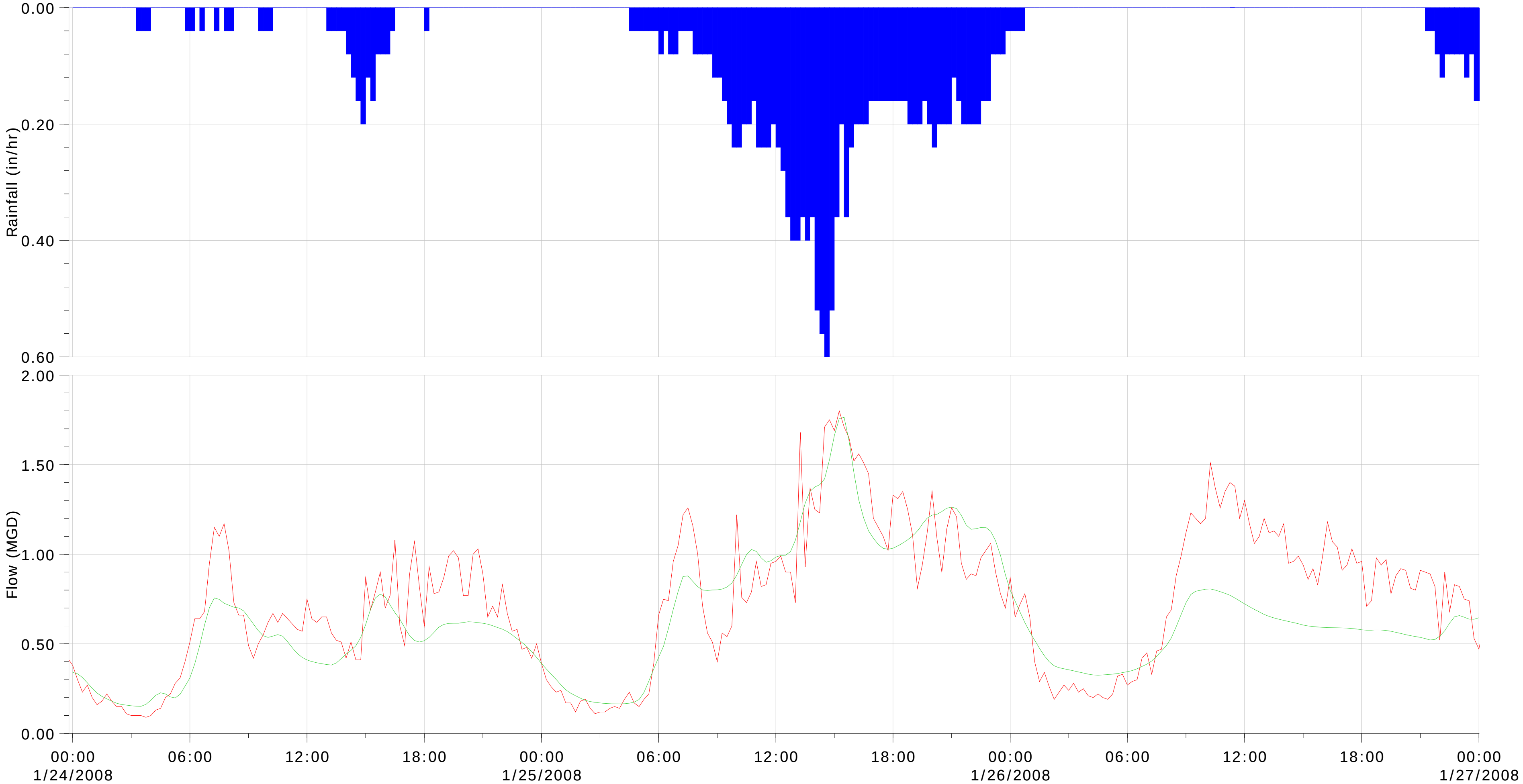
Flow Survey Location (Obs.) FM5 MH-C05-098.1, Rainfall Profile: 2



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	3.460	0.400	0.048			
Obs.				0.430	2.030	3.105
...torm_Existing>2008_Rainfall_inhr				0.293	1.888	2.953

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 6 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

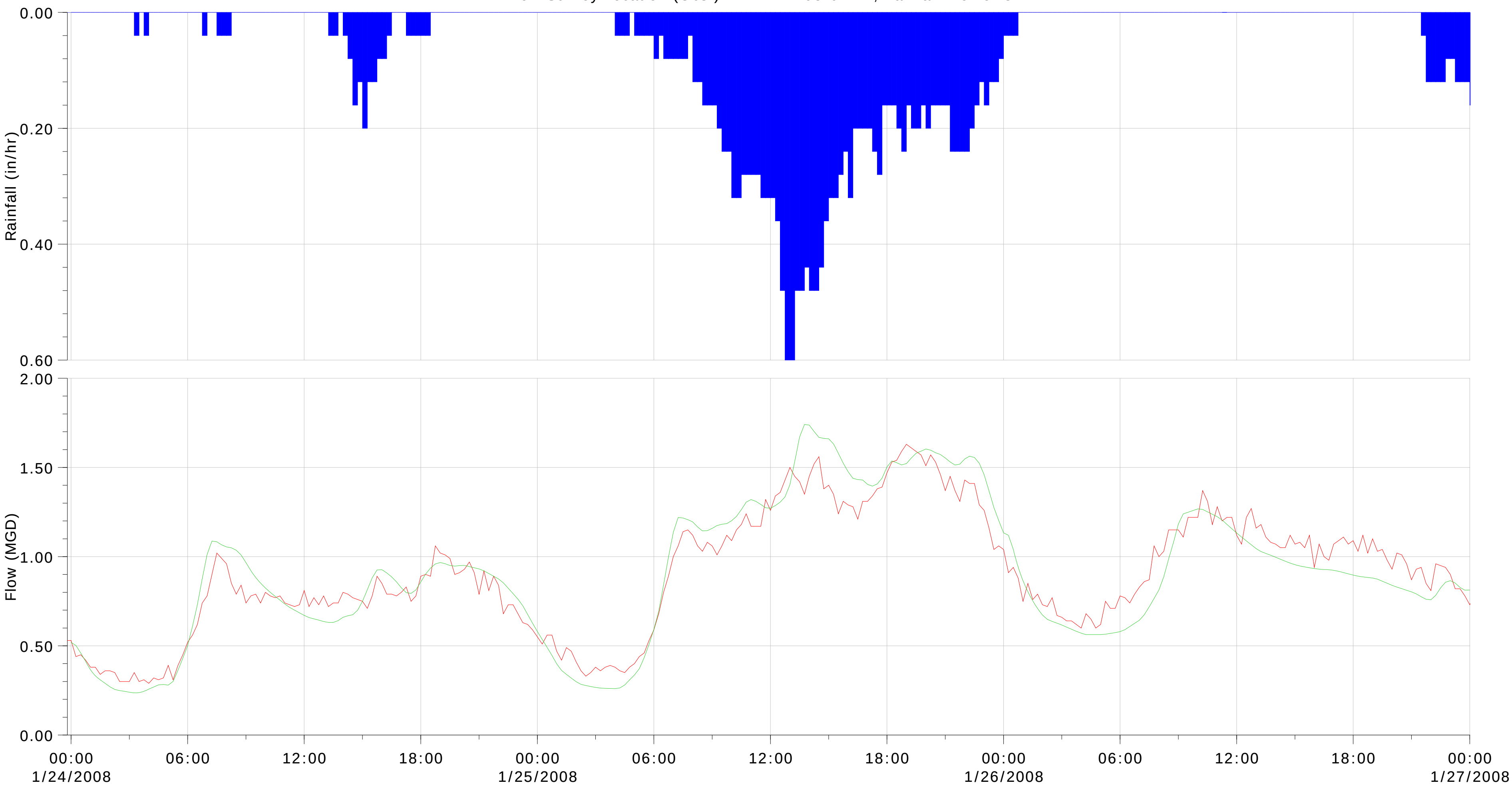
Flow Survey Location (Obs.) FM6 MH-E07-091.1, Rainfall Profile: 2



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	4.483	0.600	0.063			
Obs.				0.090	1.800	2.217
...torm_Existing>2008_Rainfall_inhr				0.152	1.764	1.907

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 7 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

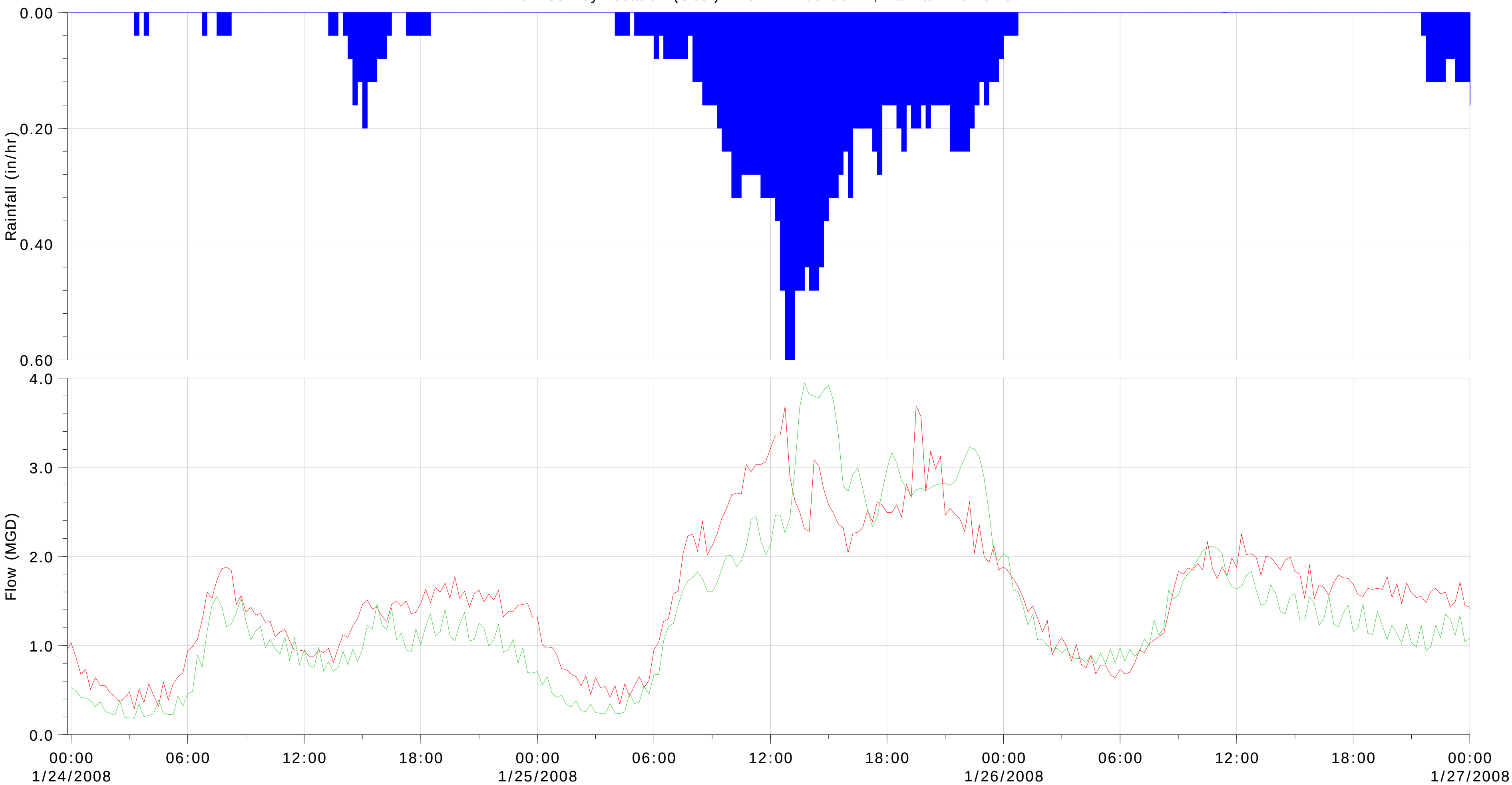
Flow Survey Location (Obs.) FM7 MH-D08-014.1, Rainfall Profile: 3



