DEXTER WILSON ENGINEERING, INC.

WATER • WASTEWATER • RECYCLED WATER

CONSULTING ENGINEERS

SEWER STUDY
FOR THE
LUISENO VILLAGE RETAIL CENTER

February 6, 2019

2234 FARADAY AVENUE • CARLSBAD, CA • (760) 438-4422

SEWER STUDY
FOR THE
LUISENO VILLAGE RETAIL CENTER

February 6, 2019

Prepared For:
Environmental Data Systems, Inc.
717 K Street, Suite 424
Sacramento, CA 92814

Prepared by:
Dexter Wilson Engineering, Inc.
2234 Faraday Avenue
Carlsbad, CA 92008
760-438-4422

REGISTERED PROFESSIONAL ENGINEER
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CIVIL
STATE OF CALIFORNIA

Job No. 1049-001

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DEXTER WILSON ENGINEERING, INC.

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ANDREW M. OVEN, P.E.
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NATALIE J. FRASCHETTI, P.E.
STEVEN J. HENDERSON, P.E.

February 6, 2019 1049-001

Environmental Data Systems, Inc.
717 K. Street, Suite 424
Sacramento, CA 92814

Attention: Joe Broadhead

Subject: Sewer Study for the Luiseno Village Retail Center

A. OBJECTIVE

This letter report provides a sewer capacity study for the Luiseno Village Retail Center. The project is located at the southwest corner of Main Street and Ramona Expressway in the City of San Jacinto. The project is proposed to be provided sewer service by the City of San Jacinto.

The project consists of approximately 9.46 acres and is in the process of rezoning from commercial neighborhood to commercial general. Attachment 1 provides a vicinity map for the project. The objective of this study is to present the City of San Jacinto with projected sewer flows for the project and proposed sewer system facilities so they can evaluate service to the project.

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B. ANALYSIS CRITERIA

In the absence of sewer planning criteria from the City of San Jacinto, the analysis in this letter-report was prepared based on criteria from the Eastern Municipal Water District (EMWD) 2015 Wastewater Collection System Master Plan prepared by Black and Veatch. The pertinent criteria from that document are included in Attachment 4 and summarized as follows:

- Commercial Average Day Flow Factor – 1,175 gpd/ac
- Peak Flow Factor – Per Figure 2 in Attachment 4
- Mannings "n" Coefficient – 0.013
- Maximum Depth-to-Diameter, 12" and smaller pipe – 0.50
- Maximum Depth-to-Diameter, 15" and larger pipe – 0.70
- Minimum Velocity – 2 feet/sec
- Maximum Velocity 10 feet/sec

C. SEWER SYSTEM

Sewer service to the Luiseno Village Retail Center will be provided by the City of San Jacinto. The City operates and maintains local collector sewers in the area that convey flow to EMWD truck sewers and interceptors. EMWD is responsible for treatment and disposal of sewage conveyed to their system from the City of San Jacinto.

In the vicinity of the project, the City has an 8-inch sewer line that extends south of Donna Way to serve the Soboba Indian Health Clinic. This sewer line conveys flow west in Donna Way and then through an easement before turning north in an easement that runs along the western boundary of the proposed Luiseno Village Retail Center. This 8-inch sewer line conveys flow to an 8-inch sewer line in Main Street that conveys flow westerly. This line leaves Main Street and increases to 12-inch and then to 15-inch prior to connecting to an EMWD 36-inch line in Palm Avenue. Currently, the only flow in the upstream reaches of the 8-inch sewer line are from the Soboba Indian Health Clinic. Attachment 2 provides a figure that shows the location of existing sewer facilities in the vicinity of the project.

Table 1 summarizes the projected sewer flows for the Luiseno Village Retail Center. Based on an average flow of 11,116 gpd and a peak factor of 2.87, the projected peak flow for the project is 31,903 gpd.

TABLE 1
LUISENO VILLAGE RETAIL CENTER
PROJECTED SEWER FLOWS

Land Use	Quantity	Unit Flows	Total Average Flow, gpd
Parcel 1 – Commercial	2.48 ac	1,175 gpd/ac	2,914
Parcel 2 – Commercial	2.82 ac	1,175 gpd/ac	3,314
Parcel 8 – Commercial	2.04 ac	1,175 gpd/ac	2,397
Parcel 9 – Commercial	2.12 ac	1,175 gpd/ac	2,491
TOTAL			**11,116**

The onsite sewer system for the Luiseno Village Retail Center is proposed to be private with a single point of connection to the existing 8-inch line that runs along the western project boundary. Attachment 3 provides the proposed sewer system for the project. The sizing of the private onsite sewer system will be based on the Plumbing Code and drainage fixture units served and is outside the scope of this study. The existing public sewer line has a lot of available capacity in the upstream reaches as it currently only serves the Soboba Indian Health Clinic. The downstream reaches of sewer line in Main Street pick up flows from other sites. The impact of the project on these sections of line was evaluated based on flow metering data that was collected as discussed below.

Flow metering data was collected at two locations in the existing sewer downstream of the project for the period from January 9, 2019 through January 25, 2019. The two locations where flow metering was performed are indicated on Figure 2 in Attachment 2 and include the 8-inch line in Main Street at Santa Fe Avenue and the 12-inch line in De Anza Drive. The flow metering data results are included in Attachment 4.

The flow metering data on the 8-inch line in Main Street indicates that the sewer experienced a maximum flow of 0.25 mgd and a maximum d/D ratio of 0.36 during the monitoring period. Based on the anticipated flows from Luiseno Village, the maximum d/D ratio would increase to 0.38 with the projected flows from the project. The calculations to support this finding are provided in Attachment 4.

The flowmeter data for the 12-inch line in De Anza Drive indicates that the sewer experienced a maximum flow of 0.51 mgd and a maximum d/D radio of 0.42 during the monitoring period. With additional flow from Luiseno Village, the maximum projected d/D ratio would increase to 0.43. The calculations to support this finding are provided in Attachment 4.

D. CONCLUSION

The Luiseno Village Retail Center proposes a private onsite sewer system with a single point of connection to the existing 8-inch San Jacinto sewer line that runs along the western project boundary. There is capacity in the upstream reaches of the existing 8-inch sewer line since it currently only serves the Soboba Indian Health Clinic. Based on sewer flow metering that was performed in the existing system downstream of the project, the existing sewer system is flowing within its design capacity at peak flows. With the addition of flows from the Luiseno Village project, the existing sewer lines are still anticipated to be operating within their design capacity at peak flows.

Dexter Wilson Engineering, Inc.

Stephen M. Niel

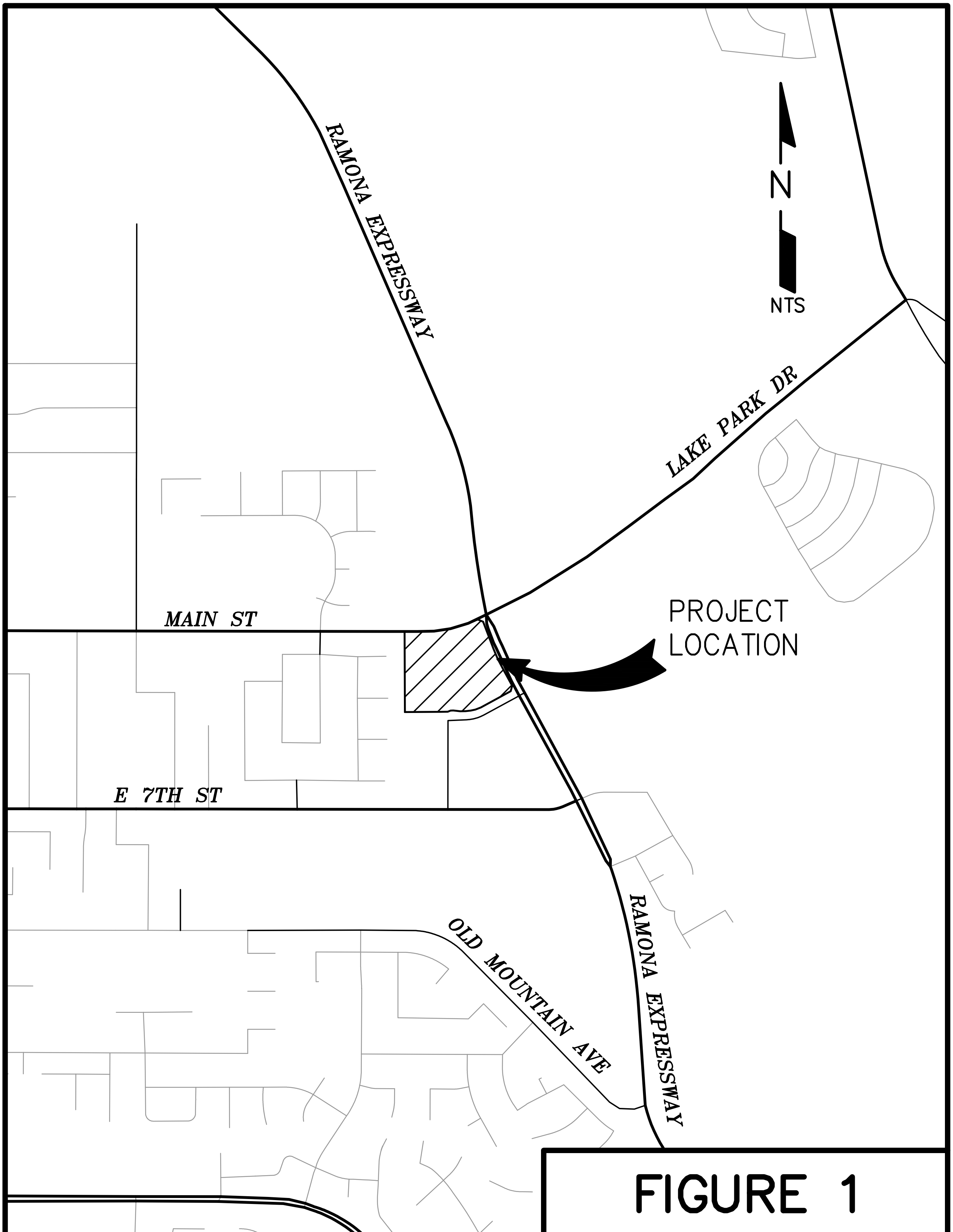
Stephen M. Nielsen, P.E.

SMN:pjs

Attachment(s)

