DEXTER WILSON ENGINEERING, INC.

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SEWER CAPACITY STUDY FOR THE KELLER CROSSING PROJECT

April 13, 2021

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

SEWER CAPACITY STUDY
FOR THE
KELLER CROSSING PROJECT

April 13, 2021

Prepared For:
Eastern Municipal Water District
2270 Trumble Road
Perris, CA 92572-8300

Prepared by:
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TABLE OF CONTENTS

PAGE NO.

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April 13, 2021 544-054

Eastern Municipal Water District
2270 Trumble Road
Perris, CA 92570

Attention: New Business Development Department

Subject: Sewer Capacity Study for Keller Crossing Project

A. OBJECTIVE

This letter report provides a sewer capacity study for the proposed Keller Crossing project. The project is located along the north side of Keller Road between Pourroy Road and Highway 79 in Winchester. The project is proposed to be served by the Eastern Municipal Water District (EMWD).

The project consists of approximately 191 acres and proposes land uses that include single family residential, high density residential, commercial, park, and open space areas. Attachment 1 provides a vicinity map and land use plan for the project.

The objective of this study is to present EMWD with the capacity analysis for the Keller Crossing project to a point of connection with a proposed 12-inch sewer line in Pourroy Road at the southwest corner of the project.

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B. ANALYSIS CRITERIA

The analysis in this letter-report was prepared based on the criteria the EMWD 2015 Wastewater Collection System Master Plan prepared by Black and Veatch. The pertinent criteria from that document are summarized as follows:

- Medium Residential Average Day Flow Factor – 235 gpd/unit
- High Density Residential Average Flow Factor – 152.75 gpd/unit
- Commercial Average Day Flow Factor 1,700 gpd/acre
- Park/Recreation Average Day Flow Factor – 500 gpd/acre
- Peak Flow Factor – Per Figure 2 in Attachment 4
- Mannings "n" Coefficient – 0.015
- 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 ANALYSIS

This sewer capacity study has been prepared based on EMWD planning criteria and evaluates the proposed sewer system for the Keller Crossing project. Attachment 2 provides a proposed sewer system map for the project. The off-site 12-inch sewer line in Pourroy Road was sized by others.

Table 1 summarizes the projected flows for the Keller Crossing project. The sewer system has been broken into several reaches for evaluation. Attachment 3 provides the supporting calculations and corresponding node diagram. In evaluating sewer slopes used in the analysis, street slopes were used for planning purposes and actual pipe slopes should be confirmed during final engineering of the project. A summary of sewer flows by sewer node is provided as Table 2.

TABLE 1 KELLER CROSSING PROJECT PROJECTED SEWER FLOWS			
Land Use	**Quantity**	**Unit Flows**	**Total Average Flow, gpd**
SF Residential	356 units	235 gpd/unit	83,660
Very High Density Residential	80 units	152.75 gpd/unit	12,220
Commercial	18.04 ac	1,700 gpd/ac	30,668
Parks/Recreation	6.5 ac	500 gpd/ac	3,250
TOTAL			**129,798**

TABLE 2 TRIBUTARY FLOWS BY NODE			
Node	**Tributary Land Use**	**Flow Factor**	**Average Flow, gpd**
18	120 SFD 80 VHDR 18.04 Ac Commercial 6.5 Ac Parks/Recreation	235 152.75 1700 500	28,200 12,220 30,668 3,250
14	18 SF Units	235	4,230
12	19 SF Units	235	4,465
10	167 SF Units	235	39,245
8	32 SF Units	235	7,520
TOTAL			**129,798**

D. CONCLUSION

The results of the hydraulic calculations in Attachment 3 support the recommended sewer line sizing for the project. Proposed gravity sewer lines range from 8-inch to 12-inch as shown on the figure in Attachment 2.

Dexter Wilson Engineering, Inc.

