APPENDIX J:

Sewer System Hydraulic Analysis Bedford Marketplace

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Sewer System Hydraulic Analysis
Bedford Marketplace
Parcel Map 37788

Prepared for

Bedford Marketplace, LLC


February 2020

Hunsaker
&
Associates

Hunsaker
& Associates
IRVINE, INC.
Inland Empire Region

PLANNING
ENGINEERING
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GOVERNMENT RELATIONS

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Sewer System Hydraulic Analysis

Date: February 14, 2020

For:
City of Corona Department of Water and Power
755 Public Safety Way
Corona, CA 92880

By:
Paul R. Huddleston, P.E.
Principal
Hunsaker & Associates, Inc.

Project: **Bedford Marketplace, Parcel Map 37788**

Introduction

Hunsaker & Associates Irvine, Inc. (H&A) is pleased to submit the supplemental Sewer System Hydraulic Analysis for Bedford Marketplace. This hydraulic analysis has been prepared to describe the proposed sewer system for the aforementioned retail commercial and hotel project in the City of Corona. The project lies within the jurisdiction of City of Corona Department of Water and Power and their standards have been used for this report. Hydraulic calculations were prepared using FlowMaster software to model peak sewer flows.

The goal of this hydraulic analysis is to evaluate the sewer collection system for the proposed retail commercial and hotel development identifying the general location and size of the pipelines. This evaluation is based on existing and known conditions and should be re-evaluated if these conditions change or new information becomes available. Any interpretation of the information presented in this report should be referred to Hunsaker & Associates to ensure the integrity of the results.

This Sewer System Hydraulic Analysis is tiered off a larger study prepared for the Arantine Hills Specific Plan, a master planned community approved for up to a maximum of 1,806 dwelling units and 80,000 square feet of General Commercial uses. The Applicant, Bedford Marketplace, LLC, proposes to expand the Bedford Marketplace from 80,000 square feet of commercial uses to approximately 134,378 square feet of commercial uses plus a 135-room hotel on approximately 27.8 acres.

Project Location

The overall Arantine Hills project comprises approximately 275.7 acres and is located south of Cajalco Road, West of Interstate 15 and north of Bedford Creek in the City of Corona, County of Riverside, California. More specifically, Bedford Marketplace comprises of 27.8 acres and is located east of Bedford Canyon Road, west of Interstate 15, south of Cajalco Road and north of an existing sewer lift station. The general project location is shown on the exhibit entitled, “Vicinity Map – Figure 1.”

Summary of Findings

1. The Bedford Marketplace commercial center has 2 points of connection to the existing 15-inch main within the sewer lift station access road. The proposed sewer system schematic is shown on the attached exhibit entitled "Proposed Sewer System Model - Figure 3."

2. The total estimated sanitary sewer flows for the proposed Bedford Marketplace commercial center (Parcel Map 37788) were based upon the direction of the "City of Corona Department of Water and Power, Sewer Master Plan" – Section 4.0.

3. The sewer system for the proposed Bedford Marketplace commercial center (Parcel Map 37788) was designed to meet all of the design criteria contained within "City of Corona Department of Water and Power, Design Policy" updated November 2012.

Sewer System Analysis

In order to calculate the pipe sizes and capacity of the proposed sewer system, we have prepared a hydraulic model using FlowMaster 2009 (v8.0) by Haestad Methods. The summary of outputs from the model runs is included in the Appendix of this report. The plan layout of the model is located in the attached exhibit entitled "Proposed Sewer System Model – Figure 3."

Sewer System Design Criteria

Sanitary sewers are designed to carry the estimated wastewater flows during peak periods. Sanitary sewers are also designed to carry these wastes while flowing only part full. The surplus design capacity, or safety factor, is available as a precaution for unusual flow peaking and to permit the sewer to function even with some minor blockage.