ATTACHMENT 1

VICINITY MAP



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(760) 438-4422

FIGURE 1

VICINITY MAP

LUISENO VILLAGE

ATTACHMENT 2

EXISTING SEWER SYSTEM

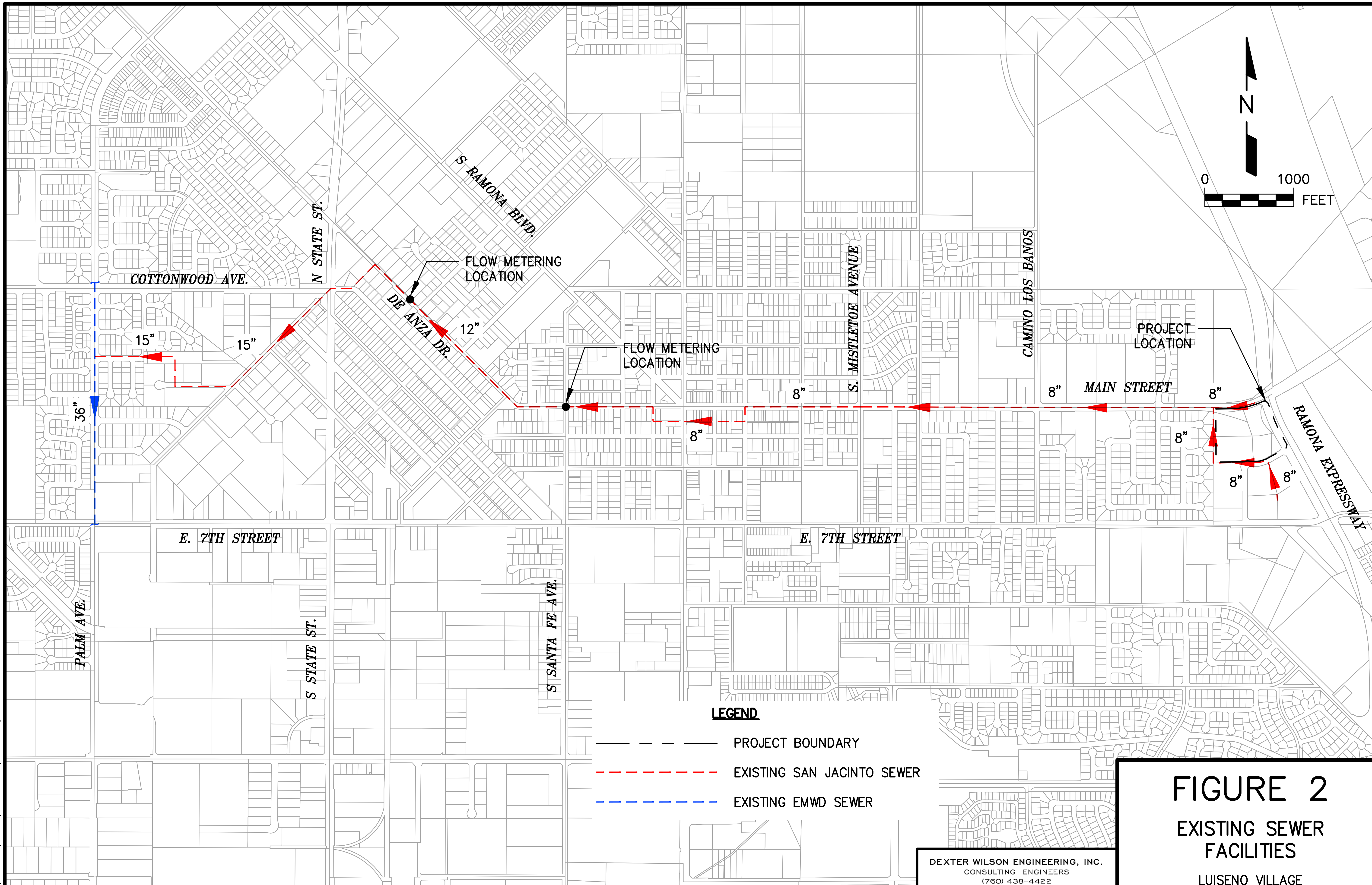

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FIGURE 2

EXISTING SEWER FACILITIES

LUISENO VILLAGE

\\ARTIC\DWG\1049001\SEWER\LV_SWR_FIGURE2_EXSWR.DWG 02-05-19 10:39:03 LAYOUT: LAYOUT

ATTACHMENT 3

PROPOSED SEWER SYSTEM

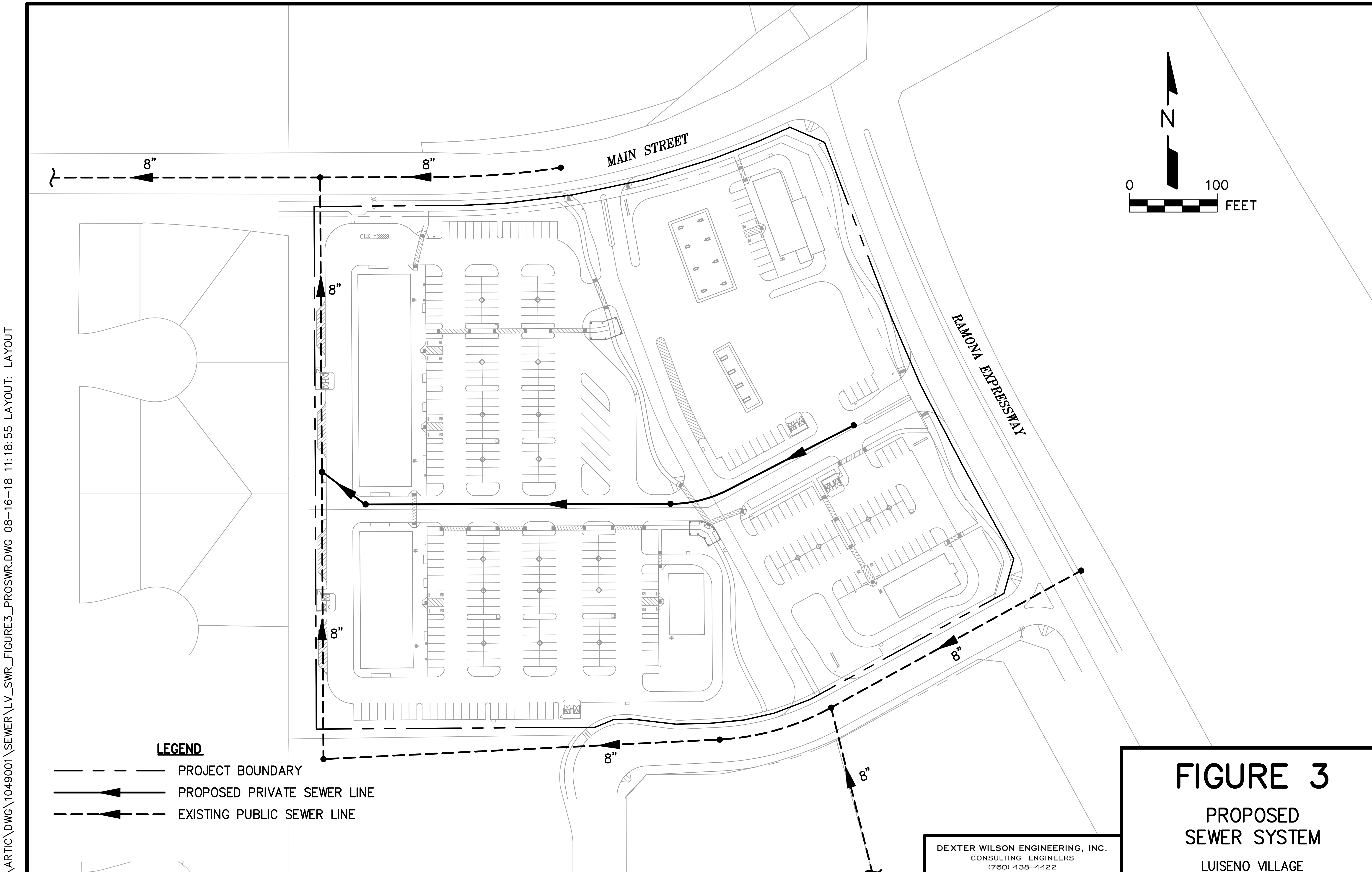

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FIGURE 3

PROPOSED
SEWER SYSTEM

LUISENO VILLAGE

ATTACHMENT 4

SEWER ANALYSIS AND FLOW METER DATA

Luiseno Village Offsite Sewer Evaluation

Main Street 8" Sewer

Per flow monitoring data, Q_{peak} = 0.25 mgd at a d/D of 0.36

Per attached chart, @ d/D of 0.36, Q/Q_{full} = 0.27

So 0.25 mgd/Q_{full} = 0.27

Q_{full} = 0.926 mgd

Existing $Q_{avg} = Q_{peak}/PF$, where PF = peaking factor = 2.87 for flows less than 0.1 mgd per Attachment 5

Q_{avg} = 0.25/2.87

Q_{avg} = 0.087 mgd

With Luiseno Village Flows, Q_{avg} = 0.087 + 0.011 = 0.098 mgd

Q_{peak} = 0.098 x 2.87 = 0.28 mgd

With Luiseno Village Flows, Q_{peak}/Q_{full} = 0.28/0.926 = 0.30

Per attached chart, @ Q/Q_{full} of 0.30, d/D = 0.38

With addition of Luiseno Village flows, maximum d/D will increase from 0.36 to 0.38

De Anza Drive 12" Sewer

Per flow monitoring data, Q_{peak} = 0.51 mgd at a d/D of 0.42

Per attached chart, @ d/D of 0.42, Q/Q_{full} = 0.37

So 0.51 mgd/Q_{full} = 0.37

Q_{full} = 1.38 mgd

Existing $Q_{avg} = Q_{peak}/PF$, where PF = peaking factor = 2.64 per Attachment 5 equation

Q_{avg} = 0.51/2.64

Q_{avg} = 0.193 mgd

With Luiseno Village Flows, Q_{avg} = 0.193 + 0.011 = 0.204 mgd

Q_{peak} = 0.204 x 2.62 = 0.53 mgd

With Luiseno Village Flows, Q_{peak}/Q_{full} = 0.53/1.38 = 0.38

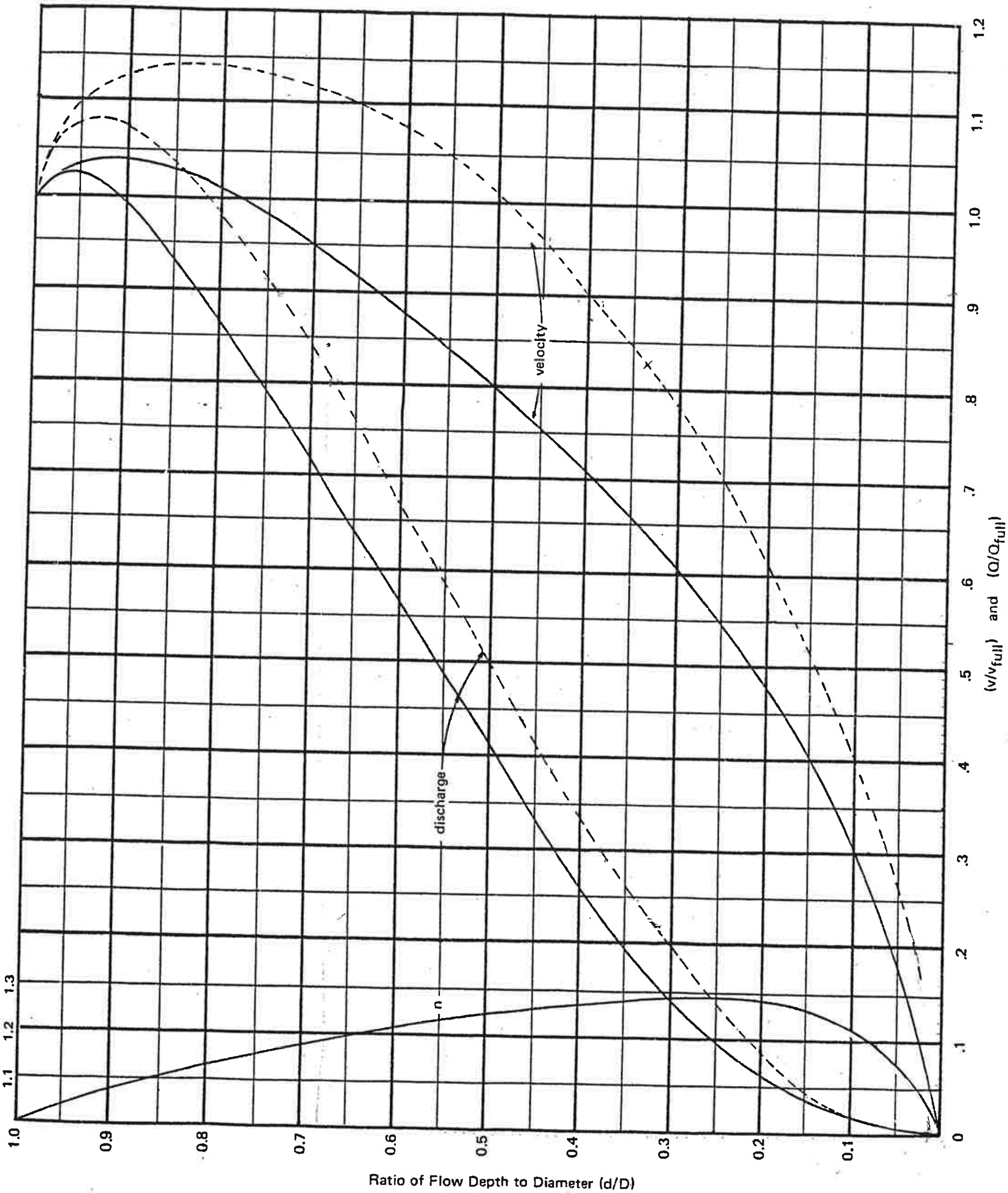
Per attached chart, @ Q/Q_{full} of 0.38, d/D = 0.43

With addition of Luiseno Village flows, maximum d/D will increase from 0.42 to 0.43

Figure 5.20

Circular Channel Ratios

Experiments have shown that n varies slightly with depth. This figure gives velocity and flow rate ratios for varying n (solid line) and constant n (broken line) assumptions.