Stephen M. Nielsen

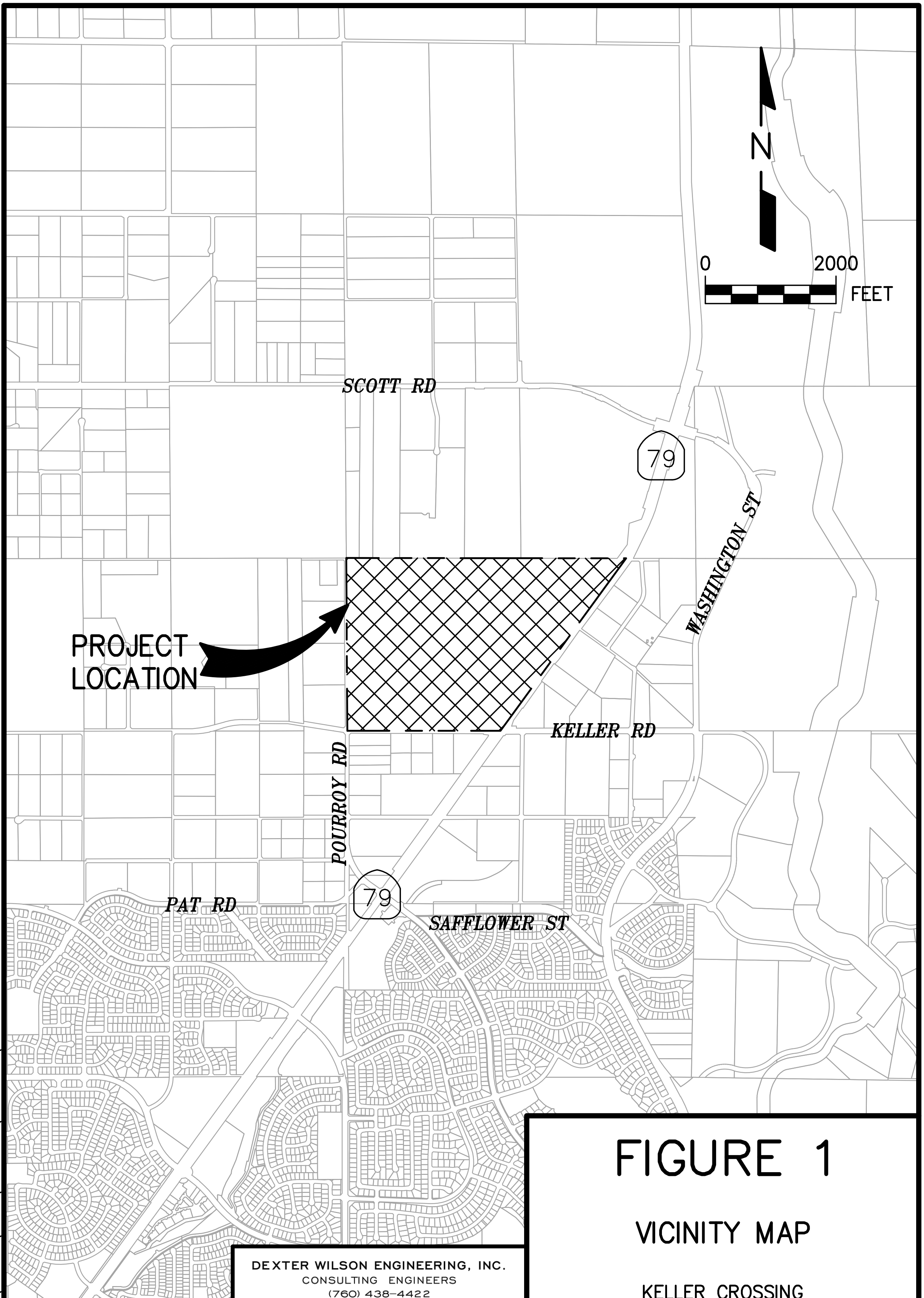
Stephen M. Nielsen, P.E.