Sanitary sewers shall be designed using the following Pipeline Hydraulics Criteria required by the "City of Corona Department of Water and Power":

- Manning's "n" of 0.013
- Minimum velocity = 2 fps or Minimum slope = 0.0100
- Maximum velocity = 8 fps
- Pipes ≤ 10-inch ≡ D/d = 50% (PWWF D/d ≤ 0.82)
- Pipes ≥ 12-inch ≡ D/d = 67% (PWWF D/d ≤ 0.82)

The minimum slope was designated in accordance with Section C.2 'Pipeline Design Criteria' in the attached "City of Corona Department of Water and Power, Design Policy"

- Preferred minimum slope for 8-inch pipes = 0.0040 ft/ft
- Preferred minimum slope for 10-inch pipes = 0.0025 ft/ft
- Preferred minimum slope for 12-inch pipes = 0.0020 ft/ft
- Preferred minimum slope for 15-inch pipes = 0.0012 ft/ft

Estimated Onsite Sewer Flows

The proposed sewer systems for Bedford Marketplace commercial center and the residential neighborhoods within Tract 37644 both connect to an existing 12-inch sewer main within Bedford Canyon Road. The estimated wastewater flow for both developments is based upon the "Unit Flow Factors" – Section 4-3 of "City of Corona Department of Water and Power, Sewer Master Plan", as the "City of Corona Department of Water and Power Design Policy", dated November 2012.

Unit Flow Requirements

For the proposed project, the unit flows for the average day sewer generation were determined from Table 4-2 of the 2005 Sewer Master Plan Report (Included in Appendix).

- Commercial =1,000 gpd/ac
- Hotel =125 gpd/room

The peaking factor for this study is the same factor utilized in the Master Plan and previous analysis, and are as provided in Section 4-4 of the Sewer Master Plan (included in Appendix). The following equations were used to determine the 'Peak Dry Weather Flow' and 'Peak Wet Weather Flow':

- Peak Dry Weather Flow
  $Q_{\text{Peak Dry (cfs)}} = 1.95 \times Q_{\text{Avg (cfs)}}^{0.92}$
  $Q_{\text{Avg Dry (cfs)}}$ = Average Dry Weather Flow

- Peak Wet Weather Flow (the larger of the two following equations govern)
  1) $Q_{\text{PWWF (cfs)}} = 2.6 \times Q_{\text{Avg Dry (cfs)}}$
  2) $Q_{\text{PWWF (cfs)}} = 1.265 \times Q_{\text{Peak Dry (cfs)}}$

Table 1 – Residential Tract 37644 Estimated Sewer Flows

Land Use	Proposed Units	Demand	$Q_{\text{Avg. Dry}}$	$Q_{\text{Peak Dry}} = 1.95 \times Q_{\text{Avg Dry}}$^0.92	$Q_{\text{PWWF}} = 2.6 \times Q_{\text{Avg Dry}}$	$Q_{\text{PWWF}} = 1.265 \times Q_{\text{Peak Dry}}$
Hotel	135 rooms	125 gpd/room	16,875 gpd 0.0261 cfs	44,049 gpd 0.0682 cfs	43,875 gpd 0.0679 cfs	**55,722 gpd 0.0862 cfs**
Commercial	27.8 ac	1,000 gpd/ac	27,800 gpd 0.0430 cfs	69,725 gpd 0.1079 cfs	72,280 gpd 0.1118 cfs	**88,203 gpd 0.1365 cfs**
Sum	---	---	44,685 gpd 0.0691 cfs	113,774 gpd 0.1760 cfs	116,155 gpd 0.1797 cfs	**143,925 gpd 0.2227 cfs**