Site Report 01-29-2019

Confidential Proprietary Information

Environmental Data Systems

~489 De Anza Dr, San Jacinto, CA 92583
Manhole No. Unknown

2019.01 De Anza Dr MH

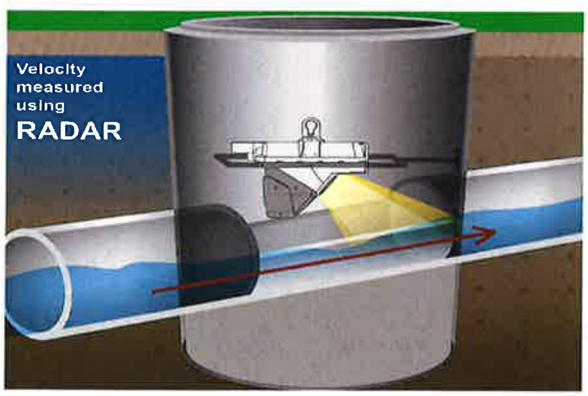
Access:
MH on CL of road, NW of Wateka St

System Type:
Sanitary [X] Storm [ ]

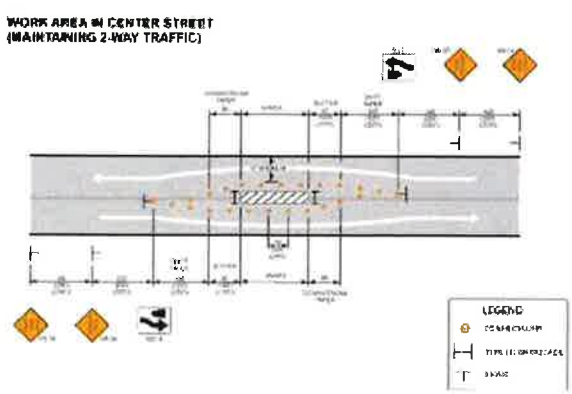
Install Date: 1/09/2019

Map

Technology



Traffic Plan



Flow Meter

Meter Depth: 90"

MH Coordinates: 33.787543, -116.969496

Moderate open channel hydraulics

Avg Velocity	Avg Measured Level	Multiplier
2.0 fps	4.0"	1

Gas

O2	H2S	CO	LEL
20.9	0	0	0

Notes

No laterals; monitored the downstream line as it provided the best hydraulics.

Traffic Safety

No formal TCP required; used cones & signs per site-specific WATCH requirements.

Land Use

Residential	Commercial	Industrial	Trunk
X			

Manhole Depth	109"
Monitored Pipe Size	12"
Inner Pipe Size (In/Out)	12"/12"
Pipe Shape	Round
Pipe Condition	Good
Manhole Material	Brick
Silt	0"
Velocity Profile Data	*
Velocity Profile Taken	0.4 2-D
Sensor Offset	19.08"
Sensor Dist. to Crown	7.08"
Sensor Direction	Downstream
Flow Heading	West

Meter Site Document

Environmental Data Systems

2019.01 De Anza Dr MH ~489 De Anza Dr, San Jacinto, CA 92583

Site

Manhole Before Install

Installation Process

Installed

Upstream

Downstream

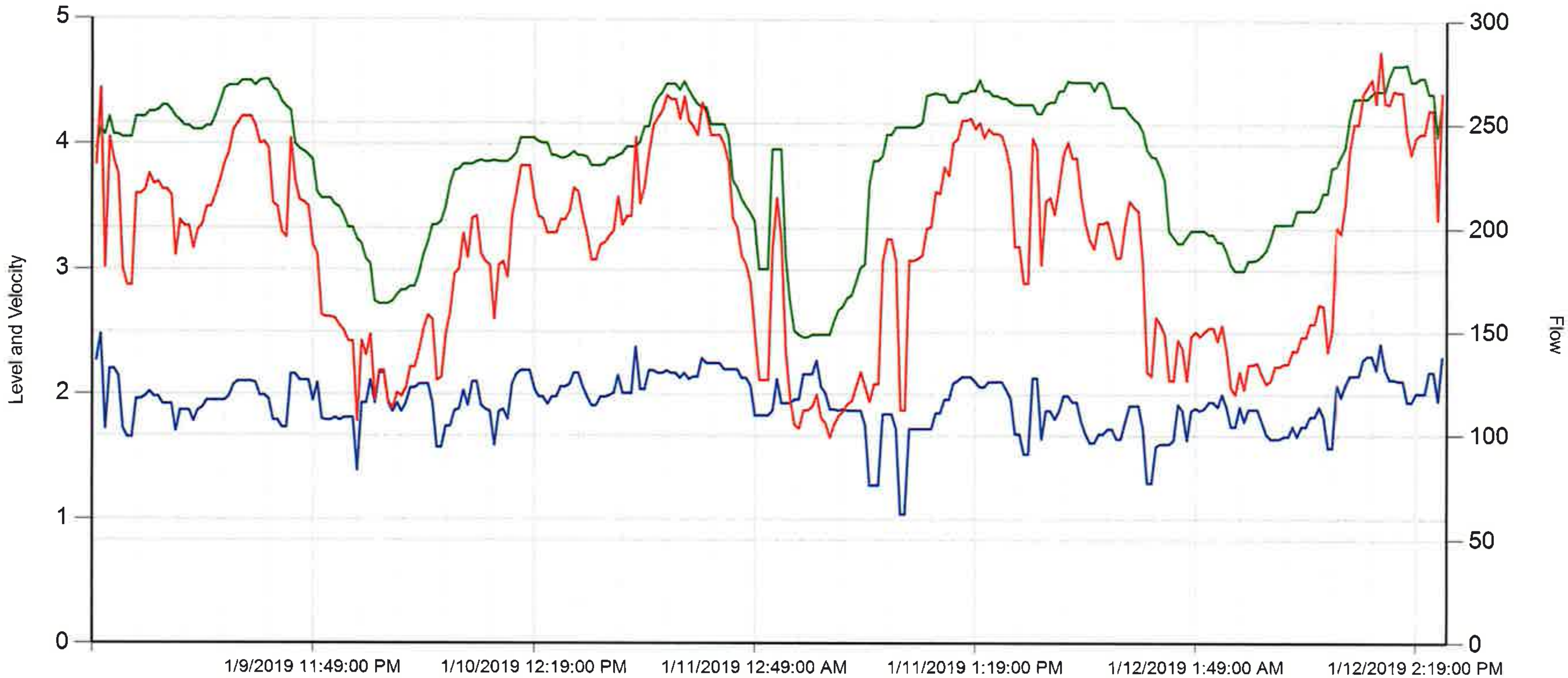
Utility Systems Science and Software

Report Date: 01/29/2019
Customer: Environmental Data Systems
Group: Luiseno Retail Proj
SiteID: 3312

Statistics for 2019.01 De Anza Dr MH : 01/09/2019 thru 01/25/2019

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Max d/D
1/9/19	218.16	266.11	172.57	0.31	0.38	0.25	1.97	2.48	1.65	4.23	4.51	3.87	314,147	0.38
1/10/19	191.25	263.05	106.94	0.28	0.38	0.15	2.01	2.38	1.39	3.75	4.50	2.73	275,402	0.38
1/11/19	181.40	252.50	98.89	0.26	0.36	0.14	1.84	2.27	1.04	3.88	4.52	2.46	261,222	0.38
1/12/19	193.76	284.93	120.07	0.28	0.41	0.17	1.98	2.41	1.57	3.81	4.65	3.00	279,021	0.39
1/13/19	204.97	291.80	89.10	0.30	0.42	0.13	2.01	2.34	1.34	3.90	4.73	2.82	295,150	0.39
Week:	**197.91**	**291.80**	**89.10**	**0.29**	**0.42**	**0.13**	**1.96**	**2.48**	**1.04**	**3.92**	**4.73**	**2.46**	**1,424,943**	**0.39**
1/14/19	194.09	251.46	115.90	0.28	0.36	0.17	1.94	2.20	1.49	3.88	4.43	3.08	279,482	0.37
1/15/19	205.88	356.60	128.82	0.30	0.51	0.19	2.00	2.59	1.62	3.98	4.97	3.11	296,469	0.41
1/16/19	201.16	259.79	116.25	0.29	0.37	0.17	1.89	2.16	1.28	4.09	4.68	2.89	289,666	0.39
1/17/19	197.90	246.53	133.47	0.29	0.36	0.19	2.07	2.35	1.41	3.79	4.51	3.15	284,972	0.38
1/18/19	172.72	235.42	84.44	0.25	0.34	0.12	1.99	2.53	1.40	3.51	4.09	2.54	248,713	0.34
1/19/19	195.73	262.43	111.11	0.28	0.38	0.16	2.11	2.42	1.83	3.69	4.50	2.68	281,855	0.38
1/20/19	195.94	289.58	96.25	0.28	0.42	0.14	1.98	2.46	1.42	3.82	4.64	2.80	282,158	0.39
Week:	**194.77**	**356.60**	**84.44**	**0.28**	**0.51**	**0.12**	**2.00**	**2.59**	**1.28**	**3.82**	**4.97**	**2.54**	**1,963,316**	**0.41**
1/21/19	216.13	291.94	121.94	0.31	0.42	0.18	2.15	2.53	1.67	3.90	4.58	2.91	311,228	0.38
1/22/19	200.09	279.58	120.49	0.29	0.40	0.17	2.03	2.48	1.37	3.86	4.84	2.97	288,130	0.40
1/23/19	232.09	314.51	130.62	0.33	0.45	0.19	2.32	2.60	1.83	3.89	4.64	2.89	334,204	0.39
1/24/19	240.24	306.74	117.92	0.35	0.44	0.17	2.15	2.49	1.42	4.20	4.80	2.91	345,951	0.40
1/25/19	219.98	330.49	129.65	0.32	0.48	0.19	2.15	2.49	1.53	3.98	5.05	3.14	316,771	0.42
Week:	**221.71**	**330.49**	**117.92**	**0.32**	**0.48**	**0.17**	**2.16**	**2.60**	**1.37**	**3.97**	**5.05**	**2.89**	**1,596,285**	**0.42**
Totals:	**203.62**	**356.60**	**84.44**	**0.29**	**0.51**	**0.12**	**2.04**	**2.60**	**1.04**	**3.89**	**5.05**	**2.46**	**4,984,543**	**0.42**

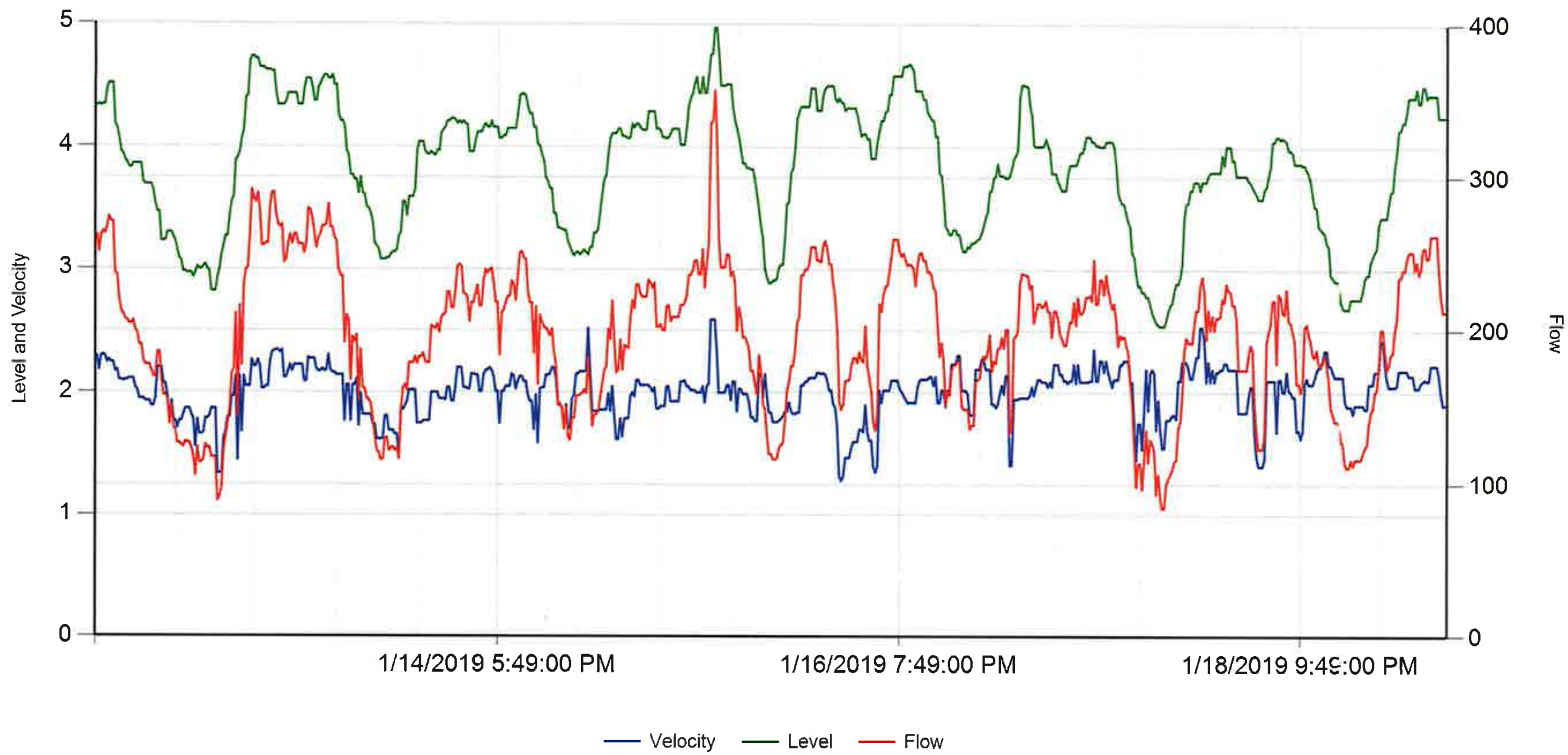
2019.01 De Anza Dr MH



	Velocity (fps)	Level (in)	Flow (gpm)	RainFall	Inches
Average	1.930	3.865	190.770		
Maximum	2.480	4.650	284.930		
Minimum	1.040	2.460	98.889		