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	5.023	0.600	0.070			
Obs.				0.290	1.630	2.736
...torm_Existing>2008_Rainfall_inhr				0.237	1.741	2.709

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 8 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

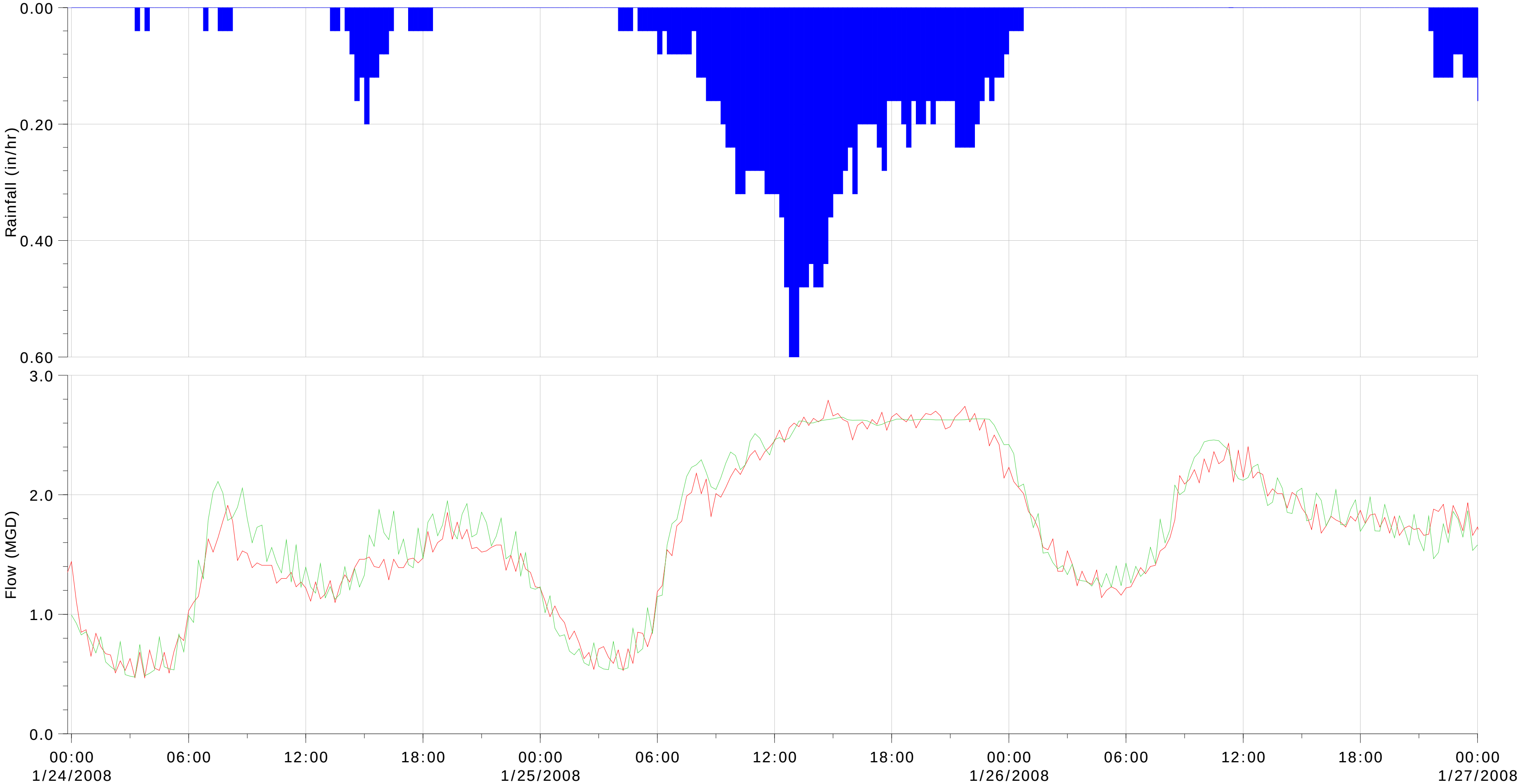
Flow Survey Location (Obs.) FM8 MH-D09-007.1, Rainfall Profile: 3



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	5.023	0.600	0.070			
Obs.				0.290	3.690	4.713
...torm_Existing>2008_Rainfall_inhr				0.183	3.940	4.169

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 9 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

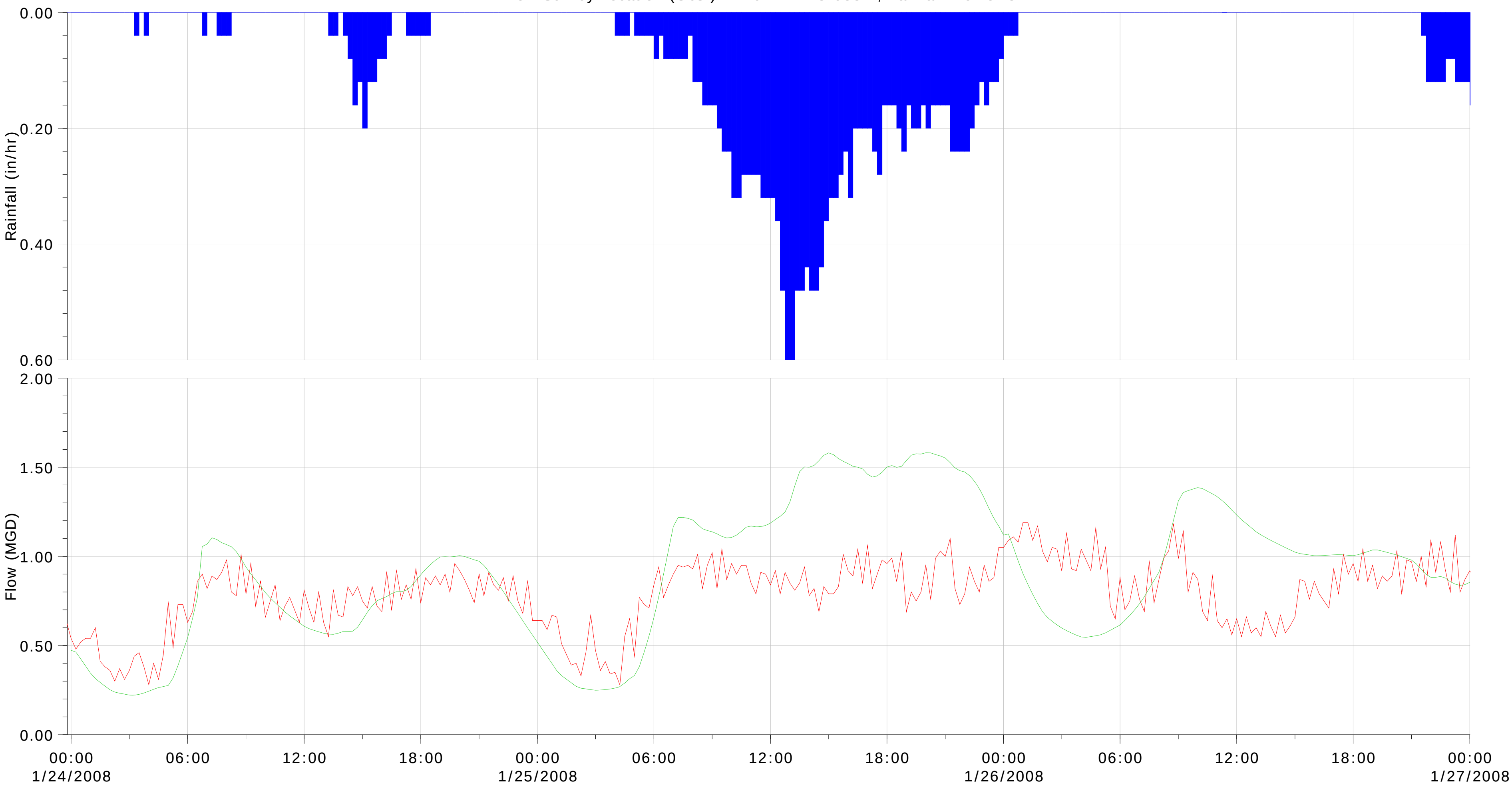
Flow Survey Location (Obs.) FM9 MH-D09-034.1, Rainfall Profile: 3



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	5.023	0.600	0.070			
Obs.				0.470	2.790	5.056
...torm_Existing>2008_Rainfall_inhr				0.475	2.650	5.148

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 10 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