Sewer System Results

Table 2 – Estimated PWWF Sewer Calculation Summary

Pipe No.	Demand	Sum Demand	Pipe Dia (in)	Slope (ft/ft)	Flow Q_{PWWF} (gpd)	Velocity (ft/s)	Calc'd D/d	Agency Max D/d (PWWF)
1	4.05 ac	4.05 ac	8	0.0100	16,767	1.40	0.104	0.82
2	0.00 ac	4.05 ac	8	0.0100	16,767	1.40	0.104	0.82
3	4.00 ac	8.05 ac	8	0.0100	33,327	1.72	0.134	0.82
4	1.75 ac	9.80 ac	8	0.0100	40,572	1.82	0.149	0.82
5	0.00 ac	9.80 ac	8	0.0100	40,572	1.82	0.149	0.82
6	6.92 ac	6.92 ac	8	0.0100	28,649	1.64	0.134	0.82
7	0.00 ac	6.92 ac	8	0.0100	28,649	1.64	0.134	0.82
8	1.80 ac	8.72 ac	8	0.0100	36,101	1.76	0.149	0.82
9	0.00 ac	8.72 ac	8	0.0100	36,101	1.76	0.149	0.82
10	0.00 ac	18.52 ac	8	0.0100	76,673	2.20	0.210	0.82
11	0.00 ac	18.52 ac	15	0.0020	76,673 (1,338,817 total)	2.56	0.624	0.82
12	6.14 ac 135 rooms	6.14 ac 135 rooms	8	0.0100	81,142	2.24	0.209	0.82
13	0.00 ac	6.14 ac 135 rooms	8	0.0100	81,142	2.24	0.209	0.82
14	0.00 ac	6.14 ac 135 rooms	8	0.0100	81,142	2.24	0.209	0.82
15	0.00 ac	6.14 ac 135 rooms	8	0.0100	81,142	2.24	0.225	0.82
16	0.00 ac	24.66 ac 135 rooms	15	0.0020	157,814 (1,419,958 total)	2.59	0.656	0.82

We sincerely trust these calculations will provide sufficient evidence that the proposed sewer system for Bedford Marketplace commercial center within the Bedford development, Parcel Map 37788, provides sufficient capacity to accommodate the proposed development and meets the City of Corona Department of Water and Power's Sewer Design Guidelines based on the preceding summaries and attached calculations. Please contact me at (951) 509-7055 if you have any questions.