1/29/2019 9:50:29 AM

2019.01 De Anza Dr MH



	Velocity (fps)	Level (in)	Flow (gpm)	RainFall	Inches
Average	2.001	3.837	196.166		
Maximum	2.590	4.970	356.596		
Minimum	1.280	2.540	84.444		

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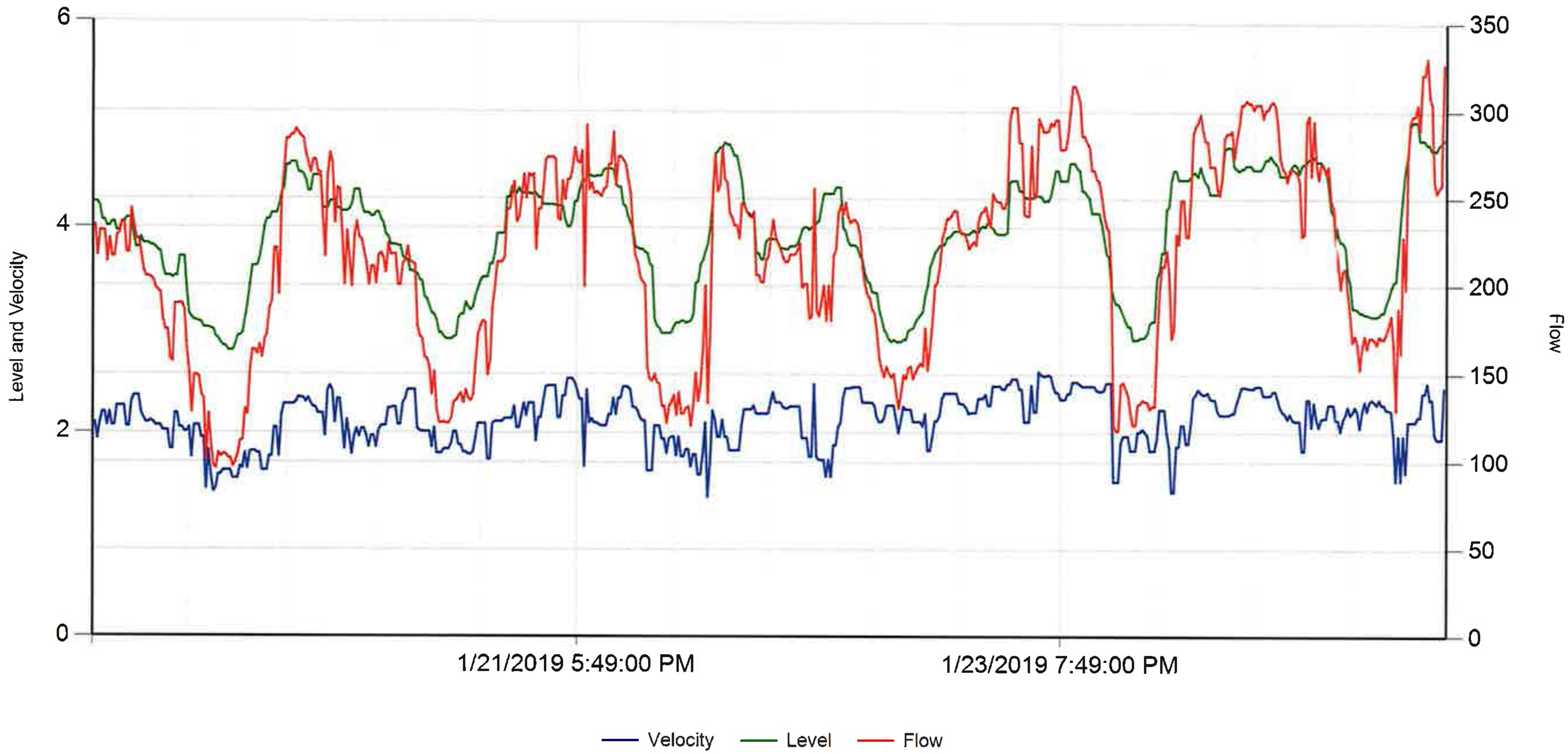
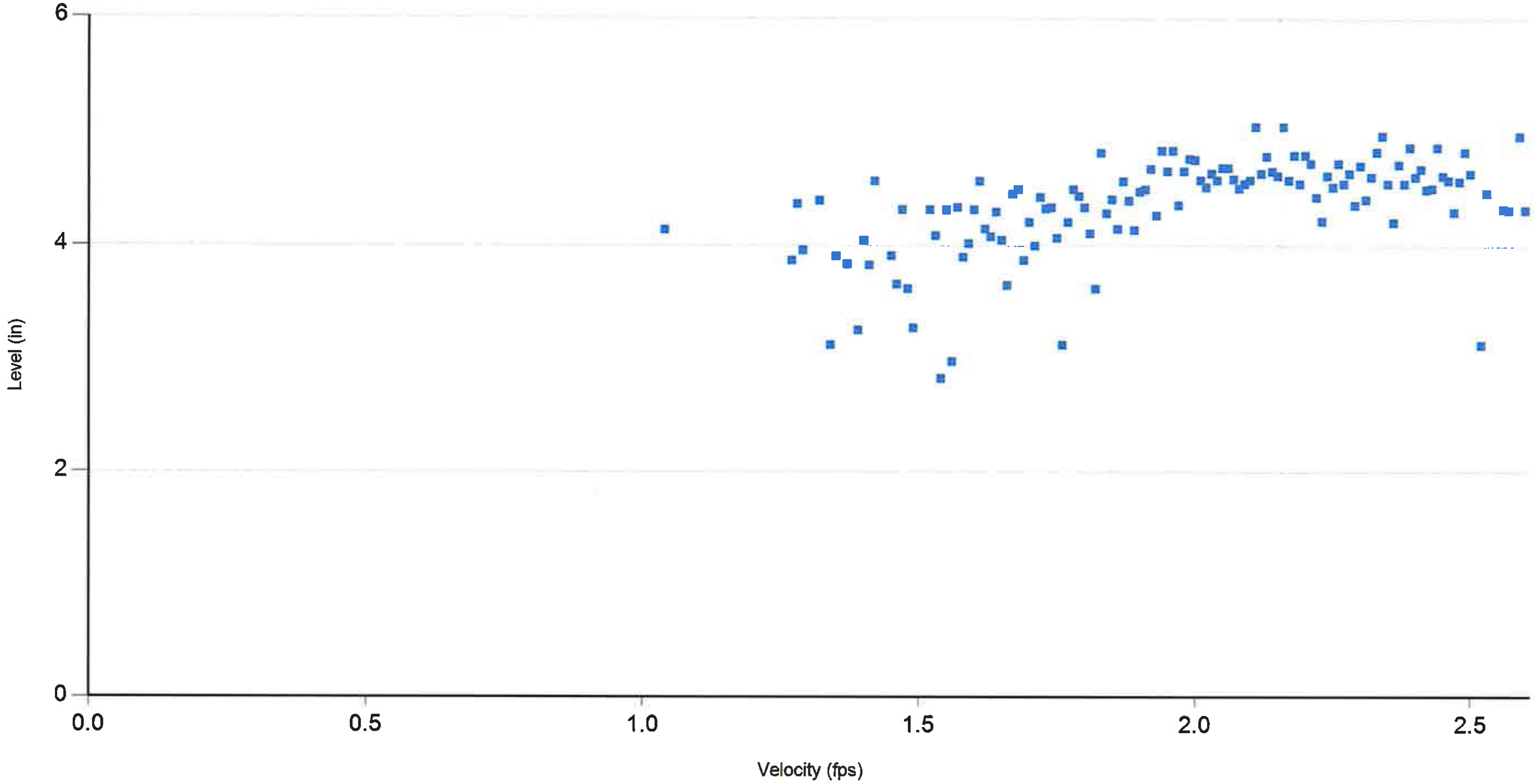
2019.01 De Anza Dr MH



	Velocity (fps)	Level (in)	Flow (gpm)	RainFall	Inches
Average	2.129	3.937	216.976		
Maximum	2.600	5.050	330.485		
Minimum	1.370	2.800	96.250		

1/29/2019 9:50:29 AM

2019.01 De Anza Dr MH



1/09/2019 thru 1/25/2019

1/29/2019 9:50:29 AM

Site Report

01-29-2019

Confidential Proprietary Information

Environmental Data Systems

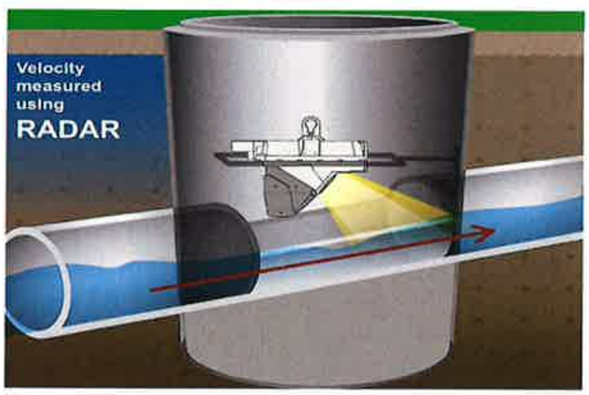
2019.01 Main St MH

~398 W Main St, San Jacinto, CA 92583
Manhole No. Unknown

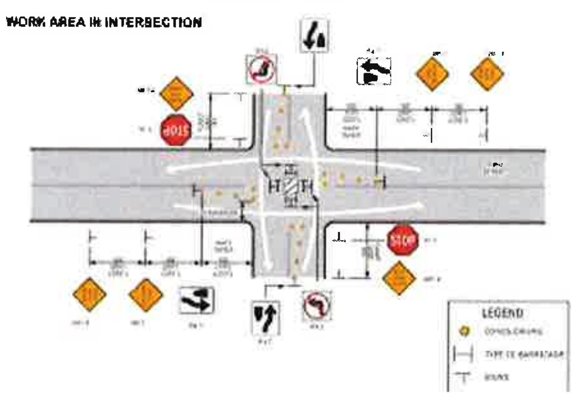
Access:	System Type:	Install Date: 1/09/2019
MH within Main St & Santa Fe Av intersection	Sanitary [X] Storm [ ]	

Map

Technology



Traffic Plan



Flow Meter

Meter Depth: 119"

MH Coordinates: 33.783861, -116.963245

Moderate open channel hydraulics

Avg Velocity	Avg Measured Level	Multiplier
2.25 fps	2.0"	1

Gas

O2	H2S	CO	LEL
20.9	0	0	0

Notes

No laterals; monitored the downstream line as it provided the best hydraulics.

Traffic Safety

No formal TCP required; used cones & signs per site-specific WATCH requirements.

Land Use

Residential	Commercial	Industrial	Trunk
X			

Manhole Depth	133"
Monitored Pipe Size	8"
Inner Pipe Size (In/Out)	8"/8"
Pipe Shape	Round
Pipe Condition	Good
Manhole Material	Brick
Silt	0"
Velocity Profile Data	*
Velocity Profile Taken	0.4 2-D
Sensor Offset	14.18"
Sensor Dist. to Crown	6.18"
Sensor Direction	Downstream
Flow Heading	West