SMN:ah

Attachment(s)

ATTACHMENT 1

VICINITY MAP AND LAND USE PLAN



\ARTIC\DWG\544054\REPORT\KC_FIGURE1_VM.DWG 04-12-21 10:31:17 LAYOUT: 8X11

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FIGURE 1

VICINITY MAP

KELLER CROSSING

ATTACHMENT 2

PROPOSED SEWER SYSTEM

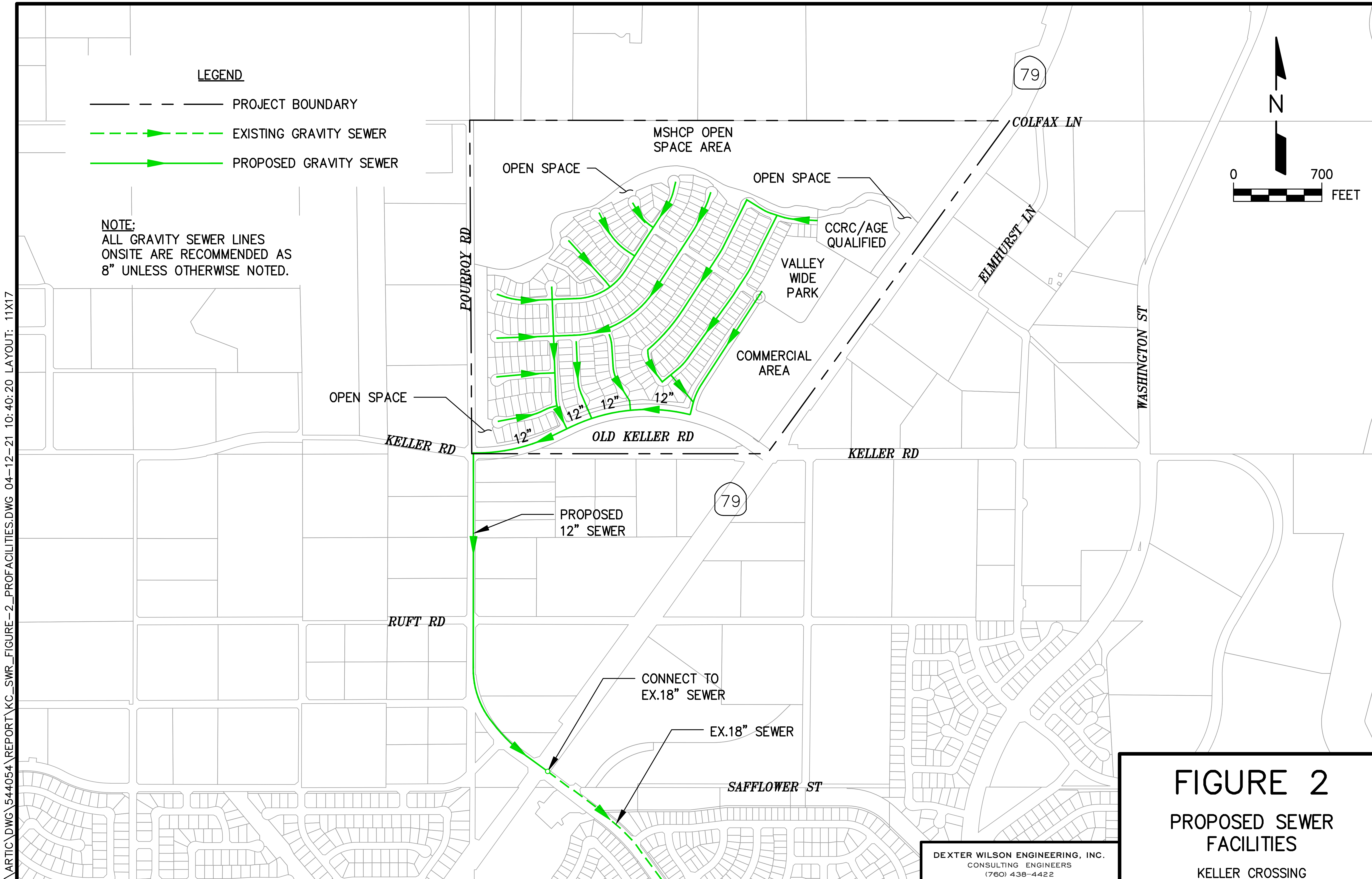

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FIGURE 2

PROPOSED SEWER FACILITIES

KELLER CROSSING

ATTACHMENT 3

SUPPORTING CALCULATIONS

DATE: 3/30/2021

FOR: SEWER STUDY SUMMARY Keller Crossing

BY: Dexter Wilson Engineering, Inc.