Sincerely,

Brian R. Lowell, PE

REGISTERED PROFESSIONAL ENGINEER
BRIAN ROBERT LOWELL
NO. 74550
CIVIL
STATE OF CALIFORNIA

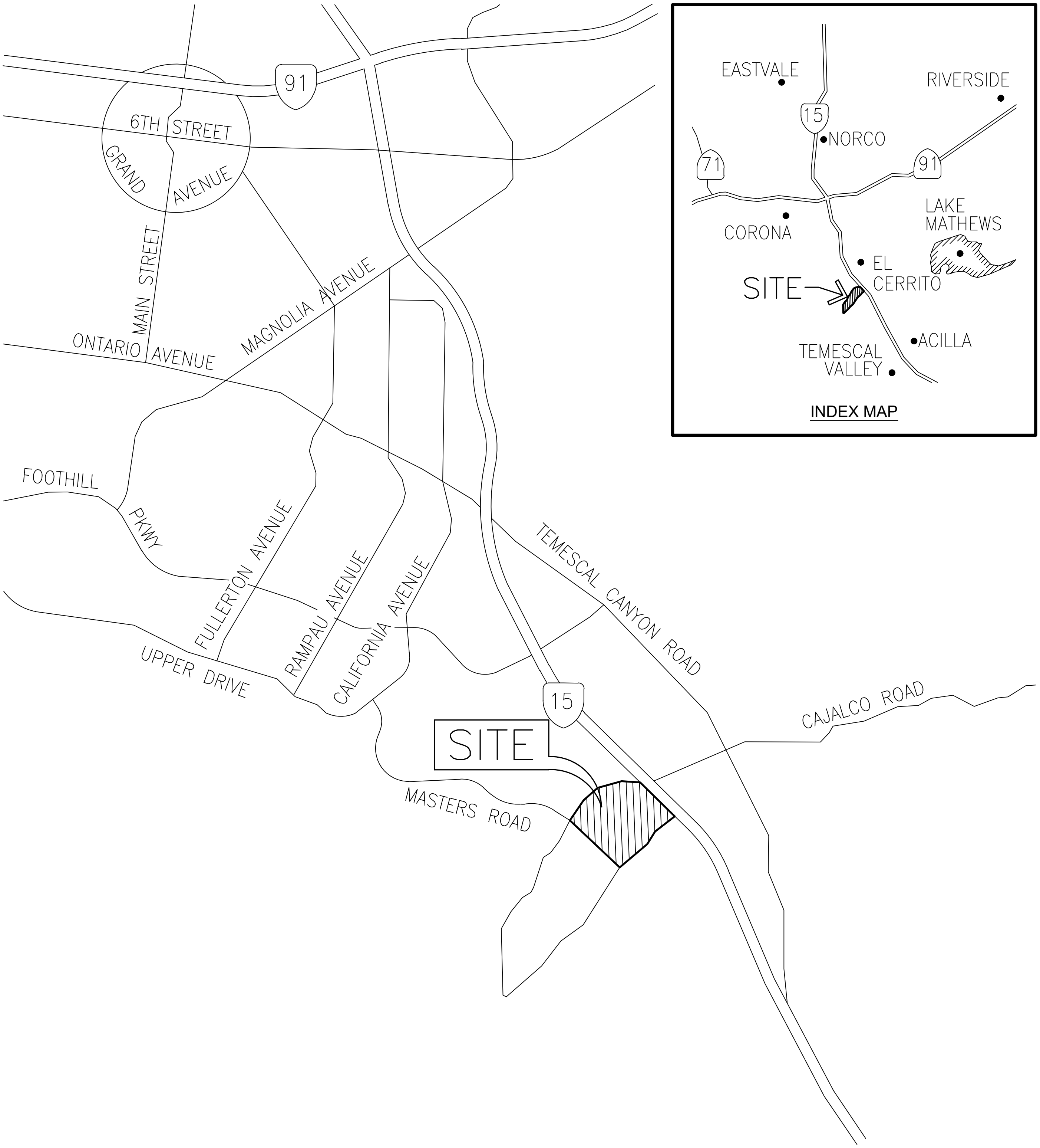

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FIGURE 1: VICINITY MAP

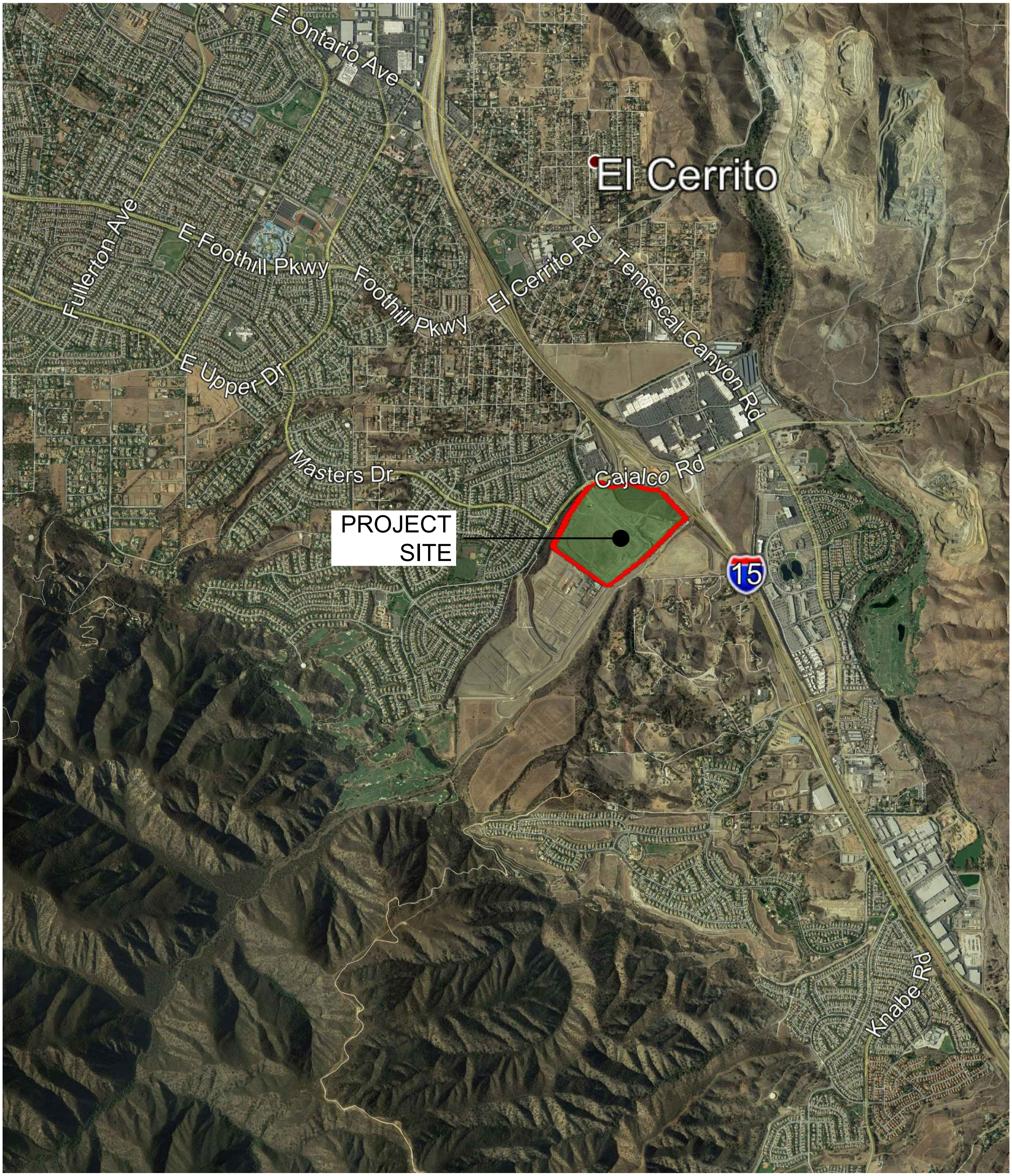

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FIGURE 2: AERIAL PHOTOGRAPH

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LEGEND

- PROPOSED 8-INCH ONSITE GRAVITY SEWER MAIN
- EXISTING 8-INCH OFFSITE GRAVITY SEWER MAIN
- EXISTING 10-INCH OFFSITE GRAVITY SEWER MAIN
- EXISTING 12-INCH OFFSITE GRAVITY SEWER MAIN
- EXISTING 15-INCH OFFSITE GRAVITY SEWER MAIN
- (#) PROPOSED ONSITE SEWER REACH
- MANHOLE LOCATION AND NUMBER

NOTE:
1. SEE APPENDIX FOR FLOWMASTER CALCULATIONS

SEWER DEMANDS

HOTEL Qavg = 125 GPD/ROOM
COMMERCIAL Qavg = 1,000 GPD/AC

$Q_{peak} = 1.95 \times (Q_{avg})^{0.92}$
$Q_{pwwf} = 2.6 \times (Q_{avg})$

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1" = 150'
75 0 150

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CITY OF CORONA DEPARTMENT OF WATER AND POWER