Meter Site Document

Environmental Data Systems

2019.01 Main St MH

~398 W Main St, San Jacinto, CA 92583

Site

Manhole Before Install

Installation Process

Installed

Upstream

Downstream

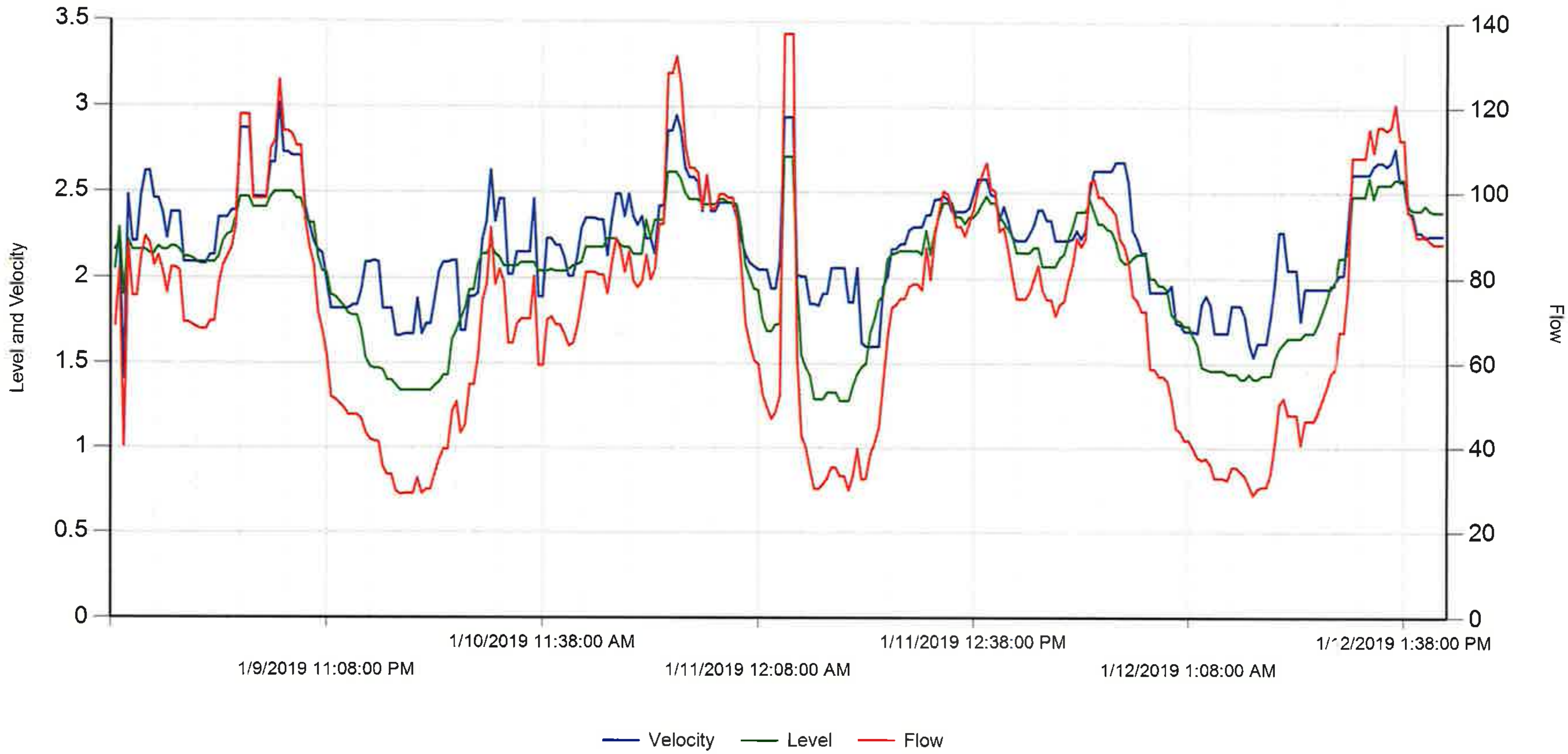
Utility Systems Science and Software

Report Date: 01/29/2019
Customer: Environmental Data Systems
Group: Luiseno Retail Proj
SiteID: 3311

Statistics for 2019.01 Main St MH : 01/09/2019 thru 01/25/2019

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Max d/D
1/9/19	85.24	126.11	40.28	0.12	0.18	0.06	2.35	3.02	1.41	2.23	2.50	1.86	122,739	0.31
1/10/19	69.52	131.74	28.82	0.10	0.19	0.04	2.18	2.95	1.66	2.00	2.62	1.34	100,111	0.33
1/11/19	73.30	137.08	29.86	0.11	0.20	0.04	2.24	2.94	1.60	2.05	2.71	1.28	105,546	0.34
1/12/19	70.01	120.83	28.75	0.10	0.17	0.04	2.16	2.76	1.54	2.02	2.59	1.41	100,819	0.32
1/13/19	78.07	146.67	28.40	0.11	0.21	0.04	2.26	3.18	1.48	2.09	2.69	1.41	112,426	0.34
Week:	**75.23**	**146.67**	**28.40**	**0.11**	**0.21**	**0.04**	**2.24**	**3.18**	**1.41**	**2.08**	**2.71**	**1.28**	**541,641**	**0.34**
1/14/19	75.40	133.12	31.46	0.11	0.19	0.05	2.24	3.09	1.68	2.08	2.58	1.32	108,578	0.32
1/15/19	80.16	175.76	29.17	0.12	0.25	0.04	2.31	3.88	1.52	2.10	2.87	1.44	115,436	0.36
1/16/19	74.64	117.85	35.90	0.11	0.17	0.05	2.28	2.80	1.49	2.05	2.51	1.47	107,478	0.31
1/17/19	73.82	123.75	31.94	0.11	0.18	0.05	2.25	2.90	1.69	2.04	2.57	1.40	106,306	0.32
1/18/19	65.62	94.24	32.71	0.09	0.14	0.05	2.08	2.42	1.66	2.01	2.39	1.47	94,491	0.30
1/19/19	74.93	149.31	25.07	0.11	0.22	0.04	2.23	3.14	1.25	2.06	2.75	1.47	107,903	0.34
1/20/19	77.00	138.47	26.81	0.11	0.20	0.04	2.26	3.49	1.44	2.08	2.69	1.41	110,885	0.34
Week:	**74.51**	**175.76**	**25.07**	**0.11**	**0.25**	**0.04**	**2.23**	**3.88**	**1.25**	**2.06**	**2.87**	**1.32**	**751,077**	**0.36**
1/21/19	85.27	165.76	28.06	0.12	0.24	0.04	2.37	3.48	1.46	2.14	2.78	1.41	122,788	0.35
1/22/19	72.17	143.75	32.29	0.10	0.21	0.05	2.20	3.10	1.63	2.05	2.69	1.46	103,932	0.34
1/23/19	68.54	101.60	33.06	0.10	0.15	0.05	2.11	2.51	1.55	2.05	2.54	1.48	98,698	0.32
1/24/19	71.21	114.51	38.26	0.10	0.17	0.06	2.19	2.68	1.64	2.05	2.54	1.47	102,549	0.32
1/25/19	57.92	89.93	37.29	0.08	0.13	0.05	2.04	2.43	1.64	1.87	2.32	1.54	83,406	0.29
Week:	**71.02**	**165.76**	**28.06**	**0.10**	**0.24**	**0.04**	**2.18**	**3.48**	**1.46**	**2.03**	**2.78**	**1.41**	**511,373**	**0.35**
Totals:	**73.70**	**175.76**	**25.07**	**0.11**	**0.25**	**0.04**	**2.22**	**3.88**	**1.25**	**2.06**	**2.87**	**1.28**	**1,804,090**	**0.36**

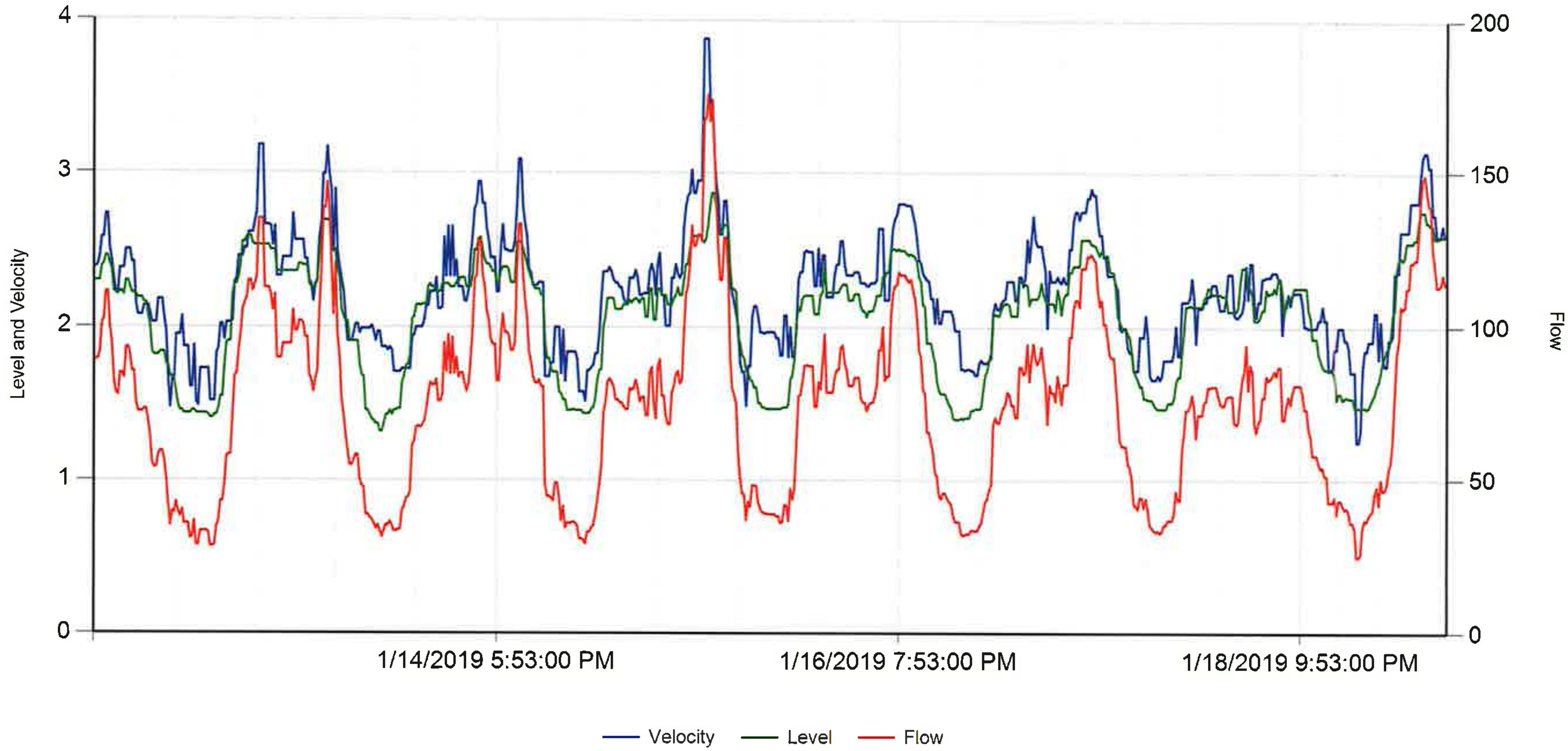
2019.01 Main St MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	2.206	2.041	72.235	RainFall	Inches
Maximum	3.020	2.710	137.083		
Minimum	1.410	1.280	28.750		

1/29/2019 9:49:51 AM

2019.01 Main St MH



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	2.238	2.065	74.925	RainFall	Inches
Maximum	3.880	2.870	175.763		
Minimum	1.250	1.320	25.069		

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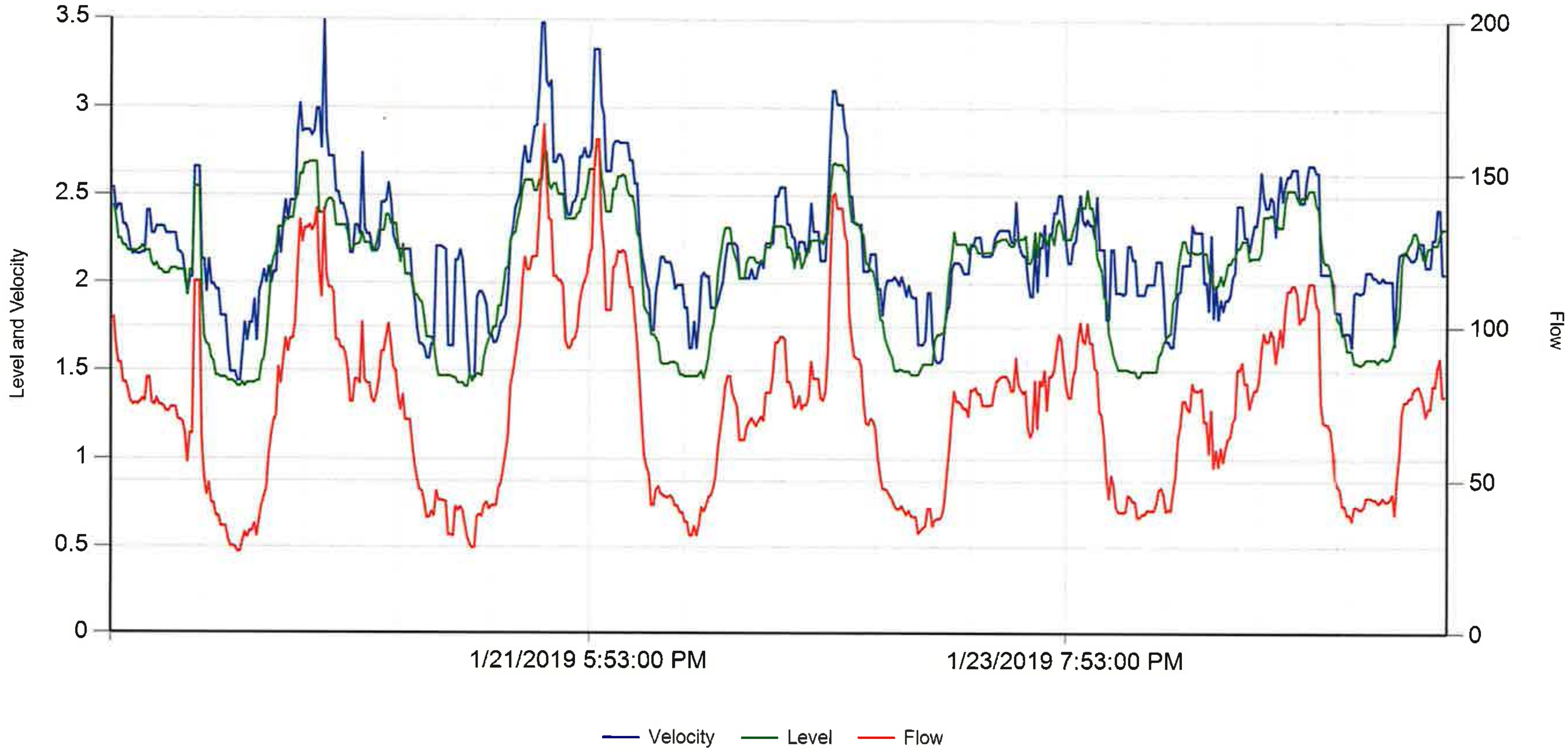
2019.01 Main St MH