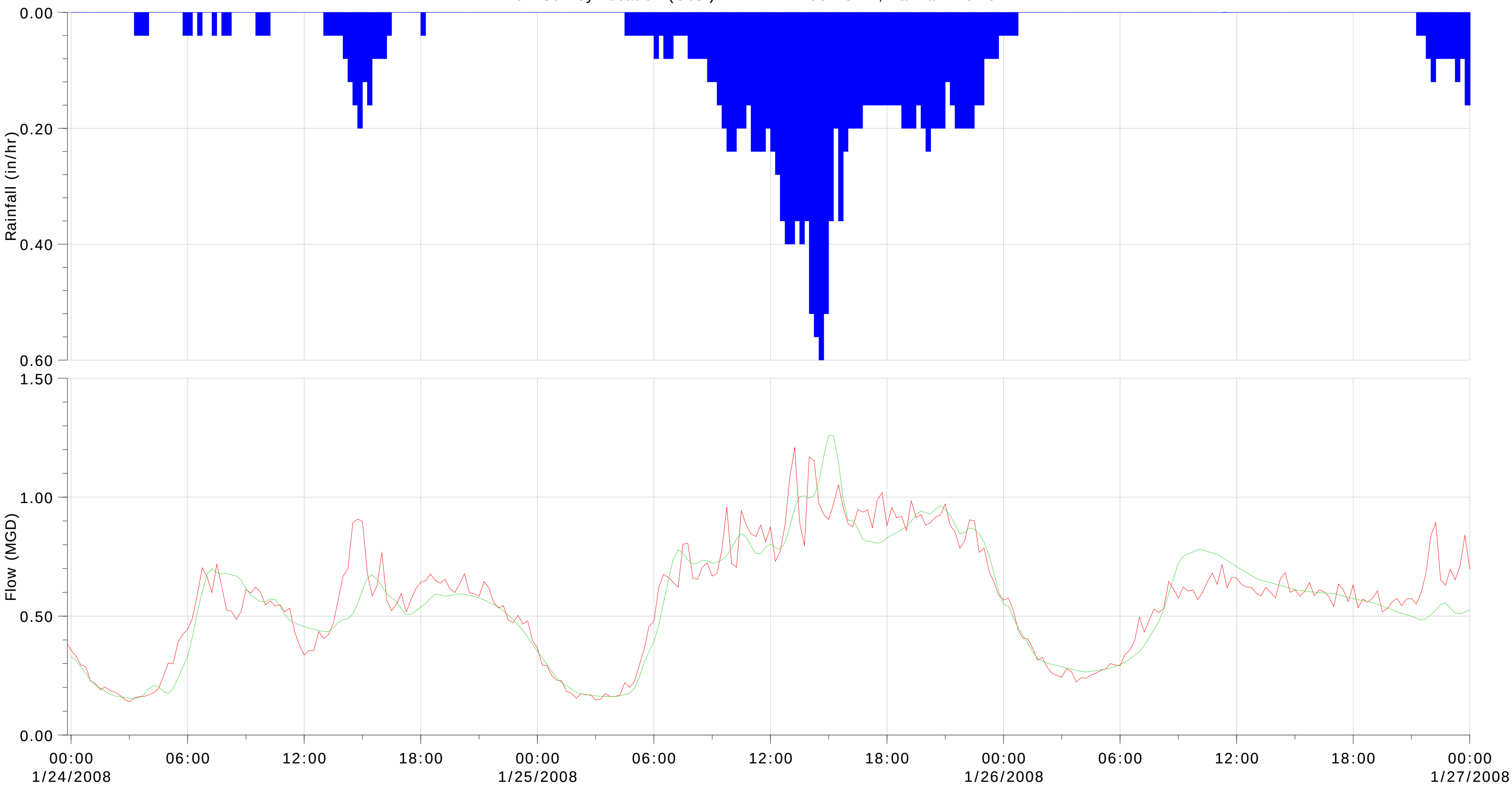
Flow Survey Location (Obs.) FM10 MH-E13-053.1, Rainfall Profile: 3



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	5.023	0.600	0.070			
Obs.				0.280	1.190	2.398
...torm_Existing>2008_Rainfall_inhr				0.222	1.581	2.725

Observed / Predicted Plot Produced by ahope (4/8/2009 5:19:25 PM) Page 11 of 11
Flow Survey: >Daly_City_Catchment>Flow Survey Group>2008 Data (1/12/2009 4:48:55 PM)
Sim: >Daly_City_Catchment>WWF_Calibration>WWF_Jan26th_Storm_Existing>2008_Rainfall_inhr (4/8/2009 2:59:28 PM)
Graph Template: >Daly_City_Catchment>Graph Template Group>2008 Graph Template (4/3/2009 6:01:43 PM)

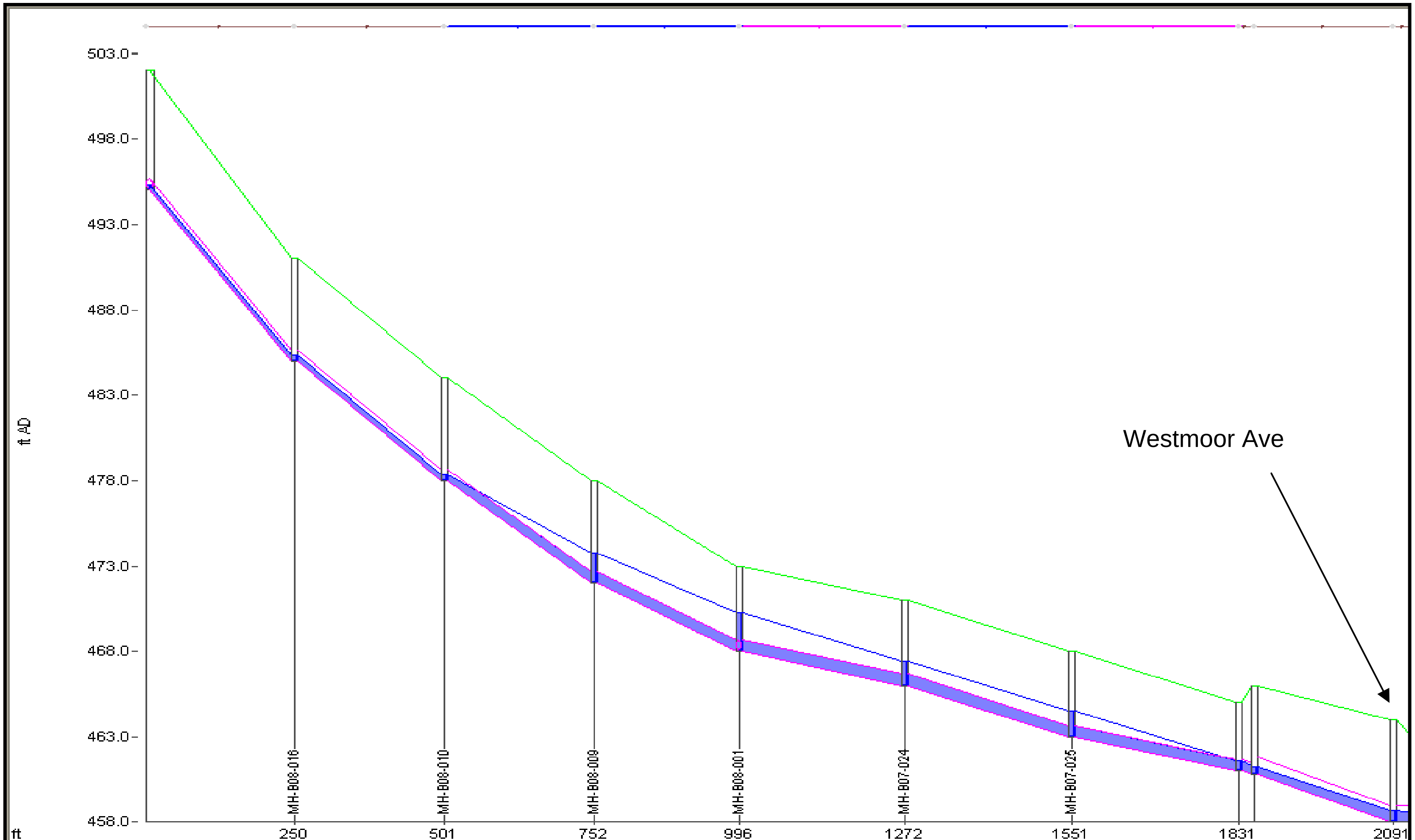
Flow Survey Location (Obs.) FM11 MH-D06-134.1, Rainfall Profile: 2



	Rainfall			Flow (MGD)		
	Depth (in)	Peak (in/hr)	Average (in/hr)	Min	Max	Volume (US Mgal)
Rain	4.483	0.600	0.063			
Obs.				0.140	1.208	1.724
...torm_Existing>2008_Rainfall_inhr				0.154	1.258	1.669

Appendix E
Capacity Deficiency Profiles

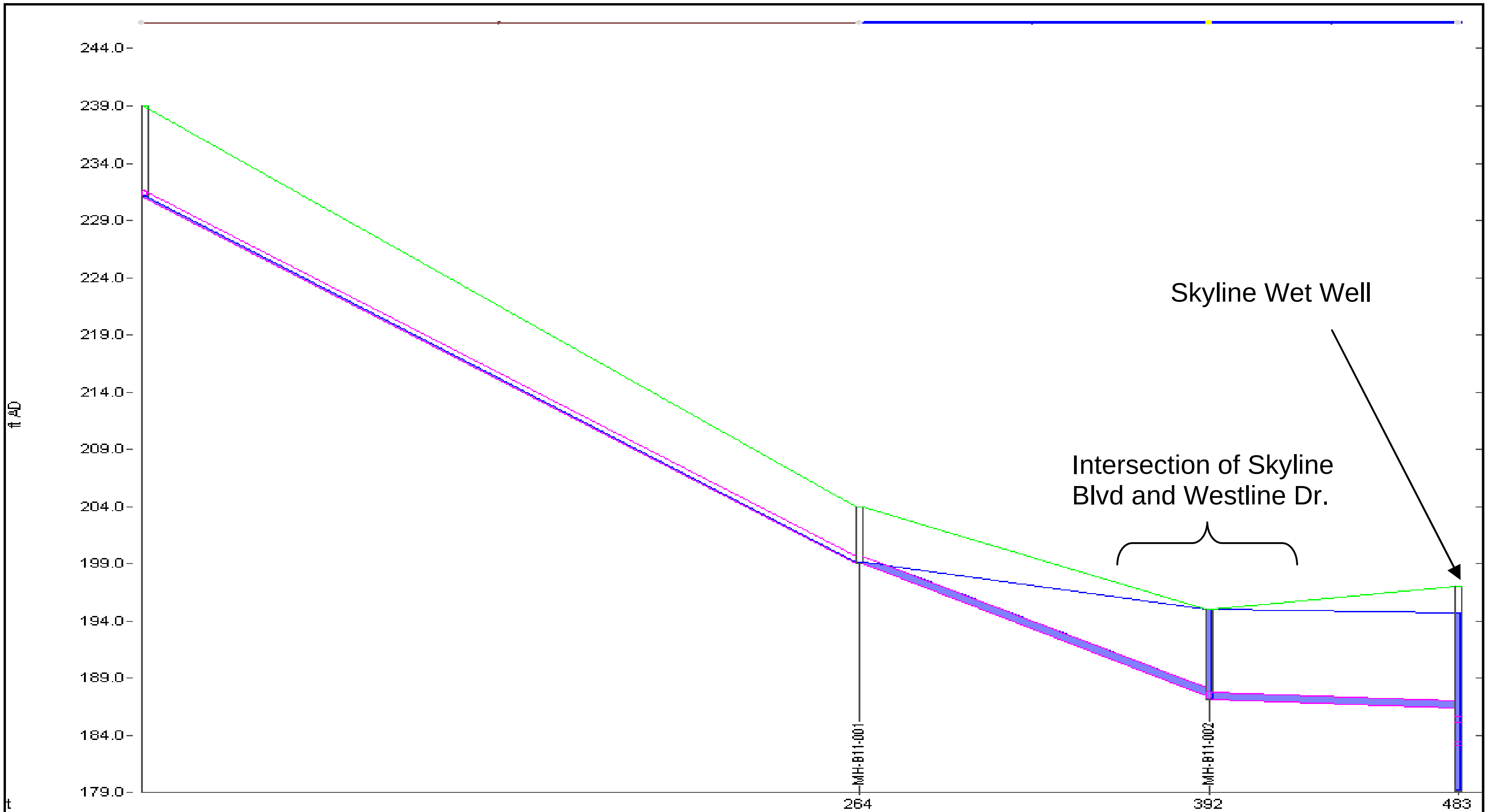
Skyline Boulevard Pipeline Profile – Future System, Varying Intensity Storm Event