BEDFORD MARKETPLACE
TRACT MAP NUMBER 37788
PROPOSED SEWER SYSTEM MODEL

W.O. No. 4334-2
DATE: Feb 14, 2020

FIGURE 3 OF 3

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PLOTTED BY: BRIANL DATE: Feb. 14, 2020 TIME: 10:14 AM

APPENDIX

SEWER SYSTEM

FLOWMASTER CALCULATIONS

Worksheet for 8-INCH PIPE

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	ft/ft
Normal Depth	0.33	ft
Diameter	0.67	ft

Results

Discharge	0.60	ft³/s
Flow Area	0.17	ft²
Wetted Perimeter	1.05	ft
Hydraulic Radius	0.17	ft
Top Width	0.67	ft
Critical Depth	0.37	ft
Percent Full	50.0	%
Critical Slope	0.00736	ft/ft
Velocity	3.46	ft/s
Velocity Head	0.19	ft
Specific Energy	0.52	ft
Froude Number	1.19	
Maximum Discharge	1.30	ft³/s
Discharge Full	1.21	ft³/s
Slope Full	0.00250	ft/ft
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	50.00	%
Downstream Velocity	Infinity	ft/s

Worksheet for 8-INCH PIPE

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.33	ft
Critical Depth	0.37	ft
Channel Slope	0.01000	ft/ft
Critical Slope	0.00736	ft/ft