3 NUMBER: 544-054

FROM	TO	IN-LINE FLOW (gpd)	AVG DRY WEATHER FLOW (gpd)	EQUIV POP.	PEAKING FACTOR	PEAK WET WEATHER FLOW (gpd)	PEAK WET WEATHER FLOW (DESIGN FLOW)		LINE SIZE (inches)	DESIGN SLOPE (%)	DEPTH K'[1]	dn (feet)	dn/D[2]	C_a for Velocity[3]	VELOCITY (f.p.s.)	Remarks
							M.G.D.	C.F.S.								
18	16	74338	74,338	743	2.87	213,350	0.21	0.33	8	0.60	0.188482	0.29333	0.44	0.3328	2.23	120 SFD + 80 VHDR + 6.5 AC Park + 18.04 AC Commercial
16	14		74,338	743	2.87	213,350	0.21	0.33	12	0.86	0.053397	0.23000	0.23	0.1365	2.42	MH 18
14	12	4230	78,568	786	2.87	225,490	0.23	0.35	12	0.24	0.106831	0.33000	0.33	0.2260	1.54	18 PA 2 Units + MH 16
12	6	4465	83,033	830	2.87	238,305	0.24	0.37	12	0.70	0.066109	0.26000	0.26	0.1623	2.27	19 PA 2 units + MH 14
10	8	39245	39,245	392	2.87	112,633	0.11	0.17	8	1.00	0.077076	0.18667	0.28	0.1800	2.18	167 SFD units
8	6	7520	46,765	468	2.87	134,216	0.13	0.21	8	1.20	0.083843	0.19333	0.29	0.1890	2.47	32 PA 1 Units + MH 10
6	4		129,798	1,298	2.78	360,515	0.36	0.56	12	0.50	0.118335	0.34000	0.34	0.2355	2.37	MH 12 + MH 8