Link	MH-B08-017.1	MH-B08-016.1	MH-B08-010.1	MH-B08-009.1	MH-B08-001.1	MH-B07-024.1	MH-B07-025.1	-	MH-B07-049.1	-
width (in)	8.0	8.0	8.0	8.0	8.0	8.0	8.0	-	12.0	-
us inv (ft AD)	495.000	485.000	478.000	472.000	468.000	466.000	463.000	-	460.800	-
ds inv (ft AD)	485.000	478.000	472.000	468.000	466.000	463.000	461.000	-	458.000	-
grad (%)	4.002	2.792	2.383	1.641	0.726	1.073	0.715	-	1.198	-
pfc (MGD)	1.56	1.31	1.21	1.00	0.67	0.81	0.66	-	2.52	-
DS Flow (MGD)	0.67438	0.67438	0.67465	0.90718	0.77832	0.77828	0.77827	-	0.83982	-

Node	-	MH-B08-016	MH-B08-010	MH-B08-009	MH-B08-001	MH-B07-024	MH-B07-025	-	-	-
Asset ID										
ground (ft AD)	502.000	491.000	484.000	478.000	473.000	471.000	468.000	465.000	466.000	464.000

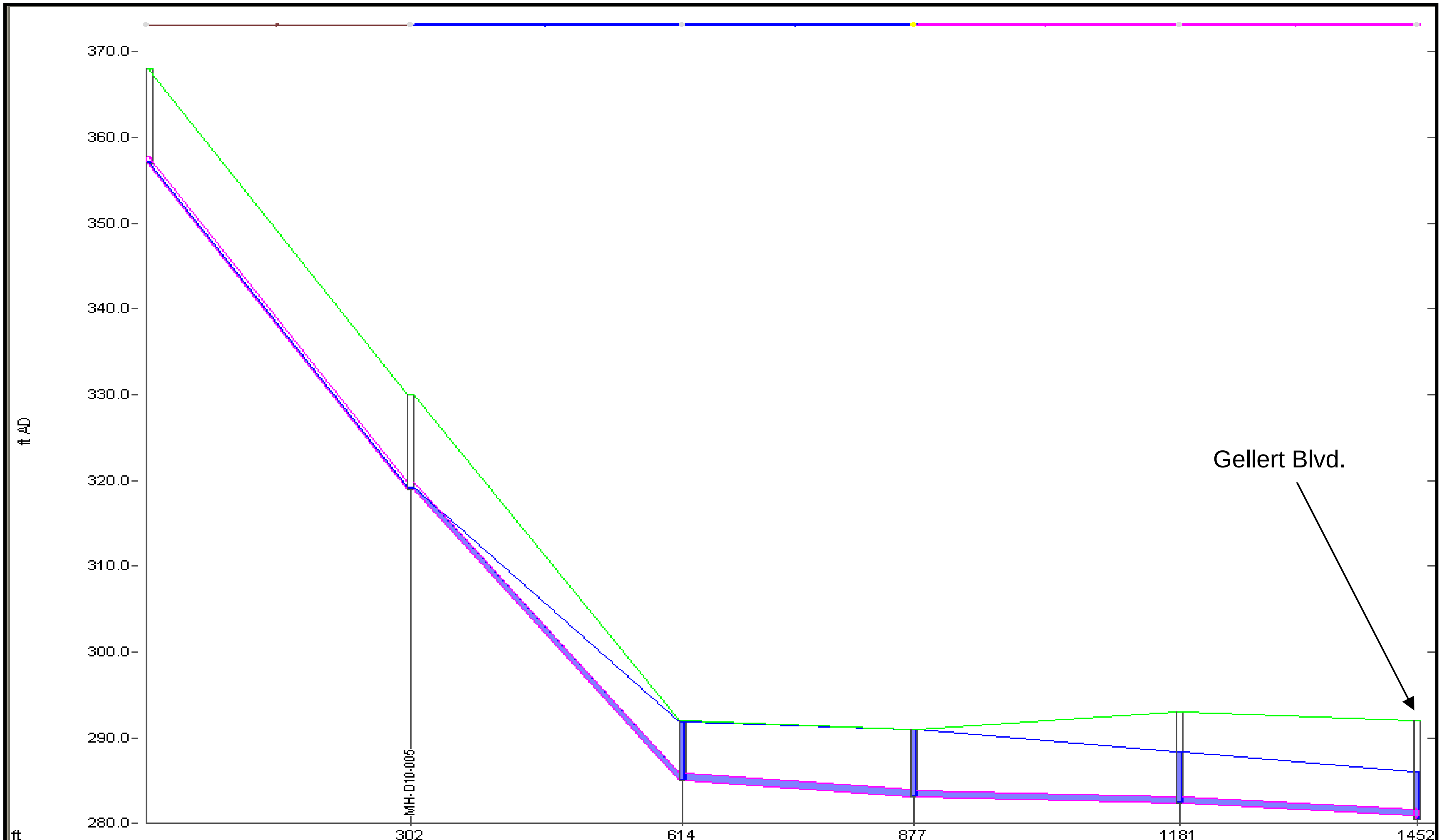
Westline Dr. and Skyline Blvd (Skyline Pump Station) Pipeline Profile – Future System, Varying Intensity Storm Event



Link	MH-B10-030.1	MH-B11-001.1	MH-B11-002.1
width (in)	8.0	8.0	8.0
us inv (ft AD)	231.000	199.000	187.100
ds inv (ft AD)	199.000	187.500	186.400
grad (%)	12.143	8.977	0.765
pfc (MGD)	2.72	2.34	0.68
DS Flow (MGD)	0.35132	0.35132	0.57782

Node	MH-B10-030	MH-B11-001	MH-B11-002	SkylineLift
Asset ID				
ground (ft AD)	239.000	204.000	195.000	197.000

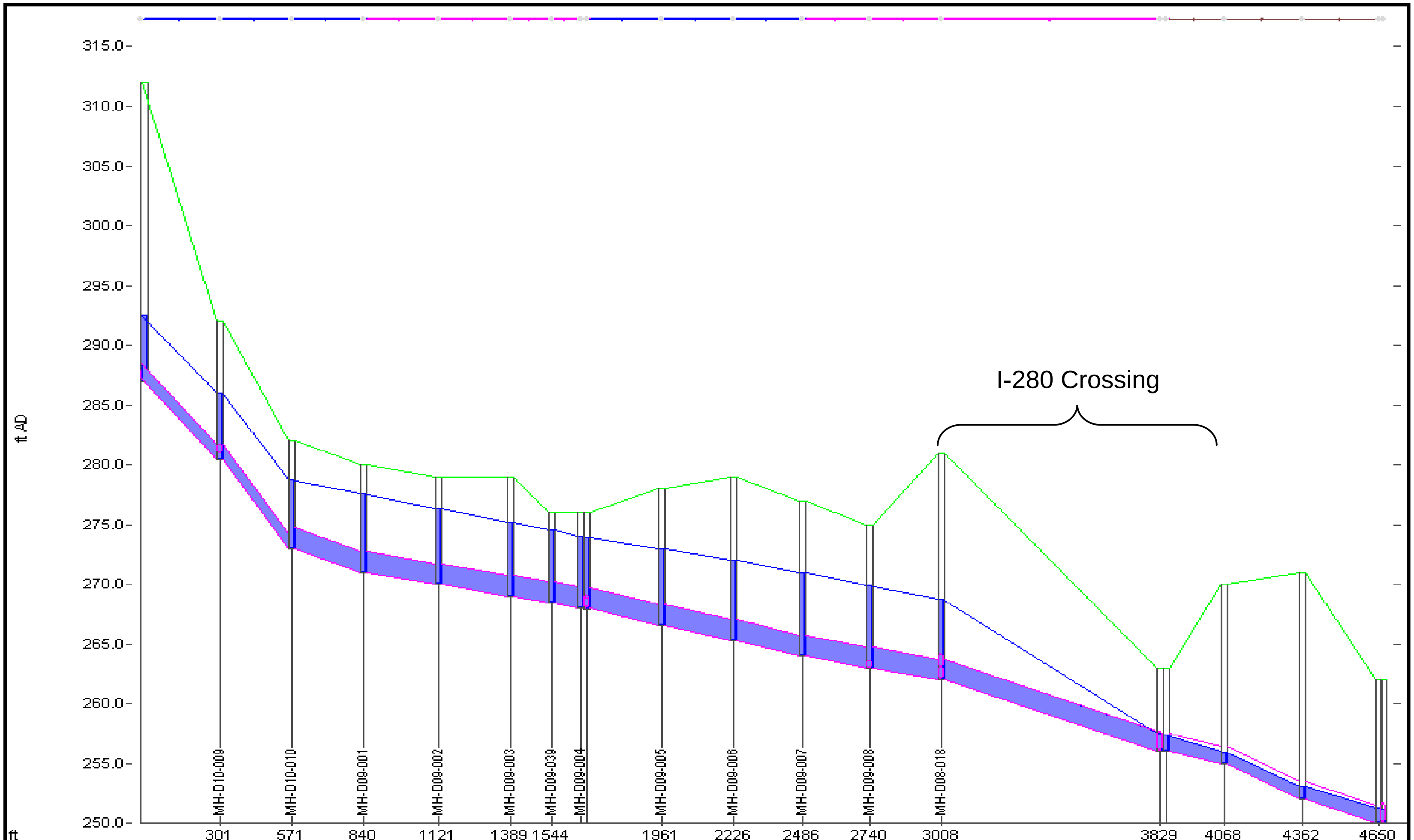
Serramonte Blvd. Pipeline Profile – Future System, Varying Intensity Storm Event



Link	MH-D10-004.1	MH-D10-005.1	MH-D10-006.1	MH-D10-007.1	MH-D10-008.1
width (in)	10.0	10.0	10.0	8.0	8.0
us inv (ft AD)	357.000	319.000	285.000	283.200	282.400
ds inv (ft AD)	319.000	285.000	283.200	282.400	281.000
grad (%)	12.589	10.904	0.684	0.263	0.516
pfc (MGD)	5.03	4.68	1.17	0.40	0.56
DS Flow (MGD)	0.83278	0.82855	0.81357	0.81062	0.80607

Node	MH-D10-004	MH-D10-005	MH-D10-006	MH-D10-007	MH-D10-008	MH-D10-009
Asset ID						
ground (ft AD)	368.000	330.000	292.000	291.000	293.000	292.000