Cross Section for 8-INCH PIPE

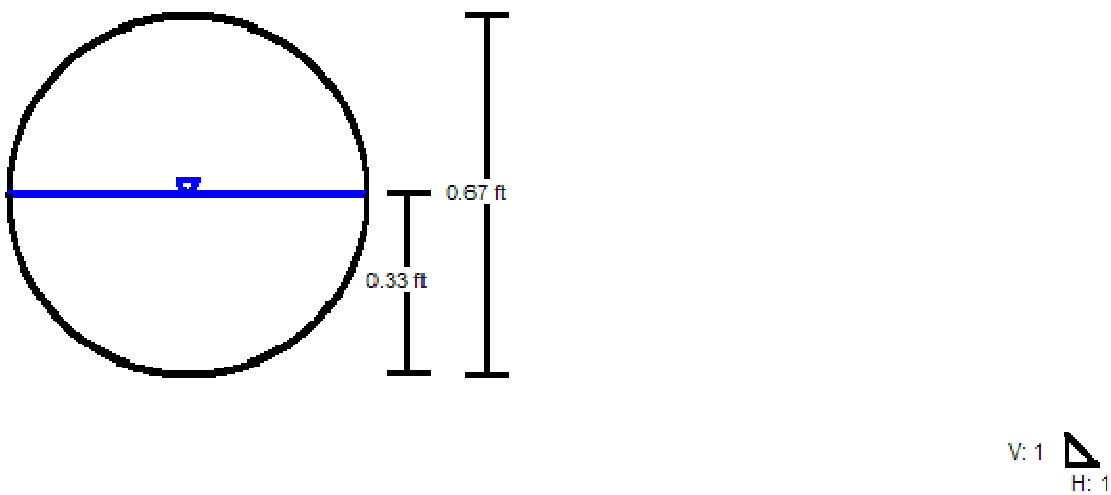
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	ft/ft
Normal Depth	0.33	ft
Diameter	0.67	ft
Discharge	0.60	ft³/s

Cross Section Image



Worksheet for 10-INCH PIPE

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	ft/ft
Normal Depth	0.42	ft
Diameter	0.83	ft

Results

Discharge	1.10	ft³/s
Flow Area	0.27	ft²
Wetted Perimeter	1.31	ft
Hydraulic Radius	0.21	ft
Top Width	0.83	ft
Critical Depth	0.47	ft
Percent Full	50.0	%
Critical Slope	0.00691	ft/ft
Velocity	4.02	ft/s
Velocity Head	0.25	ft
Specific Energy	0.67	ft
Froude Number	1.24	
Maximum Discharge	2.36	ft³/s
Discharge Full	2.19	ft³/s
Slope Full	0.00250	ft/ft
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	50.00	%
Downstream Velocity	Infinity	ft/s

Worksheet for 10-INCH PIPE

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.42	ft
Critical Depth	0.47	ft
Channel Slope	0.01000	ft/ft
Critical Slope	0.00691	ft/ft

Cross Section for 10-INCH PIPE

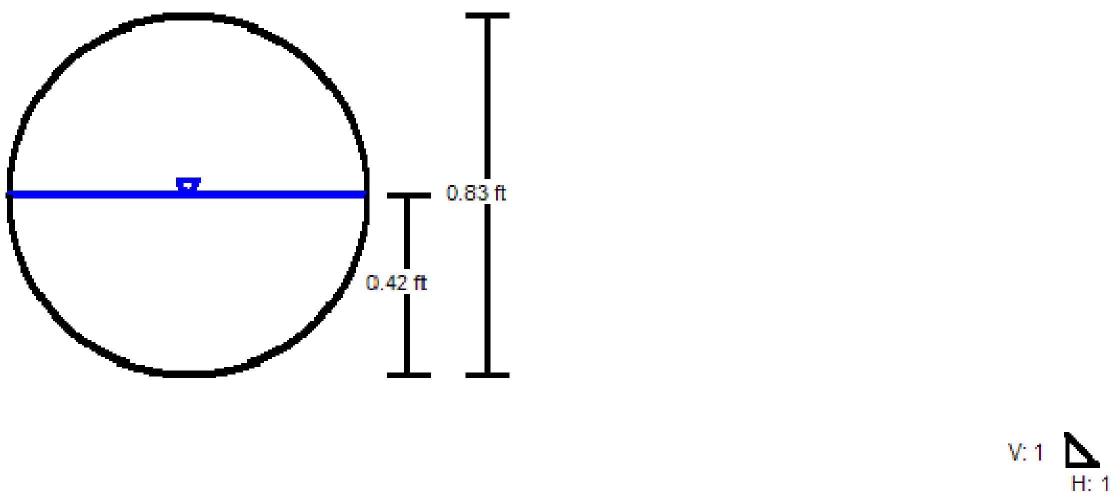
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	ft/ft
Normal Depth	0.42	ft
Diameter	0.83	ft
Discharge	1.10	ft³/s