1 K' based on n = 0.015
2 dn/D using K' in Brater King Table 7-14
3 From Brater King Table 7-4, based on dn/D

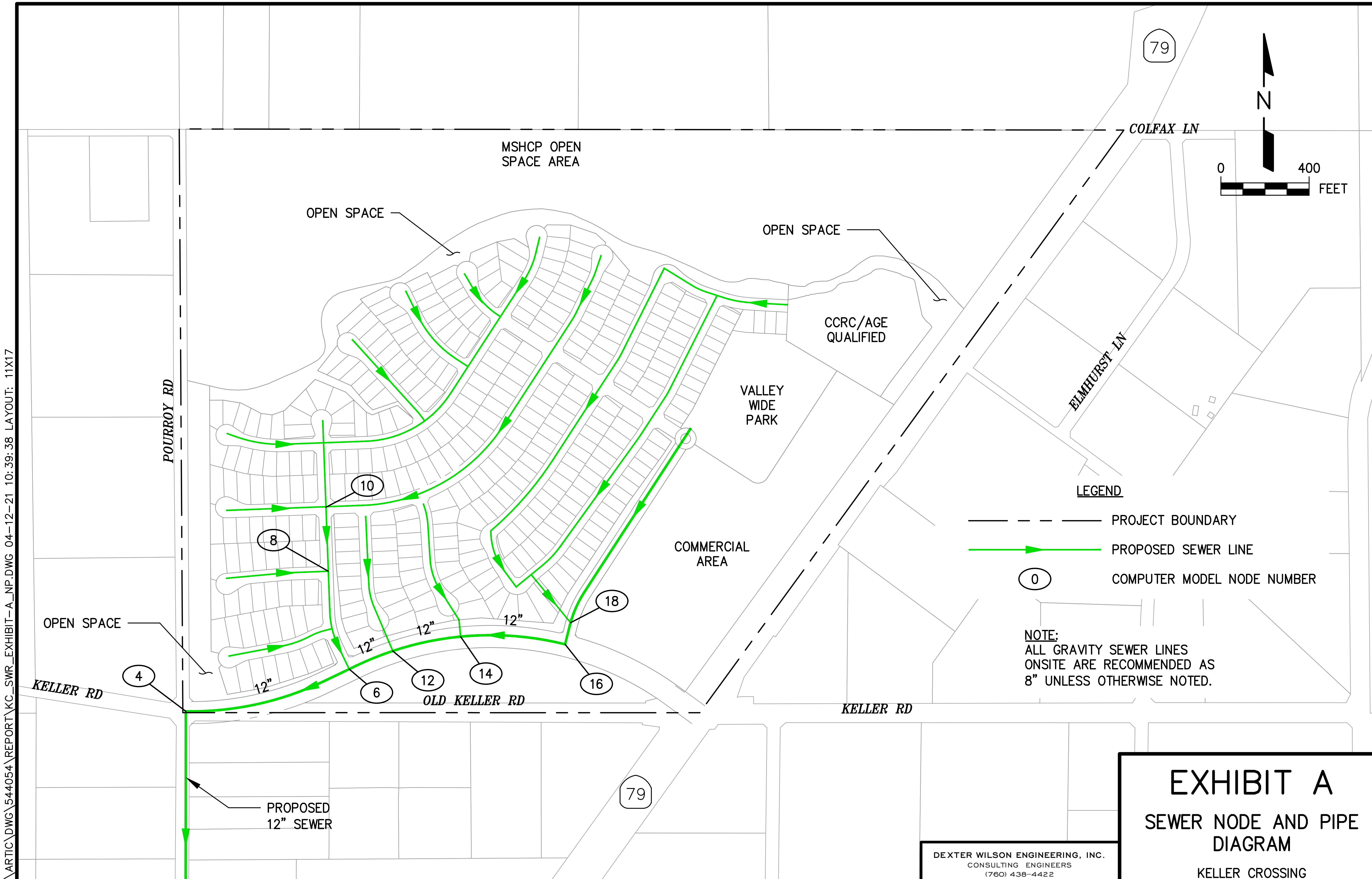

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EXHIBIT A

SEWER NODE AND PIPE DIAGRAM

KELLER CROSSING

ATTACHMENT 4

REFERENCE DOCUMENTS

Master Plan Supplement

FINAL

Planning and Sizing Criteria

2015 WASTEWATER COLLECTION SYSTEM MASTER PLAN

B&V Project No. 187976

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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.