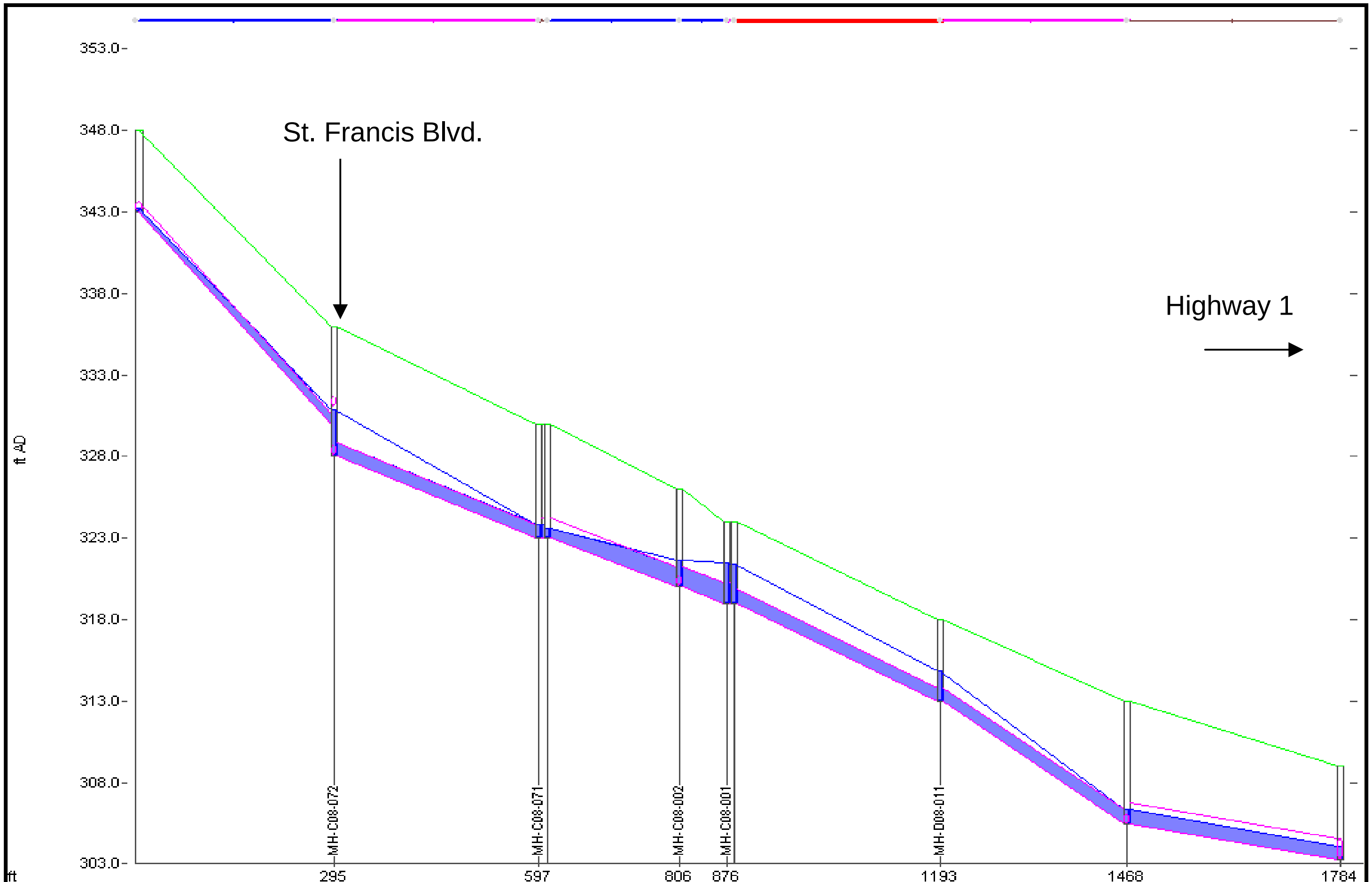
I-280 Crossing (21-Inch Pipe) Pipeline Profile – Future System, Varying Intensity Storm Event



Link	-	-	-	-	-	-	-	-	-	-	-	-	MH-D08-018.1	-	-	-
width (in)	15.0	15.0	21.0	21.0	21.0	21.0	-	21.0	21.0	21.0	21.0	21.0	21.0	18.0	18.0	18.0
us inv (ft AD)	287.200	280.500	273.000	271.000	270.000	-	-	268.000	266.600	265.290	264.000	263.000	262.000	-	255.000	252.000
ds inv (ft AD)	280.500	273.000	271.000	270.000	269.000	-	-	266.600	265.290	264.000	263.000	262.000	256.000	-	252.000	250.000
grad (%)	2.228	2.771	0.745	0.356	0.373	0.346	-	0.496	0.495	0.496	0.393	0.374	0.731	0.456	1.021	0.696
pfc (MGD)	6.23	6.95	8.84	6.11	6.25	6.02	-	7.21	7.21	7.21	6.42	6.27	8.76	4.59	6.86	5.66
DS Flow (MGD)	6.01718	6.69350	6.66091	6.63176	6.60735	-	-	5.87429	5.86013	6.55197	6.53758	6.58796	11.86766	-	4.87187	4.87096

Node	-	-	-	-	-	-	-	-	-	-	-	-	MH-D08-018	MH-D08-019	-	-	-	-
Asset ID																		
ground (ft AD)	-	292.000	282.000	280.000	279.000	-	-	-	278.000	279.000	277.000	275.000	281.000	263.000	-	270.000	271.000	-

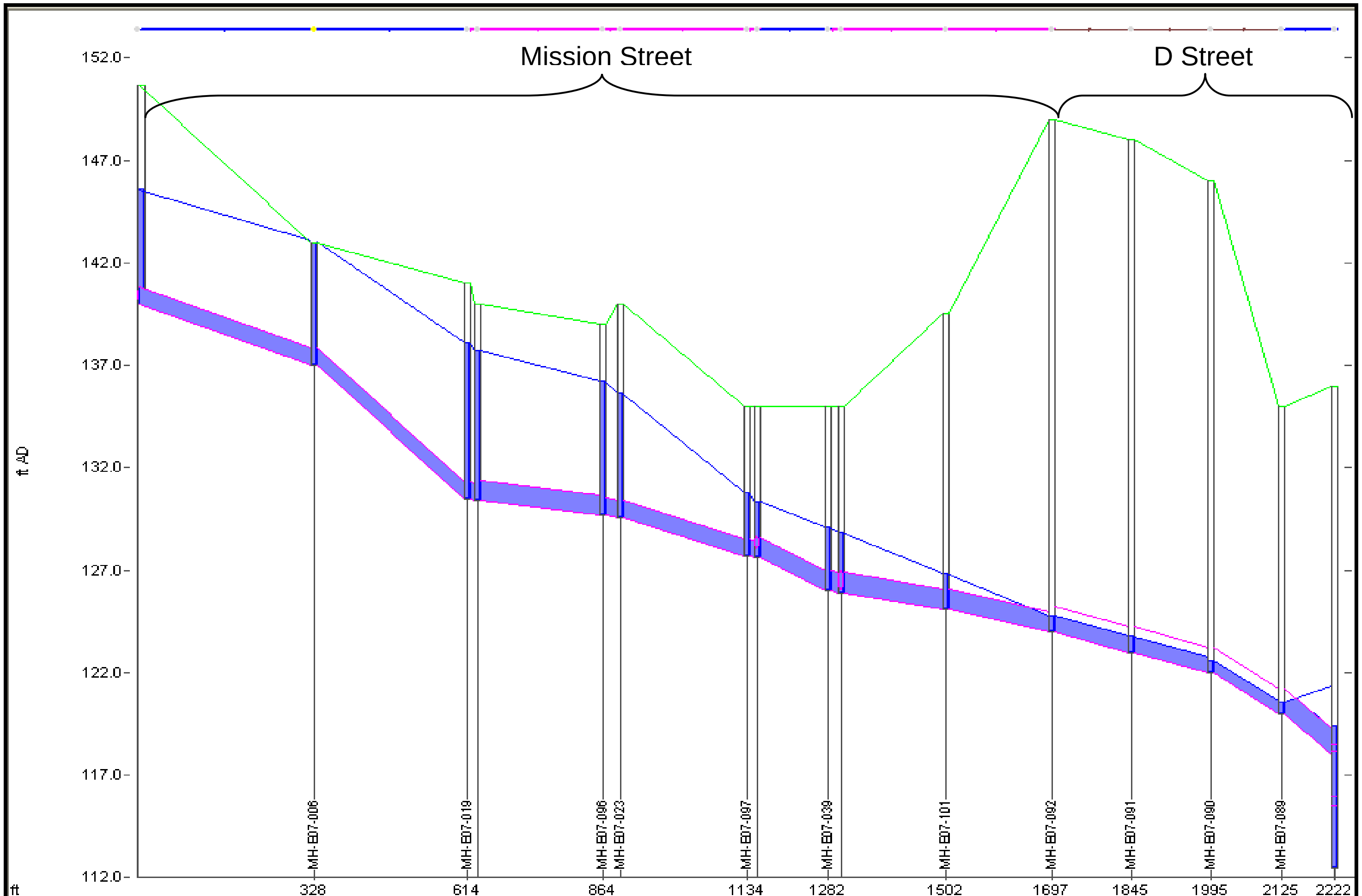
Southgate Ave Pipeline Profile – Future System, Varying Intensity Storm Event



Link	MH-C08-073.1	MH-C08-072.1	MH-C08-003.1	-	MH-C08-069.1	MH-D08-011.1	MH-D08-012.1
width (in)	8.0	10.0	15.0	15.0	10.0	10.0	15.0
us inv (ft AD)	343.000	328.000	323.000	-	319.000	313.000	305.500
ds inv (ft AD)	330.000	323.000	320.000	-	313.000	305.500	303.300
grad (%)	4.402	1.655	1.527	1.419	1.970	2.727	0.696
pfc (MGD)	1.64	1.82	5.16	4.97	1.99	2.34	3.48
DS Flow (MGD)	0.53946	2.11839	2.09566	-	2.03180	2.42592	2.67007

Node	MH-C08-073	MH-C08-072	MH-C08-071	-	-	-	MH-C08-069	MH-D08-011	MH-D08-012	MH-D08-013
Asset ID										
ground (ft AD)	348.000	336.000	330.000	330.000	326.000	-	324.000	318.000	313.000	309.000