	Velocity (fps)	Level (in)	Flow (gpm)	RainFall	Inches
Average	2.211	2.062	73.523		
Maximum	3.490	2.780	165.763		
Minimum	1.440	1.410	26.805		

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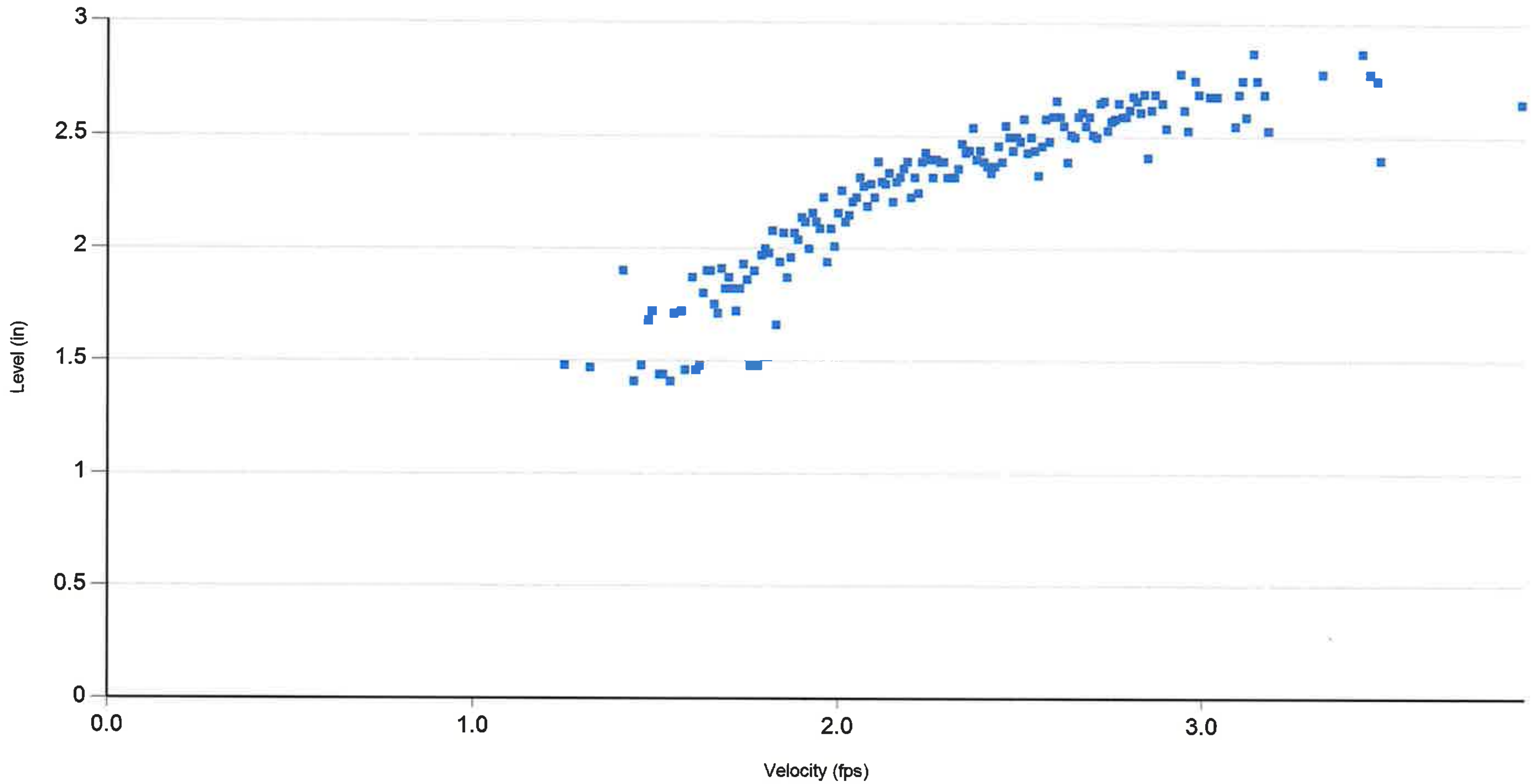
2019.01 Main St MH



1/09/2019 thru 1/25/2019

1/29/2019 9:49:51 AM

ATTACHMENT 5

REFERENCE DOCUMENTS

FINAL

2015 WASTEWATER COLLECTION SYSTEM MASTER PLAN

B&V Project No. 187976

BLACK & VEATCH
Building a world of difference.

emwd EASTERN MUNICIPAL WATER DISTRICT

APPENDIX 3A – PLANNING CRITERIA

3A.1 PLANNING CRITERIA

The purpose of a master plan is to plan for future development and assess the impact of the development to existing infrastructure performance. As part of the master plan process, areas of future growth are projected, additional infrastructure needs to serve future growth areas are identified, and recommendations are made for improvements to existing infrastructure impacted by growth. Recommendations are made using planning criteria specific to the service provider.

The following technical memorandum outlines the planning criteria used for the Eastern Municipal Water District's (District) Wastewater Collection System Master Plan Update (2015 Master Plan). The District serves five collection systems: Moreno Valley, Temecula Valley, Perris Valley, Sun City, and San Jacinto. The Sun City operational boundary is generally combined with the Perris Valley operational boundary since they are both served by the Perris Valley Regional Water Reclamation Facility (RWRF). These criteria have been developed to allow the District to evaluate their existing facilities and plan for the future, while maintaining a reliable and safe wastewater collection system:

- Wastewater Flows
  - Land use density
  - Flow per equivalent dwelling unit (EDU)
  - Peaking factors and diurnal patterns
- Pipe Capacity and Sizing
  - Allowable depth
  - Slope
  - Velocity
  - Roughness factors
- Hydraulic Modeling Approach
- Lift Station Capacity and Sizing

Note that this master planning effort does not negate the need for developers to prepare a site-specific wastewater planning studies to demonstrate that new development or redevelopment does not have negative impacts on the existing wastewater system or to identify required improvements.

3A.2 WASTEWATER FLOWS

Wastewater flows in a collection system vary significantly depending on the time of day and climatic conditions. During dry weather conditions wastewater flows are produced based on wastewater generated from various land uses, while during wet weather conditions, wastewater flows may be significantly impacted by rainfall entering the wastewater collection system. Figure 1 shows typical wastewater flow components.

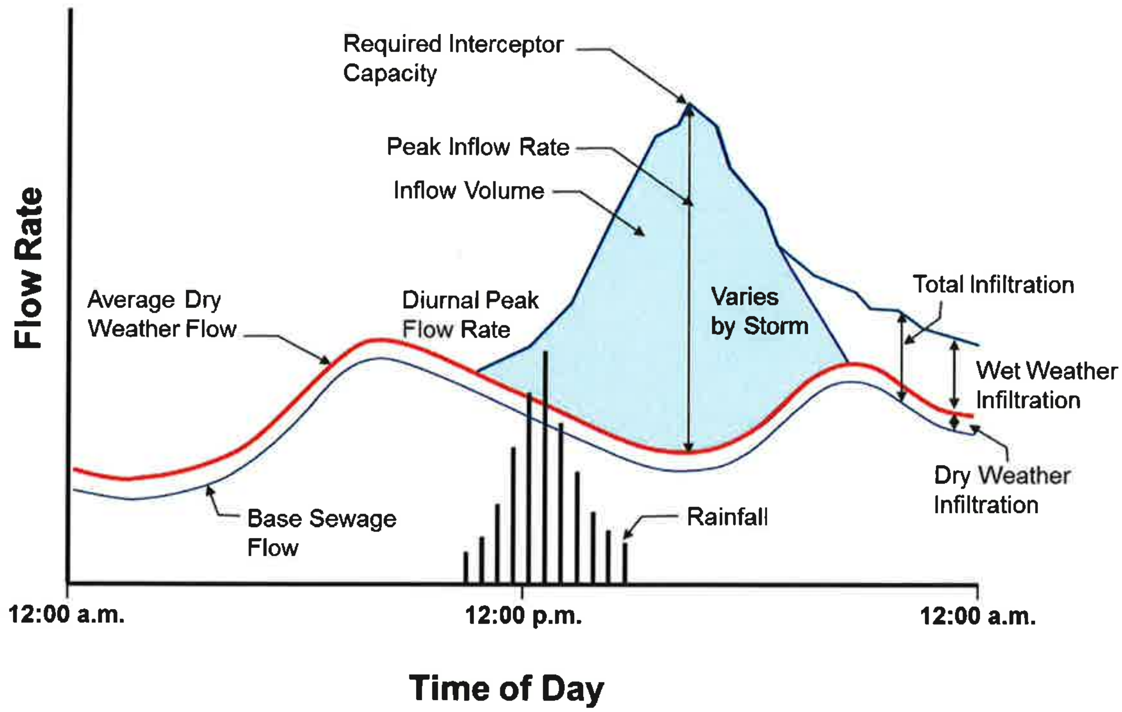


Figure 1: Typical Wastewater Flow Components

As shown, wastewater components include:

- Base sewage flow is the portion of the flow that is the return flow from customer water use.
- Average dry weather flow (ADWF) comprises of base sewage flow and dry weather infiltration. ADWF is the expected wastewater flow on a day with no precipitation events. ADWF can vary seasonally as groundwater levels change (causing fluctuations in dry weather infiltration).
- Diurnal Pattern is the change in ADWF over the course of the day and is attributable to variations in domestic, industrial, and commercial base wastewater generation.
- Infiltration is groundwater that seeps into a collection system through defective pipes, pipe joints, and manhole structures below the manhole corbel and chimney. The rate of infiltration depends on the depth of groundwater above the defects, the size of the defects, and the percentage of the collection system that is submerged. Variation in groundwater levels and the associated infiltration is both seasonal and weather dependent.
- Wet weather flows are comprised of wet weather infiltration and inflow. Wet weather infiltration is the additional infiltration that occurs due to rainfall induced higher groundwater conditions and is typically seen in the hours or days following significant rain events. Inflow is rainfall related

water that enters a collection system from sources such as private laterals, downspouts, manhole defects, foundation piping, and cross-connections with storm sewers.

The District service area receives little rainfall, making it difficult to collect meaningful rainfall data to correlate rainfall to the wet weather response in the collection system. In response to lack of rainfall data and historically low observed rates of wet weather infiltration and inflow, the District has elected to evaluate their wastewater collection system capacity based on peak dry weather flows. An allowance for wet weather flows is provided by adopting a conservative allowable depth of flow in the pipe sizing criteria, as described in Section 4.1.3.

3A2.1 EXISTING AND PROJECTED FLOWS

The District's service area includes both existing and future development. Wastewater flows are based on land use development type, development density, and flow rate by land use (gallons per day [gpd] per acre). Wastewater flows for existing and future development are calculated separately, as described in the following sections.

3A2.1.1 Existing Development

Prior to the Master Plan update, the District performed flow monitoring and sewer model calibration studies for each wastewater service area. The data obtained during the flow monitoring studies was used to calibrate the model, calculate typical unit flow factors, and develop diurnal patterns for various types of development within the service areas.

The District provided GIS land use layers for the existing development areas served by the District. The existing development flows are based on the model-calibrated unit flow factors for each land use type. Actual flows from the calibrated model were used to evaluate and analyze existing collection system capacity.

3A2.1.2 Future Development

The District maintains a Database of Proposed Projects (DOPP). The DOPP tracks information from the planning departments of cities, Riverside County, and District staff regarding proposed developments. The DOPP provides information about the type of development, size, and the anticipated number of EDUs.

In addition to the information from the known developments tracked in the DOPP, General Plan Land Use data was obtained from the cities and Riverside County to project future development to build out conditions. Development in these areas is based on less specific information than the DOPP; generally land use category and acreage.

In addition to the DOPP and general land use planning, the District also maintains detailed information about special development areas (Special Projects). These areas include unusual types of development, or redevelopment of existing areas. The anticipated development from the Special Projects is included in the future development and is described in more detail in Chapter 3.