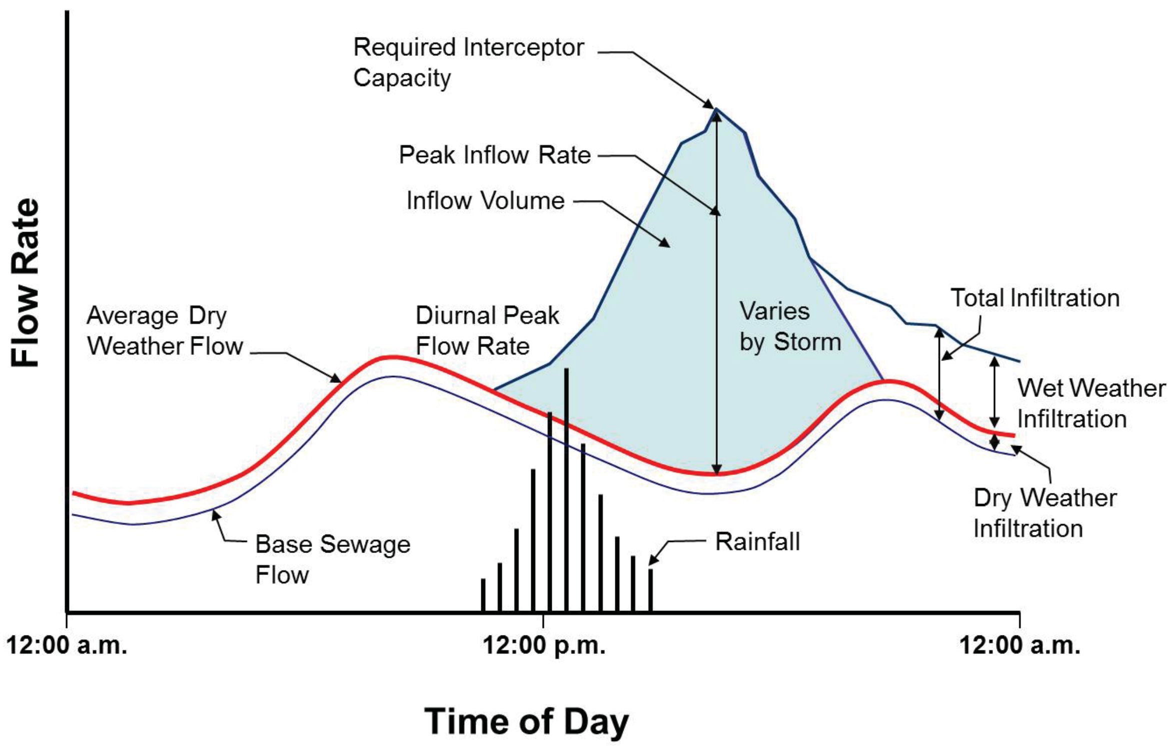

Figure 1: Typical Wastewater Flow Components

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.

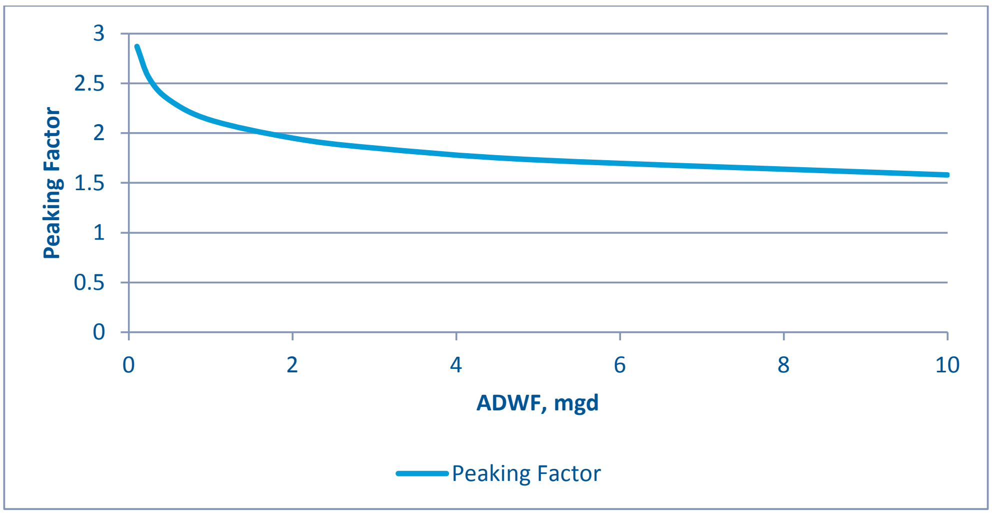

Figure 2: Peaking Factor Curve

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.