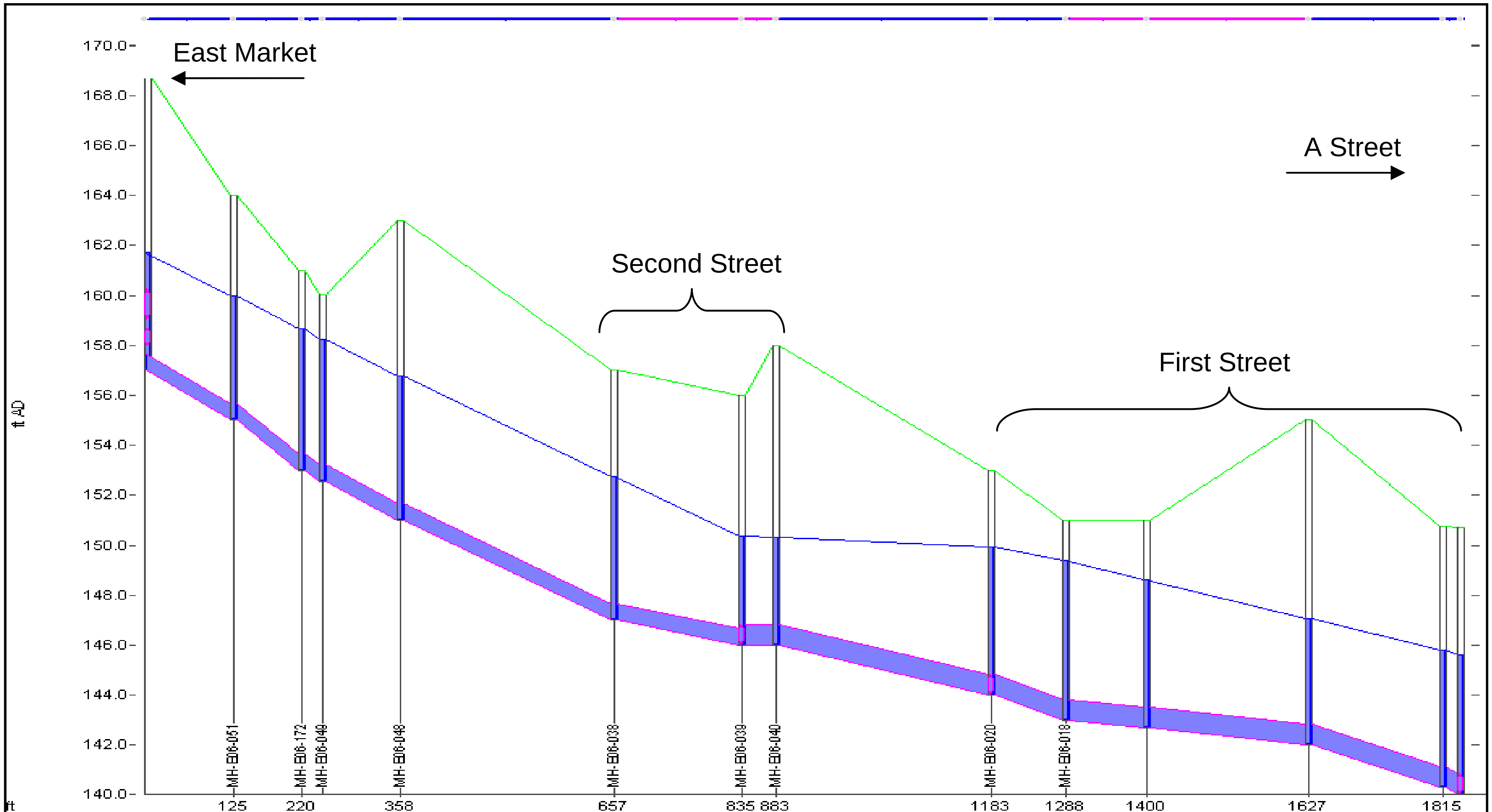
D Street/MissionStreet Pipeline Profile – Future System, Varying Intensity Storm Event



Link	MH-E06-164.1	MH-E07-006.1	MH-E07-095.1	-	MH-E07-023.1	-	MH-E07-022.1	MH-E07-101.1	-	-	-	-
width (in)	10.0	10.0	12.0	-	10.0	12.0	12.0	12.0	15.0	15.0	15.0	15.0
us inv (ft AD)	140.000	137.000	130.400	-	129.600	127.600	125.900	125.100	124.000	123.000	122.000	-
ds inv (ft AD)	137.000	130.500	129.700	-	127.700	126.000	125.100	124.000	123.000	122.000	120.000	-
grad (%)	0.914	2.279	0.304	-	0.802	1.224	0.410	0.564	0.678	0.668	1.537	2.055
pfc (MGD)	1.35	2.14	1.27	-	1.27	2.55	1.47	1.73	3.44	3.41	5.18	5.99
DS Flow (MGD)	1.21788	1.88081	1.87052	-	1.99927	2.16275	2.28902	2.28901	2.37273	2.37271	2.37270	-

Node	-	MH-E07-006	-	-	-	-	-	-	-	-	MH-E07-101	-	-	-	-	-
Asset ID																
ground (ft AD)	150.699	143.000	141.000	140.000	139.000	140.000	135.000	-	-	-	139.559	149.000	148.000	146.000	-	-

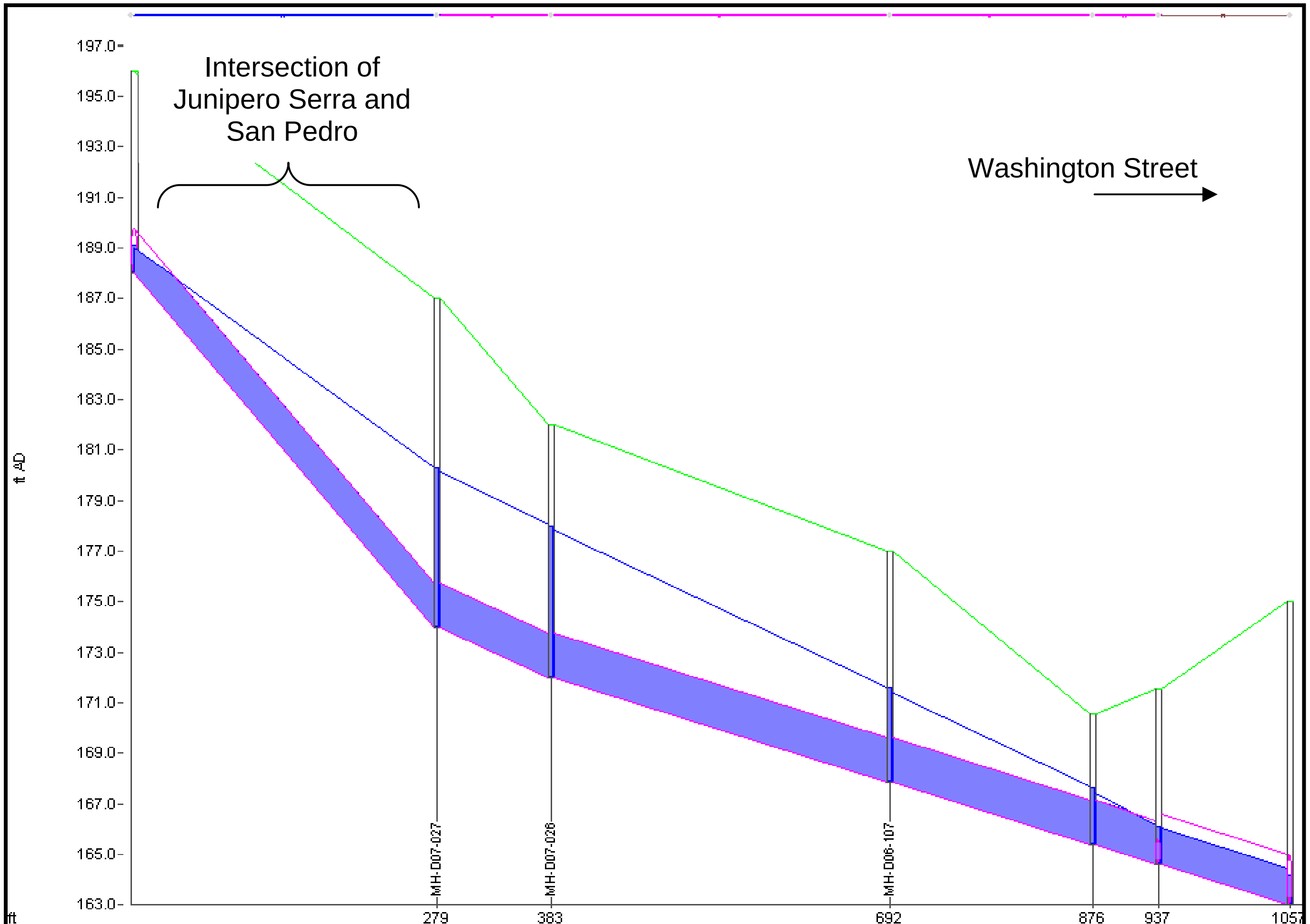
A Street to East Market Street Pipeline Profile – Future System, Varying Intensity Storm Event



Link	-	-	-	-	MH-E06-048.1	MH-E06-038.1	-	MH-E06-040.1	-	-	MH-E06-166.1	MH-E06-017.1	-
width (in)	8.0	8.0	-	8.0	8.0	8.0	10.0	10.0	10.0	10.0	10.0	10.0	-
us inv (ft AD)	157.000	155.000	-	152.563	151.000	147.000	-	146.000	144.000	143.000	142.700	142.000	-
ds inv (ft AD)	155.000	153.000	-	151.000	147.000	146.000	-	144.000	143.000	142.700	142.000	140.300	-
grad (%)	1.600	2.100	-	1.451	1.339	0.560	-	0.668	0.951	0.267	0.308	0.906	-
pfc (MGD)	0.99	1.13	-	0.94	0.90	0.58	0.00	1.16	1.38	0.73	0.79	1.35	-
DS Flow (MGD)	0.92139	0.89435	-	0.88640	0.88790	0.89020	-	0.89870	0.99563	1.14380	1.14396	1.14438	-

Node	-	-	-	-	MH-E06-048	MH-E06-038	-	MH-E06-040	MH-E06-020	-	MH-E06-166	MH-E06-017	-
Asset ID													
ground (ft AD)	-	164.000	-	-	163.000	157.000	156.000	158.000	153.000	151.000	151.000	155.000	150.742