Future development for each land use and DOPP was assigned a number of EDUs per acre for each land use category. Table 1 summarizes the assumed development densities for various land uses.

Table 1: Development Densities

LAND USE CATEGORY	UNITS	AVERAGE RESIDENTIAL DENSITY (DU/ACRE)	RESIDENTIAL (EDU/DU)	DEVELOPMENT DENSITY (EDU/ACRE)
Residential Land Use				
Estate Density	DU	0.5	1.5	0.8
High Density	DU	12	0.7	8.4
Low Density	DU	2	1.3	2.6
Medium Density	DU	4.5	1	4.5
Medium High Density	DU	6	0.9	5.4
Mobile Home Park	DU	10	0.65	6.5
Rural Mountainous (1)	DU	0.1	3	0.3
Rural (1)	DU	0.2	3	0.6
Very High Density	DU	17	0.65	11.1
Very Low Density (1)	DU	1	1	1.5
Non-Residential Use				
Agriculture (1)	acre			0
Business Park/Light Industrial	acre			5
Business Park/Light Industrial/Warehouse	acre			1.25
Commercial Office	acre			5
Commercial Retail	acre			5
Heavy Industrial	acre			7.5
Hospital	acre			5
Mixed Use Policy Area	acre			5
Open Space (Conservation, Landscape, Recreation, Rural, or Water) (1)	acre			5
Public Facilities (Municipal or School)	acre			5

(1) The following uses were assumed to be served by septic systems and do not contribute flow to the wastewater collection system: Rural Mountainous, Rural, Very Low Density, and Agriculture, and Open Space.

3A2.1.3 Flow Per Equivalent Dwelling Unit

For all types of development, the land use categories were converted to EDUs based on Table 1. Wastewater flow (ADWF) was calculated by multiplying the number of EDUs per land parcel by a rate of 235 gpd/EDU; the District's criteria used for regional planning.

3A2.2 PEAKING FACTORS AND DIURNAL PATTERNS

Peaking factors and diurnal curves are applied to the existing and projected wastewater flows and are used to evaluate the collection system capacity and to appropriately size recommended improvements.

3A2.1.4 Peaking Factor Curve

A peaking factor curve was developed based on the results from the calibration studies to project peak dry weather flow for a given average dry weather flow. The peaking curve is used for sizing pipe replacements or extensions.

The curve is shown in Figure 2 and is described by the equation PF = 2.13 $Q_{ADWF}^{-0.13}$, where Q_{ADWF} is the average dry weather flow and PF is the peaking factor. The peak flow is estimated by multiplying Q_{ADWF} times PF. The maximum peaking factor was identified as 2.87, so all flows less than or equal to 0.1 mgd are assumed to have a peaking factor of 2.87.

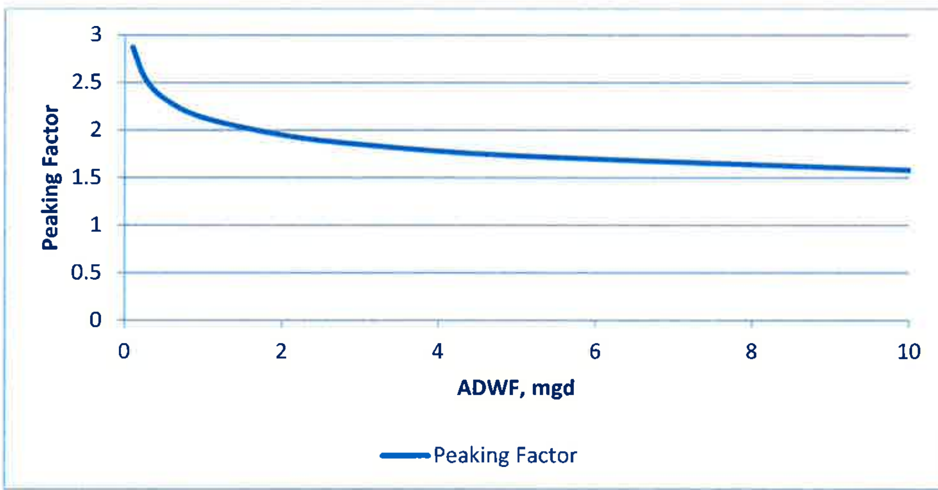


Figure 2: Peaking Factor Curve

3A2.1.5 Diurnal Patterns

The diurnal patterns developed during the calibration studies will be used to evaluate and analyze existing collection systems. For modeling future development, two diurnal patterns were developed; one for use with residential land use and the other for non-residential land use. Each pattern represents a 7-day period beginning at 1:00 a.m. on Saturday and continuing to midnight on Friday. The patterns were developed using the following rules:

- Each day, a peaking factor of 2.87 is achieved for two hours
- The flows are normalized over a 24-hour period (average PF of 1)
- Diurnal patterns can only be applied to loads ≤ 0.1 mgd (~425 EDUs)
- Patterns were based on typical residential or office/retail curves to establish the timing of the peak and minimum flows

Figure 3 shows the standard residential and non-residential diurnal patterns to be used in the model for future flows.

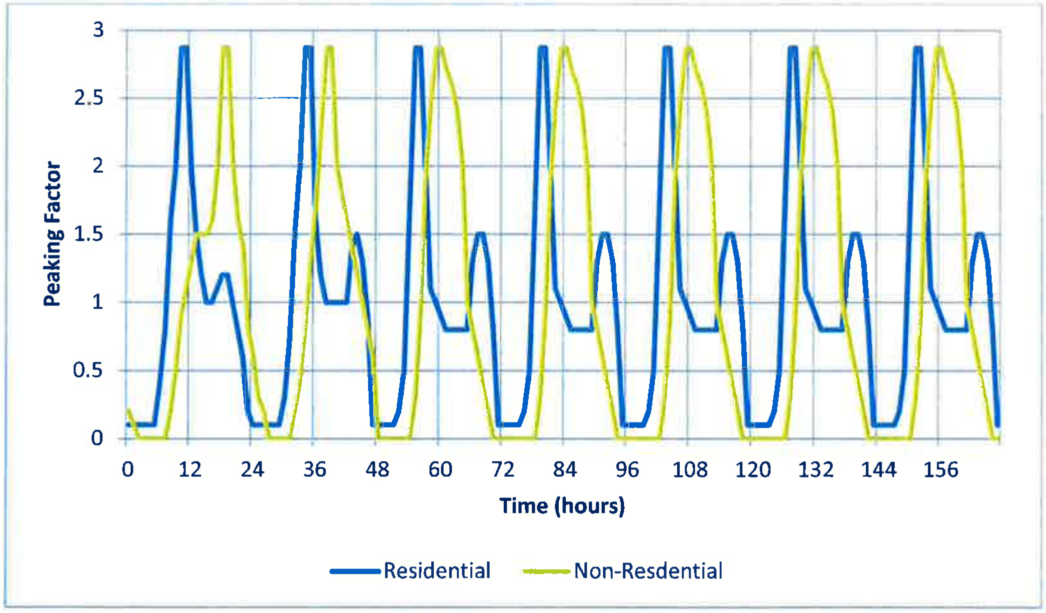


Figure 3: Residential and Non-Residential Patterns

Additional diurnal patterns were created for two of the Special Projects in Temecula Valley, Old Town and Wine Country, to account for the impacts of special events that take place within these areas. These areas in Temecula Valley have been observed to have higher peaking factors at different times in comparison to other areas due to the additional flow generated during special events, such as festivals. These patterns follow the same rules as the standard curves with the exception of having a peaking factor of 3.00 instead of 2.87. Figure 4 shows the patterns for old town and wine country.

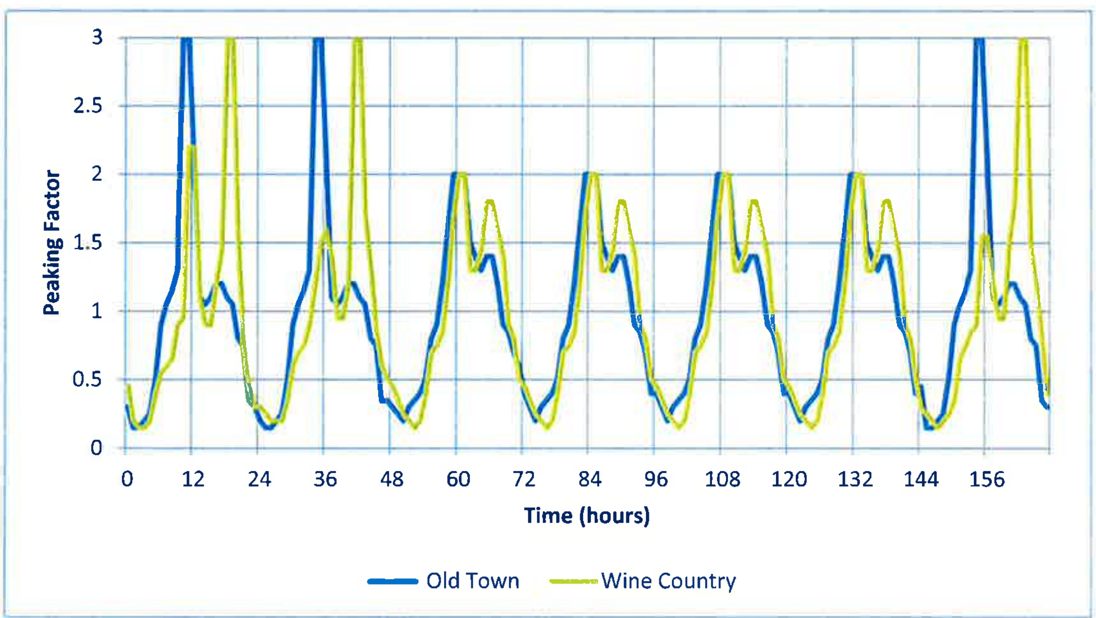


Figure 4: Old Town and Wine Country Patterns

3A.3 HYDRAULIC MODELING APPROACH

The District's existing calibrated wastewater models for each basin use an extended period simulation to analyze their existing collection systems under average dry weather flow and peak dry weather flow. To analyze the collection systems for future growth, various approaches were discussed with the District. Black & Veatch prepared a pilot model using the Moreno Valley hydraulic model to test three different approaches for peaking future flows. The three approaches and general results are summarized below.

- **Approach 1:** Perform steady state runs using a peaking factor equation. This approach may overestimate expected flows, but provides a level of protection/conservatism.

- **Approach 2:** Existing flows are peaked using the calibrated diurnal patterns and future flows are applied to the model using a constant peaking factor of 2.87 (extended period simulation). This approach generally overestimates results as compared to the PF equation.

- **Approach 3:** Existing flows are peaked using the calibrated diurnal patterns (extended period simulation). Representative diurnal patterns identified in Section 2.2.2 reflect the typical shape of the calibration patterns but are adjusted to meet the 2.87 peaking factor. This approach generally underestimates results as compared to the PF equation, but may provide results that better align with existing or expected system flows.

It was decided that the system would be evaluated using Approach 3 to identify CIP projects and Approach 1 will be used to size the new facilities. Approach 3 will generate the most likely/expected flows caused by future development. Model results will be assessed against the District's planning criteria and CIP projects will be identified where the criteria are not met. Where deficiencies are identified using Approach 3, the peaking factor equation (Approach 1) will be used to estimate the projected wastewater flow for the new facility. It has been established that new facilities will be sized for build out conditions, so it is expected that Approach 1 would only be performed under the build-out modeling scenario.

3A.4 CAPACITY AND SIZING CRITERIA

The capacity and sizing criteria are used both to evaluate existing capacity due to future growth and to size new facilities to serve future developments. In some cases the existing facilities are allowed to exceed the criteria especially if additional growth in the area is not expected and no problems with operations have been reported.

3A4.1 GRAVITY PIPES

The capacity of a gravity pipe is a function of its slope, diameter, and roughness. Manning's formula for open-channel flows is used to calculate flow capacity in gravity mains:

$$Q = (1.486/n)\ AR^{2/3}\ S^{1/2}$$

Where:

Q = flows, cfs
n = Manning's coefficient of roughness
A = cross sectional area of pipe, cu ft
R = hydraulic radius (flow area divided by wetted perimeter), ft
S = slope of the pipe, ft/ft

The District assumes a Manning's coefficient of 0.013 for all wastewater pipe material and uses a minimum pipe size of 8 inches for new collection system pipe.

3A4.1.1 Velocity Criteria

Velocity is an important criterion for proper operation of a wastewater collection system. The District requires that pipe velocities be designed for 2 fps to 10 fps.

The minimum allowable velocity is 2 fps at calculated peak dry weather flow to avoid excessive deposition of solids in the collection system. In pipes where the minimum criterion will not be achieved on a regular basis, or will not be achieved for many years, the District will need to make arrangements to clean the pipes on a regular basis.