Cross Section Image



Worksheet for 12-INCH PIPE

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	ft/ft
Normal Depth	0.67	ft
Diameter	1.00	ft

Results

Discharge	2.79	ft³/s
Flow Area	0.56	ft²
Wetted Perimeter	1.91	ft
Hydraulic Radius	0.29	ft
Top Width	0.94	ft
Critical Depth	0.72	ft
Percent Full	66.7	%
Critical Slope	0.00825	ft/ft
Velocity	5.02	ft/s
Velocity Head	0.39	ft
Specific Energy	1.06	ft
Froude Number	1.15	
Maximum Discharge	3.83	ft³/s
Discharge Full	3.56	ft³/s
Slope Full	0.00614	ft/ft
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	66.67	%
Downstream Velocity	Infinity	ft/s

Worksheet for 12-INCH PIPE

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.67	ft
Critical Depth	0.72	ft
Channel Slope	0.01000	ft/ft
Critical Slope	0.00825	ft/ft

Cross Section for 12-INCH PIPE

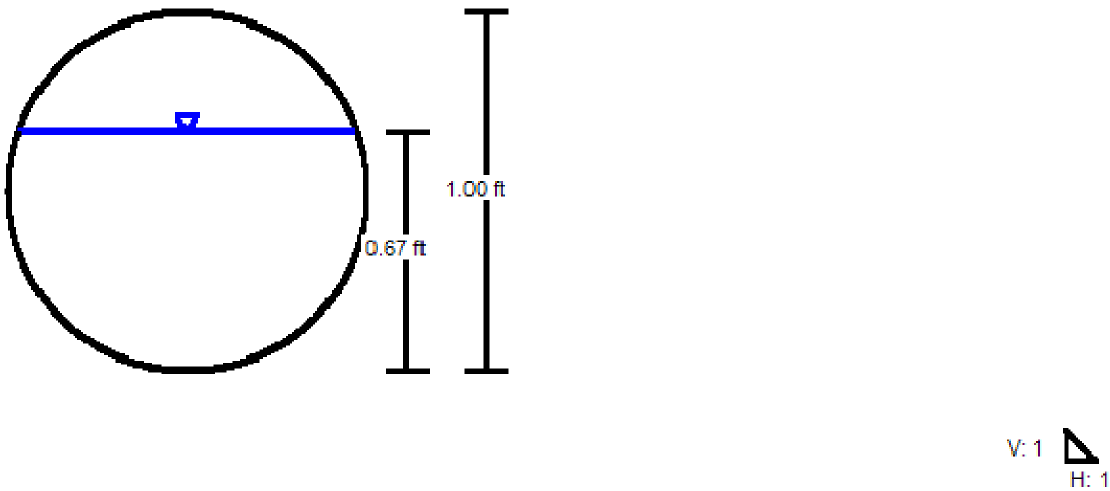
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	ft/ft
Normal Depth	0.67	ft
Diameter	1.00	ft
Discharge	2.79	ft³/s

Cross Section Image



Worksheet for 15-inch pipe

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00200	ft/ft
Normal Depth	0.84	ft
Diameter	1.25	ft

Results

Discharge	2.28	ft^3/s
Flow Area	0.87	ft^2
Wetted Perimeter	2.40	ft
Hydraulic Radius	0.36	ft
Top Width	1.18	ft
Critical Depth	0.60	ft
Percent Full	67.0	%
Critical Slope	0.00562	ft/ft
Velocity	2.61	ft/s
Velocity Head	0.11	ft
Specific Energy	0.94	ft
Froude Number	0.53	
Maximum Discharge	3.11	ft^3/s
Discharge Full	2.89	ft^3/s
Slope Full	0.00125	ft/ft
Flow Type	SubCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	67.00	%
Downstream Velocity	Infinity	ft/s