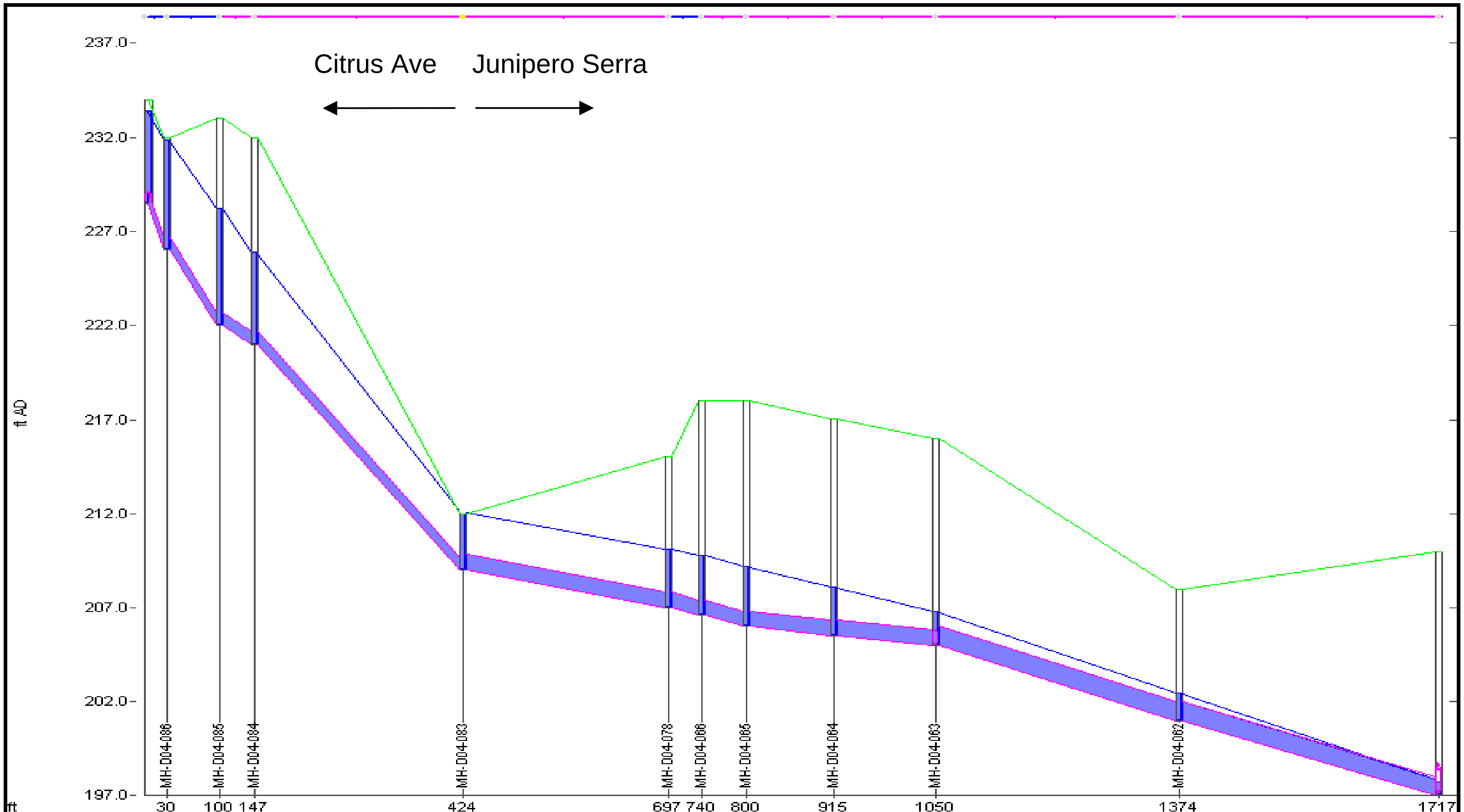
Junipero Serra/San Pedro Pipeline Profile – Future System, Varying Intensity Storm Event



Link	MH-D07-028.1	MH-D07-027.1	MH-D07-026.1	MH-D06-107.1	-	MH-D06-105.1
width (in)	21.0	21.0	21.0	21.0	21.0	24.0
us inv (ft AD)	188.000	174.000	172.000	167.865	165.412	164.600
ds inv (ft AD)	174.000	172.000	167.865	165.412	164.600	163.000
grad (%)	5.013	1.932	1.335	1.335	1.336	1.336
pfc (MGD)	22.93	14.24	11.84	11.84	11.84	16.90
DS Flow (MGD)	14.32402	14.30549	14.29657	14.29481	14.29451	14.33385

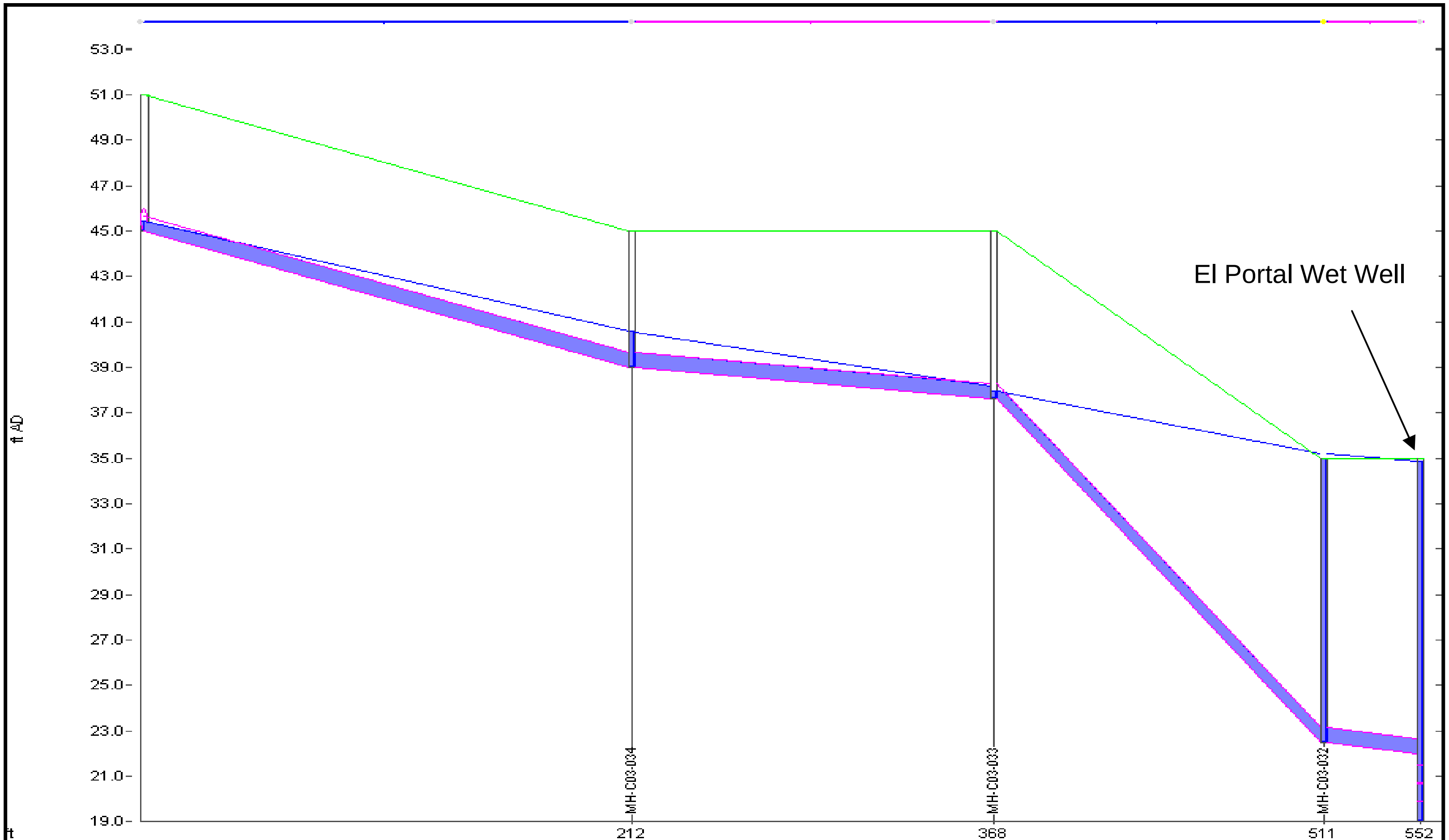
Node	MH-D07-028	MH-D07-027	MH-D07-026	MH-D06-107	MH-D06-106	MH-D06-105	-
Asset ID							
ground (ft AD)	196.000	187.000	182.000	177.000	170.574	171.581	175.000

Junipero Serra and Citrus Ave. Pipeline Profile – Future System, Varying Intensity Storm Event



Link	-	-	-	MH-D04-084.1	MH-D04-083.1	-	-	-	-	MH-D04-063.1	MH-D04-062.1
width (in)	-	8.0	8.0	8.0	10.0	10.0	10.0	10.0	10.0	12.0	12.0
us inv (ft AD)	-	-	-	221.000	209.000	-	-	206.000	205.540	205.000	201.000
ds inv (ft AD)	-	-	-	209.000	207.000	-	-	205.540	205.000	201.000	197.000
grad (%)	-	5.694	-	4.333	0.733	-	0.967	0.401	0.400	1.235	1.163
pfc (MGD)	-	1.86	1.14	1.63	1.21	1.39	1.39	0.90	0.90	2.56	2.48
DS Flow (MGD)	-	-	-	1.70229	1.35224	-	-	1.46759	1.46691	2.58371	2.58334

Node	-	-	MH-D04-084	MH-D04-083	MH-D04-078	-	-	-	MH-D04-063	MH-D04-062	MH-D04-022
Asset ID											
ground (ft AD)	-	-	232.000	212.000	215.000	-	-	217.000	216.000	208.000	210.000



Link	MH-C03-035.1	MH-C03-034.1	MH-C03-033.1	-
width (in)	8.0	8.0	8.0	8.0
us inv (ft AD)	45.000	39.000	37.600	22.500
ds inv (ft AD)	39.000	37.600	22.500	22.000
grad (%)	2.828	0.897	10.596	1.202
pfc (MGD)	1.31	0.74	2.54	0.86
DS Flow (MGD)	0.95130	0.95125	0.95124	0.91924

Node	MH-C03-035	MH-C03-034	MH-C03-033	MH-C03-032	-
Asset ID					
ground (ft AD)	51.000	45.000	45.000	35.000	-

Appendix F
Project Descriptions and Cost Estimates

PROJECT DESCRIPTION

PROJECT ID:……………………. C-1
PROJECT NAME: Skyline Blvd.
PRIORITY:………………………… 4
PROJECT DESCRIPTION:……… Upsize 1,080 ft of 8" pipe to 10" along Skyline Dr. from Carmel Ave. to south of Westmoor Ave
DEFICIENCY DESCRIPTION:….. Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:…………… MH-B08-009 to MH-B07-026
PEAK DESIGN FLOW:…………. 1.2 MGD
ASSUMPTIONS: Pipe bursting
ALTERNATIVES:………………… Open-cut replacement or parallel pipe

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
8"	10"	1078		215	$231,800
				Construction Cost Sub-Total:	**$231,800**
			Contingencies	30%	$69,540
				Estimated Construction Cost:	**$301,340**
	Technical Services and Administration			25%	$75,335
				Total Project Cost:	**$377,000**

PROJECT DESCRIPTION	
PROJECT ID:……………………	C-2
PROJECT NAME:	Belcrest LS/Forcemain
PRIORITY:…………………………	1
PROJECT DESCRIPTION:………	New parallel 6" forcemain from Belcrest Lift Station to Oceanside Dr. (3,082 ft)
DEFICIENCY DESCRIPTION:…..	Belcrest pumps are operating below their design point because of undersized discharge force main
PROJECT EXTENTS:…………..	Belcrest Lift Station and MH-B09-014
PEAK DESIGN FLOW:………….	0.7 to 0.8 MGD (both forcemains)
ASSUMPTIONS:	Increasing the size of the force main will allow the existing pumps to operate more efficiently and prevent signficant upstream backup, based on model results.
ALTERNATIVES:…………………	An alternative to upsizing the force main would be to replace the pumps with ones with more capacity. The city should consider this alternative if the existing pumps are in poor condition. New pumps could potentially keep up with peak inflows during the design storm, however, the flow rate necessary to balance peak inflows would result in very high velocities and high head losses in the existing 6" force main. Given the length of the Belcrest force main, this may result in unreasonably large pumps.