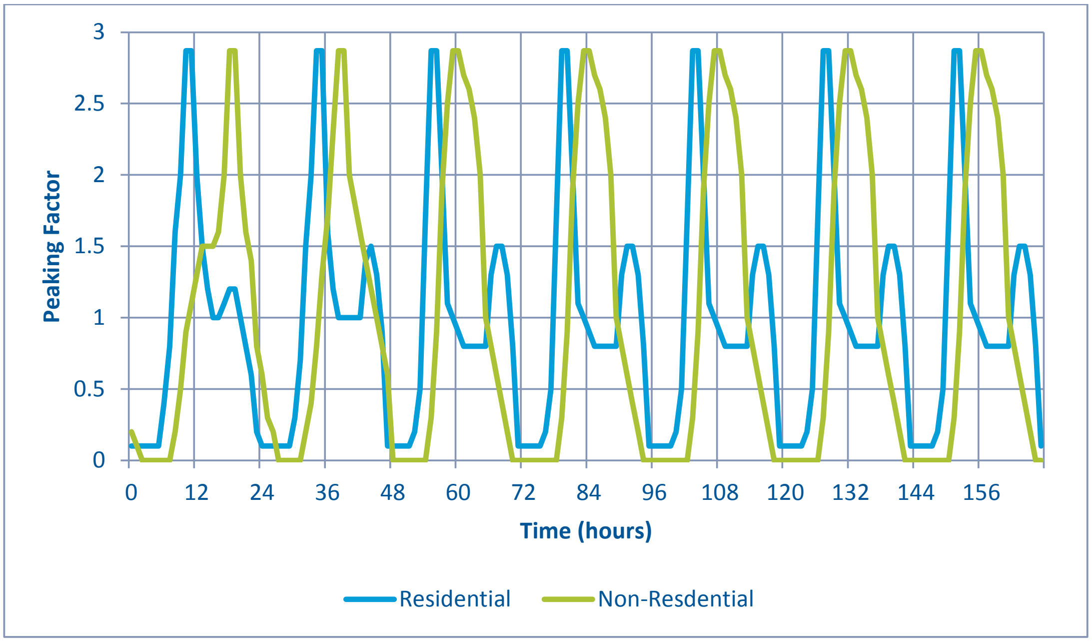

Figure 3: Residential and Non-Residential Patterns

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.

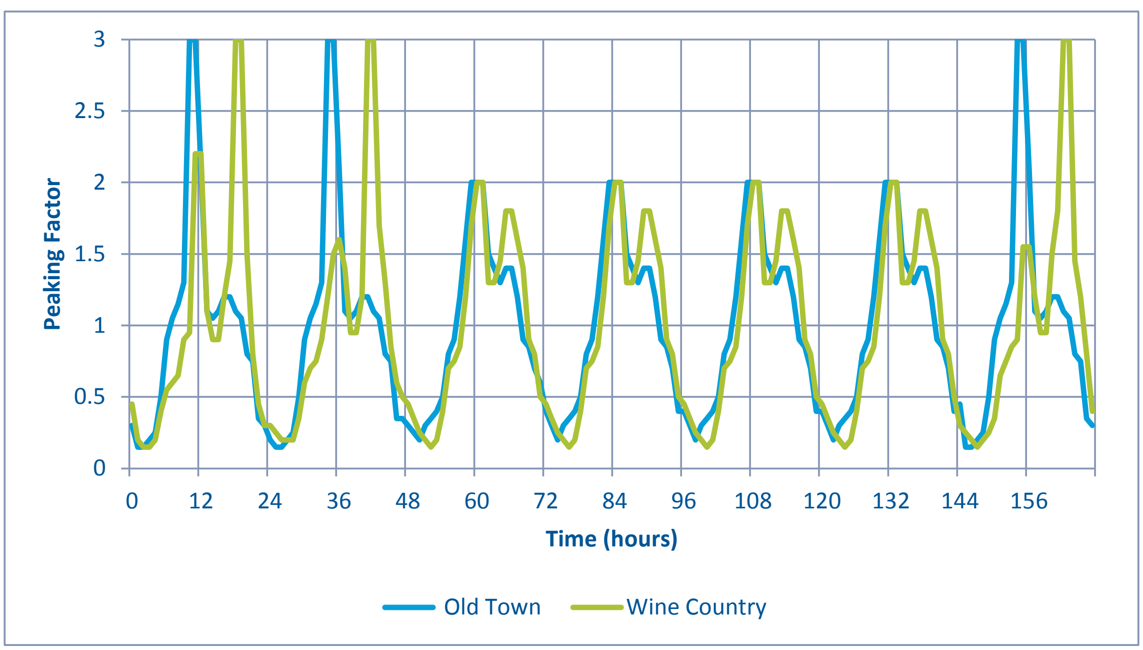

Figure 4: Old Town and Wine Country Patterns

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. While the District utilizes n=0.013 for Capital Improvement Projects, all private development projects shall use n=0.015 to account for long term pipe conditions.