Velocities in excess of 10 fps could result in excessive wear on the pipe due to the abrasive nature of grit in the wastewater flow. Typically, drop manholes can be used to avoid peak velocities in excess of 10 fps, but may cause odor problems.

3A4.1.2 Slope

A minimum slope is set for each pipe size to help ensure acceptable velocity and avoid solids deposition in the collection system. Table 2 summarizes the minimum slope for various pipe sizes used for the Master Plan.

Table 2: Minimum Pipe Slopes

PIPE SIZE (INCHES)	MINIMUM SLOPE (FT/FT)	PIPE SIZE (INCHES)	MINIMUM SLOPE (FT/FT)
8	0.0040	21	0.0012
10	0.0032	24	0.0010
12	0.0024	27	0.0010
15	0.0016	30	0.0010
18	0.0014	36	0.0010

3A4.1.3 Depth to Diameter (d/D) Criteria

Depth to Diameter (d/D) is the ratio of the depth of wastewater to the diameter of the pipe. The table below shows the design criteria for gravity mains. All new sewer mains less than 15 inches in diameter shall be sized to carry the projected PDWF at a depth not greater than half of the diameter of the pipe (d/D not to exceed 0.5). New sewer mains 15 inches and larger shall be sized to carry the projected PDWF at a depth of flow not greater than 70 percent of the diameter of the pipe (d/D not to exceed 0.7). Table 3 provides a summary of pipe design criteria for capacity evaluation.

Table 3: Gravity Pipe Capacity Design Criteria

INFRASTRUCTURE	PEAK ADWF D/D	MANNING'S N	MINIMUM VELOCITY (FPS)	MAXIMUM VELOCITY (FPS)
Diameter < 15 inches	< 0.5	0.013	2	10
Diameter ≥ 15 inches	< 0.7	0.013	2	10

Note: The minimum pipe size for new collection system pipe is 8 inches.

3A4.2 LIFT STATIONS AND FORCE MAINS

Based on historical flow data, the District has determined that a 20% allowance for wet weather flows is adequate for lift station capacity planning. The District's lift stations and force mains are evaluated based on the ability to service the Peak LS Flow (Peak ADWF x 1.2).

3A4.1.4 Lift Stations

Lift station capacity is evaluated in terms of total capacity and firm capacity. The total capacity is the maximum capacity of the lift station with all pumps operating. The firm capacity is defined as the capacity of the lift station with the largest pump out of service. Lift stations will be evaluated to determine both total and firm capacity of the station.

The capacity of a lift station is dependent upon the pumping capacity and the system head that is experienced in the downstream force main. The system head is determined by the static pumping requirements as well as the head loss experienced through the force main under the varying flow conditions. The system head is determined using the force main diameter, length, assumed C-factor, and static pump requirements (wet well and discharge elevation).

For each station, the pump curves will be plotted against the system head curve that is expected to occur under the peak lift station flow for all planning years. Figure 5 shows an example lift station capacity assessment graph for the Day Street Lift Station.

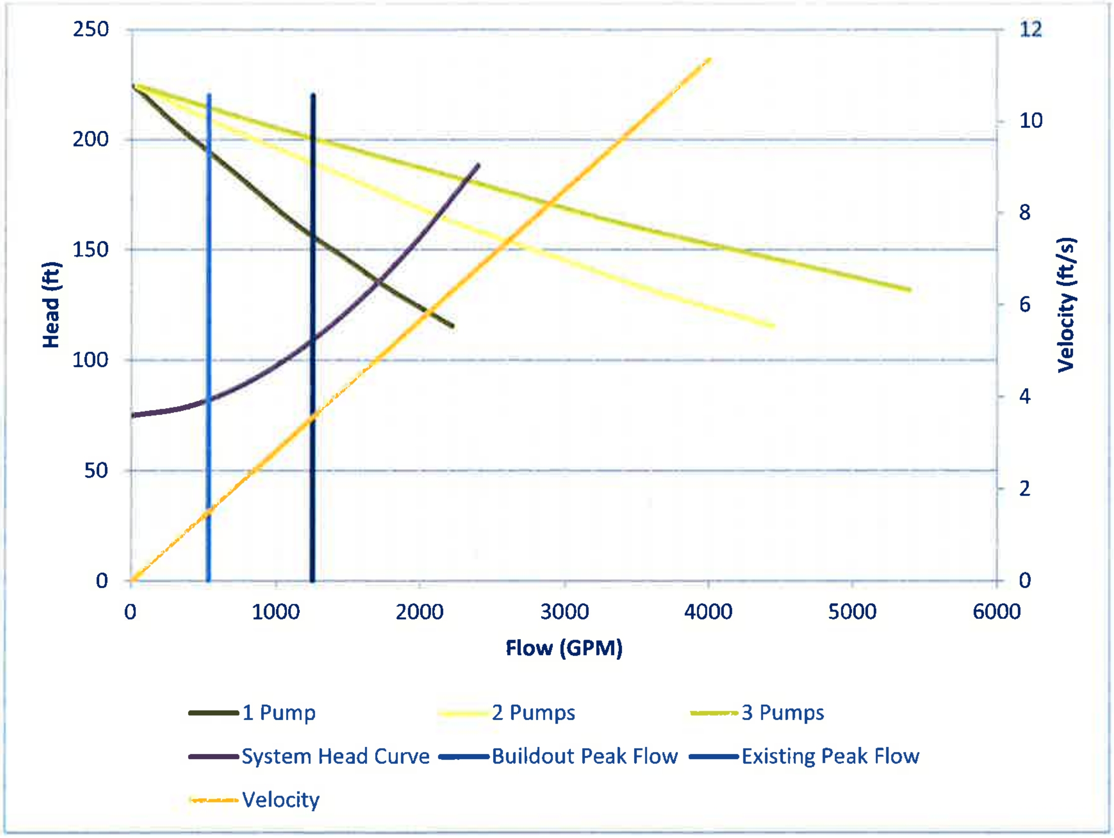


Figure 5: Day Street Lift Station Capacity Assessment

The capacity assessment graph for each lift station will determine the existing lift station capacity as well as future flow and head pumping requirements. All lift stations will be sized to provide adequate firm capacity to pump Peak LS Flow at build-out conditions

3A4.1.5 Force Mains

The capacity of a force main pipe is a function of the velocity in the pipe. The Hazen-Williams equation is used to calculate flows in force mains:

$V = 1.318CR^{0.63}S^{0.54}$

Where:

V = Velocity, fps
C = Hazen-Williams coefficient of roughness
R = hydraulic radius (flow area divided by wetted perimeter), ft
S = Slope of energy grade line, ft/ft

The District assumes a Hazen-Williams coefficient value of 100 for all force mains. Velocity is the major criterion when sizing force mains. In general, force mains should be sized to convey Peak LS Flows at build out conditions with a velocity between 2 fps and 6 fps. Velocities less than 2 fps will result in wastewater spending additional time in the force main, which can cause downstream operational problems. Force mains with a velocity greater than 6 fps tend to have excessive head loss and can affect the ability of the lift station to operate properly.

APPENDIX 3B – COORDINATION WITH WATER MASTER PLAN

3B.1 COORDINATION WITH WATER MASTER PLAN

The 2015 Update is being developed concurrently with the District's Water System Master Plan which is being updated by a separate consultant. The District is interested in maintaining consistency and comparable appearance between its wastewater and potable water hydraulic models. In an effort to maintain consistency, the District provided the following information for the both sewer and potable water models:

- Additional user information fields for the nodes and pipeline tables in the models.
- Model scenarios for all planning years: 2014, 2016, 2018, 2020, 2022, 2025, 2030, 2035, 2045, 2065, 2099 (build-out).
- Pre-set database queries.

3B.1.1 Additional Hydraulic Model Fields

The District added additional fields to the "Element Information" tables in the wastewater hydraulic model for manholes and pipelines. No existing information fields were removed from the table and no existing information was cleared. Table 3B-1 shows the additional fields: 23 additional fields for the manhole table and 8 additional fields for the pipeline table.

Table 3B-1 Wastewater Hydraulic Model Additional Informational Fields

TABLE	LS_LOCAL	LS_REGNL	FLOW_NODE	ALT	CIP_ID	COMMENT	DOPP_NODE	DOPP_ID	METER_Q	METER_YEAR	SUBAGENCY	2016_DOPP	2018_DOPP	2020_DOPP	2022_DOPP	2025_DOPP	2030_DOPP	2035_DOPP	2045_DOPP	2065_DOPP	ULT_DOPP	ULT_LU_Q	ULT_SS_Q	DOPP_PIPE	EXST_D/D	EXST_Q
MH	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X			
Pipe	X	X		X	X	X																		X	X	X

3B.1.2 Hydraulic Model Scenarios

All four hydraulic models provided by the District included separate hydraulic model scenarios for each planning year: 2014 (Existing), 2016, 2018, 2020, 2022, 2025, 2030, 2035, 2045, 2065, and Build-out. Each year contains two scenarios: capacity analysis and capital improvement program (CIP) analysis. The capacity analysis uses existing 2014 facilities for all scenarios; however, the flows vary in each scenario, corresponding to respective years. The CIP analysis uses CIP facilities and flows corresponding to each respective year. All scenarios in the model utilize the same pipe data set; however node data changes for each planning year.

3B.1.3 Hydraulic Model Queries

The District created database queries in the wastewater model similar to the queries created in the water model. These queries include database queries for MHs, Pipes (PI), Pumps (PU), and Wet Wells (WW) based on facility installation year. Existing and new facilities are retired or become active based on the [Installation Year] and [Retirement Year] field. Queries are used to select the appropriate facilities for each scenario. The field called YR_INST is populated with year of installation and the queries can be used to identify facilities needed based on each planning year. The years for these queries correspond to the District's plan for existing and future capital improvements. The same years are used for facility selection as seen in model scenarios: 2014 (Existing), 2016, 2018, 2020, 2022, 2025, 2030, 2035, 2045, 2065, and build-out. The queries have the following naming convention with YA referring to the active year of installation:

- [YA_20XX_MH/PI/PU/WW]. For example for year 2020 pipe query, the naming for that query is YA_2020_PI.

3B.1.4 Future Wastewater Flows

As discussed in Chapter 3, future ADWF is allocated in the model along with corresponding diurnal patterns to simulate flow fluctuations, including the PDWF, within the collection system. The District estimates future wastewater flows using future land use categories and the DOPP. The District owns and maintains the DOPP to track planned development. For the 2015 Update, future development data was extracted from this database into point, line and polygon shapefiles in GIS. The polygons represent the physical area of the proposed / future developments / projects. The point layer places a point at the center of the polygon (called a DOPP point), and the line layer displays a pipe (called a DOPP pipe) from the DOPP point to an existing manhole, which represents the entrance of the flow into the wastewater collection system. The District determines the entrance point (either an existing or future MH) by performing a locating routine using GeoWizard to automatically attribute a downstream manhole to the DOPP pipe based on proximity.

A second step was performed by the District to verify downstream manhole locations for each DOPP node and pipe. This included the following process to verify the location of the downstream manholes and update the DOPP pipe and node databases.

A field called (LOC_VERF) was added to the DOPP pipeline database to document verification progress and populated with the following information:

- "Yes" – Downstream location is verified.
- "Yes, updated" – Downstream location was updated to a more appropriate MH. The length field was recalculated and [Facility] field was updated with correct manhole number (MHXXX).
- "No, large DOPP" – DOPP basin covers a large area over multiple MHs; the DOPP will need to be evaluated and flows split to appropriate MHs as part of the 2015 Update.
- "No, split DOPP" – DOPP basin polygon is not contiguous; the DOPP will need to be evaluated and flows split to appropriate MHs as part of the 2015 Update.
- "No, MP to review" – Downstream location unclear; the DOPP will need to be evaluated and flow allocated to appropriate MHs as part of the 2015 Update.

1. Verified downstream connection using contour layer, existing pipe network and DOPP polygon.
   - Contour layer – Checked direction of grade to verify correct downhill manhole
   - Existing pipe network – Checked existing pipeline to confirm the DOPP pipe is not crossing a property
   - DOPP polygon – Checked if polygon is near the stub-out of another development, if so, track back to that line
2. Added fields to DOPP
   - DOPP MH attribute table (for both commercial and single family residential (SFR)):
     - [INSTALL_YR], [RETIRE_YR], [MHRIM_FT], [MHINV_FT], [DOPP_Node], [MH_DIA_FT], [DOPP_ID]
   - DOPP pipe attribute table (for both commercial and SFR):
     - [INSTALL_YR], [RETIRE_YR], [DOPP_ID], [DOPP_Pipe], [DIA_IN], [MANN_N], [LENGTH_FT], [UpMH], [DnMH], [DnMH_GIS], [Pipe_ID], [UPINV_FT], [DNINV_FT]

As a final step, flows into the appropriate MHs were verified and the DOPP files were populated with information fields for use in importing DOPP nodes and pipes into the wastewater model.