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
NA	6"	3082		170	$523,900
		Construction Cost Sub-Total:			**$523,900**
		Contingencies		30%	$157,170
		Estimated Construction Cost:			**$681,070**
		Technical Services and Administration		25%	$170,268
		Total Project Cost:			**$851,000**

PROJECT DESCRIPTION

PROJECT ID:……………………	C-3
PROJECT NAME:	Skyline LS/Forcemain
PRIORITY:………………………	1
PROJECT DESCRIPTION:…….	New parallel 6" forcemain from Skyline Lift Station to Belcrest Lift Station (1,925 ft)
DEFICIENCY DESCRIPTION:…	Skyline pumps are operating below their design point because of undersized discharge force main
PROJECT EXTENTS:…………	Skyline Lift Station to Belcrest Lift Station wet well
PEAK DESIGN FLOW:…………	0.7 to 0.8 MGD (both forcemains)
ASSUMPTIONS:	Increasing the size of the force main will allow the existing pumps to operate more efficiently and prevent signficant upstream backup, based on model results.
ALTERNATIVES:……………….	An alternative to upsizing the force main would be to replace the pumps with ones with more capacity. The city should consider this alternative if the existing pumps are in poor condition. New pumps could potentially keep up with inflows during the design storm, however, the flow rate necessary to balance peak inflows would result in very high velocities and high head losses in the existing 6" force main.

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
NA	6"	1925		170	$327,300
			Construction Cost Sub-Total:		**$327,300**
			Contingencies	30%	$98,190
			Estimated Construction Cost:		**$425,490**
			Technical Services and Administration	25%	$106,373
			Total Project Cost:		**$532,000**

PROJECT DESCRIPTION

PROJECT ID:…………………… C-4
PROJECT NAME: Serramonte Blvd.
PRIORITY:………………………… 4
PROJECT DESCRIPTION:……… Upsize 575 ft of 8" pipe to 10" along Serramonte Blvd. from Gellert Blvd. to Serramonte Center Driveway
DEFICIENCY DESCRIPTION:….. Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:…………… MH-D10-007 to MH-D10-009
PEAK DESIGN FLOW:…………. 1.0 MGD
ASSUMPTIONS: Pipe bursting
ALTERNATIVES:………………… Open cut replacement or parallel pipe

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
8"	10"	575		215	$123,600
			Construction Cost Sub-Total:		**$123,600**
			Contingencies	30%	$37,080
			Estimated Construction Cost:		**$160,680**
			Technical Services and Administration	25%	$40,170
			Total Project Cost:		**$201,000**

PROJECT DESCRIPTION	
PROJECT ID:………………………	C-5
PROJECT NAME:	I-280 Crossing
PRIORITY:…………………………	3
PROJECT DESCRIPTION:………	New 30" I-280 crossing, from Southgate Ave to Junipero Serra (750 ft of 48" casing and 275 feet open cut pipe)
DEFICIENCY DESCRIPTION:…..	Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:……………	MH-D08-018 to MH-D08-020
PEAK DESIGN FLOW:………….	12.8 MGD (both pipes)
ASSUMPTIONS:	The proposed I-280 crossing is the most cost effective alterative identified, but would require close coordination with Caltrans; acquiring all the necessary permits could be challenging.
ALTERNATIVES:…………………	There is potential for a new pump station near Mervyns' that would discharge to a new force main along Southgate under the freeway, to Junipero Serra and then north to the sewer. This alternative would like be more expensive than the proposed I-280 crossing.

Existing Diameter (inches)	**New Diameter (inches)**	**Length (feet)**		**Unit Cost ($/lf)**	**Total Cost ($)**
NA	48" Crossing	**750**		**2200**	$1,650,000
NA	30" open cut	275		550	$151,300
				Construction Cost Sub-Total:	**$1,801,300**
			Contingencies	30%	$540,390
				Estimated Construction Cost:	**$2,341,690**
			Technical Services and Administration	25%	$585,423
				Total Project Cost:	**$2,927,000**

PROJECT DESCRIPTION

PROJECT ID:……………………… C-6
PROJECT NAME: Southgate Ave.
PRIORITY:………………………… 5
PROJECT DESCRIPTION:……… Upsize 578 ft of 10" pipe to 15" along Southgate Ave from Escuela Dr. to Cerro Dr
DEFICIENCY DESCRIPTION:…… Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:…………… MH-C08-069 to MH-D08-012
PEAK DESIGN FLOW:…………… 2.2 to 2.6 MGD
ASSUMPTIONS: Pipe bursting
ALTERNATIVES:………………… Open-cut replacement or parallel pipe

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
10"	15"	578		240	$138,700
			Construction Cost Sub-Total:		**$138,700**
			Contingencies	30%	$41,610
			Estimated Construction Cost:		**$180,310**
			Technical Services and Administration	25%	$45,078
			Total Project Cost:		**$225,000**

PROJECT DESCRIPTION

PROJECT ID:…………………….. C-7
PROJECT NAME: D Street/Mission Street
PRIORITY:………………………… 2
PROJECT DESCRIPTION:……… Upsize 1,083 ft of 10" and 12" pipe to 15" and 285 ft of 10" pipe to 12" along Mission Street and D St
DEFICIENCY DESCRIPTION:….. Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:…………… MH-E07-006 to MH-E07-092
PEAK DESIGN FLOW:…………. 2.5 to 2.9 MGD
ASSUMPTIONS: Pipe bursting
ALTERNATIVES:………………… Open-cut replacment or parallel pipe

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
10-12"	15"	1083		240	$259,900
10"	12"	285		220	$62,700
			Construction Cost Sub-Total:		**$322,600**
			Contingencies	30%	$96,780
			Estimated Construction Cost:		**$419,380**
			Technical Services and Administration	25%	$104,845
			Total Project Cost:		**$524,000**

PROJECT DESCRIPTION	
PROJECT ID:……………………	C-8
PROJECT NAME:	A Street to E. Market
PRIORITY:…………………………	5
PROJECT DESCRIPTION:………	Upsize 339 ft of 10" pipe to 12" in 1st Ave. between A St. and Valley St. and 178 ft of 8" pipe to 10" in 2nd Ave. between Valley St. and East Market
DEFICIENCY DESCRIPTION:……	Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:……………	MH-E06-018 to MH-E06-017 and MH-E06-038 to MH-E06-039
PEAK DESIGN FLOW:………….	1.0 to 1.3 MGD
ASSUMPTIONS:	Pipe bursting
ALTERNATIVES:…………………	Open-cut replacement or parallel pipe

Existing Diameter (inches)	**New Diameter (inches)**	**Length (feet)**		**Unit Cost ($/lf)**	**Total Cost ($)**
10"	12"	339		220	$74,600
8"	10"	178		215	$38,300
			Construction Cost Sub-Total:		**$112,900**
			Contingencies	30%	$33,870
			Estimated Construction Cost:		**$146,770**
			Technical Services and Administration	25%	$36,693
			Total Project Cost:		**$183,000**

PROJECT DESCRIPTION

PROJECT ID:………………………	C-9
PROJECT NAME:	Junipero Serra/San Pedro
PRIORITY:…………………………	4
PROJECT DESCRIPTION:………	New parallel 24" pipe along Junipero Serra Blvd. between San Pedro and Washington (516 ft)
DEFICIENCY DESCRIPTION:……	Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:……………	New manhole between MH-D07-026 and MH-D06-107 to MH-D06-105
PEAK DESIGN FLOW:……………	10.4 MGD (15.6 MGD both pipes)
ASSUMPTIONS:	Divert flow from 21" trunk and parallel or replace existing 12" pipe in Dunks St.
ALTERNATIVES:…………………	There is potential for alternative alignments along Junipero Serra Blvd or to utilize some capacity in the 12" pipe between MH-D06-167 and MH-D06-105

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
NA	24"	516		385	$198,700
			Construction Cost Sub-Total:		**$198,700**
			Contingencies	30%	$59,610
			Estimated Construction Cost:		**$258,310**
			Technical Services and Administration	25%	$64,578
			Total Project Cost:		**$323,000**

PROJECT DESCRIPTION	
PROJECT ID:………………………	C-10
PROJECT NAME:	Junipero Serra/Citrus
PRIORITY:…………………………	2
PROJECT DESCRIPTION:………	Upsize 116 ft of 8" pipe to 10" and 593 ft of 8" and 10" pipe to 12" and 977 ft of 10" and 12" pipe to 15" along Junipero Serra Blvd and Citrus Avenue
DEFICIENCY DESCRIPTION:…..	Pipes may surcharge under significant rainfall events
PROJECT EXTENTS:……………	MH-D04-086 to MH-D04-022
PEAK DESIGN FLOW:………….	2.2 to 3.1 MGD
ASSUMPTIONS:	Pipe bursting
ALTERNATIVES:…………………	Open-cut replacement or parallel pipe

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
8"	10"	116		215	$24,900
8-10"	12"	593		220	$130,500
10-12"	15"	977	5 to 10	240	$234,500
			Construction Cost Sub-Total:		**$389,900**
			Contingencies	30%	$116,970
			Estimated Construction Cost:		**$506,870**
		Technical Services and Administration		25%	$126,718
			Total Project Cost:		**$634,000**

PROJECT DESCRIPTION

PROJECT ID:……………………	C-11
PROJECT NAME:	El Portal LS/Forcemain
PRIORITY:…………………………	2
PROJECT DESCRIPTION:………	New 8" parallel force main at El Portal along Cliffside St (1,219 ft)
DEFICIENCY DESCRIPTION:……	El Portal pumps are operating somewhat below their design point because of undersized discharge force main
PROJECT EXTENTS:……………	El Portal LS to MH-C03-079
PEAK DESIGN FLOW:………….	0.88 MGD (1.29 MGD both pipes)
ASSUMPTIONS:	Increasing the size of the force main will allow the existing pumps to operate more efficiently and prevent significant upstream backup, based on model results. During dry weather flow, velocities in the two force mains would be low - it is assumed that the lift station could be operated in a way that maximizes velocities during low flow periods.
ALTERNATIVES:…………………	The pumps could be replaced with higher capacity pumps; however, based on model results, sizing new pumps to keep up with peak inflows during the design storm would results in unreasonable velocities and head losses in the existing 6" force main. An additional alternative would be to increase the size of the wet well to attenuate peak flows, however, the cost implications of this alternative is beyond the scope of this analysis

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
NA	8"	1219		180	$219,400
			Construction Cost Sub-Total:		**$219,400**
			Contingencies	30%	$65,820
			Estimated Construction Cost:		**$285,220**
			Technical Services and Administration	25%	$71,305
			Total Project Cost:		**$357,000**