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.

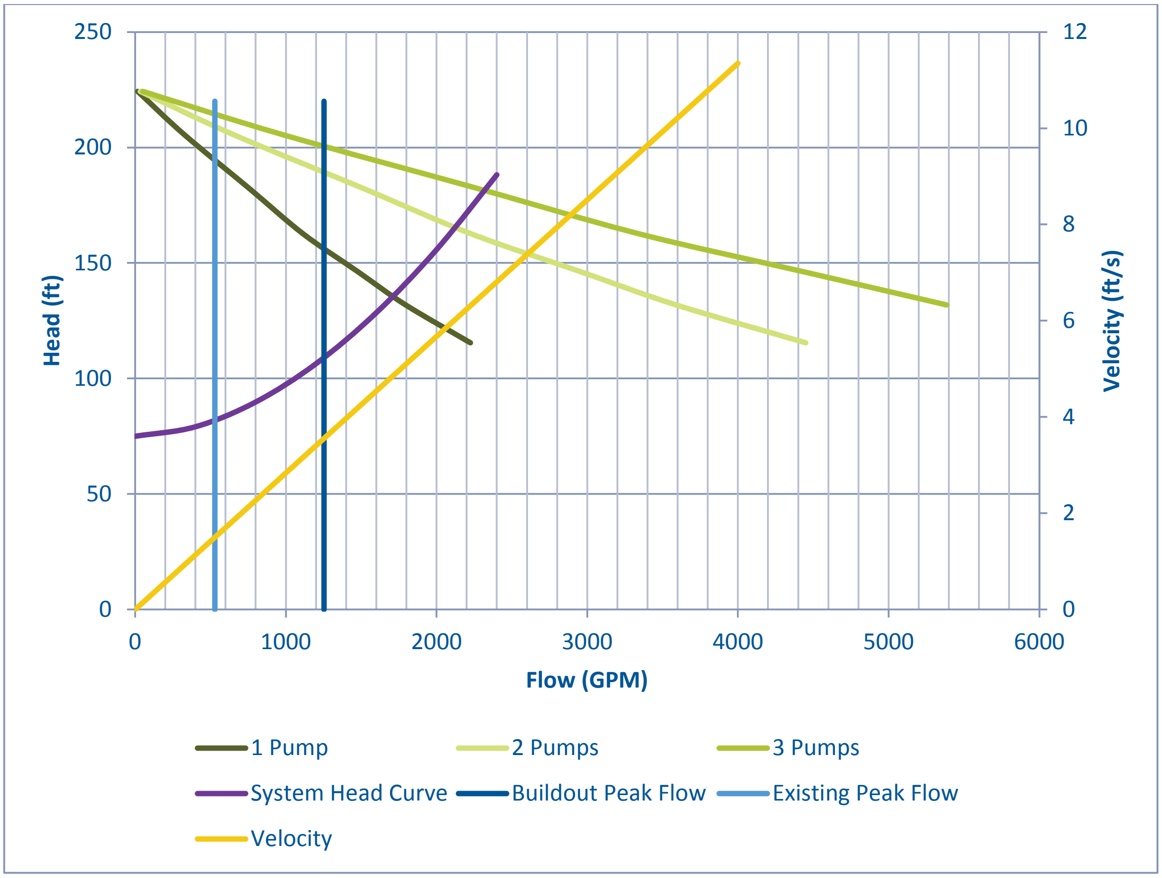

Figure 5: Day Street Lift Station Capacity Assessment

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.