Worksheet for 15-inch pipe

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.84	ft
Critical Depth	0.60	ft
Channel Slope	0.00200	ft/ft
Critical Slope	0.00562	ft/ft

Cross Section for 15-inch pipe

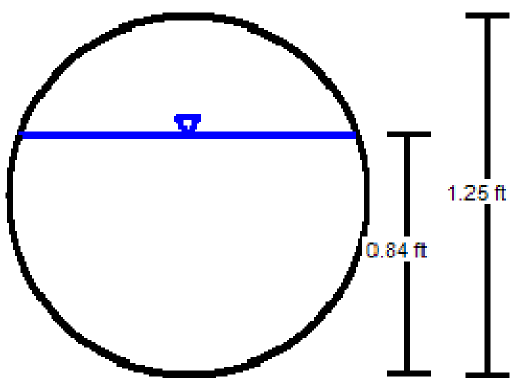
Project Description

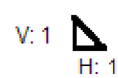
Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00200	ft/ft
Normal Depth	0.84	ft
Diameter	1.25	ft
Discharge	2.28	ft³/s

Cross Section Image





LOS ANGELES COUNTY
SEWER GENERATION RATES

TABLE 1

LOADINGS FOR EACH CLASS OF LAND USE

DESCRIPTION	UNIT OF MEASURE	FLOW (Gallons Per Day)	COD (Pounds Per Day)	SUSPENDED SOLIDS (Pounds Per Day)
RESIDENTIAL				
Single Family Home	Parcel	260	1.22	0.59
Duplex	Parcel	312	1.46	0.70
Triplex	Parcel	468	2.19	1.05
Fourplex	Parcel	624	2.92	1.40
Condominiums	Parcel	195	0.92	0.44
Single Family Home (reduced rate)	Parcel	156	0.73	0.35
Five Units or More	No. of Dwlg. Units	156	0.73	0.35
Mobile Home Parks	No. of Spaces	156	0.73	0.35
COMMERCIAL				
Hotel/Motel/Rooming House	Room	125	0.54	0.28
Store	1000 ft^2	100	0.43	0.23
Supermarket	1000 ft^2	150	2.00	1.00
Shopping Center	1000 ft^2	325	3.00	1.17
Regional Mall	1000 ft^2	150	2.10	0.77
Office Building	1000 ft^2	200	0.86	0.45
Professional Building	1000 ft^2	300	1.29	0.68
Restaurant	1000 ft^2	1,000	16.68	5.00
Indoor Theatre	1000 ft^2	125	0.54	0.28
Car Wash				
Tunnel - No Recycling	1000 ft^2	3,700	15.86	8.33
Tunnel - Recycling	1000 ft^2	2,700	11.74	6.16
Wand	1000 ft^2	700	3.00	1.58
Financial Institution	1000 ft^2	100	0.43	0.23
Service Shop	1000 ft^2	100	0.43	0.23
Animal Kennels	1000 ft^2	100	0.43	0.23
Service Station	1000 ft^2	100	0.43	0.23
Auto Sales/Repair	1000 ft^2	100	0.43	0.23
Wholesale Outlet	1000 ft^2	100	0.43	0.23
Nursery/Greenhouse	1000 ft^2	25	0.11	0.06
Manufacturing	1000 ft^2	200	1.86	0.70
Dry Manufacturing	1000 ft^2	25	0.23	0.09
Lumber Yard	1000 ft^2	25	0.23	0.09
Warehousing	1000 ft^2	25	0.23	0.09
Open Storage	1000 ft^2	25	0.23	0.09
Drive-in Theatre	1000 ft^2	20	0.09	0.05

TABLE 1
(continued)

LOADINGS FOR EACH CLASS OF LAND USE

DESCRIPTION	UNIT OF MEASURE	FLOW (Gallons Per Day)	COD (Pounds Per Day)	SUSPENDED SOLIDS (Pounds Per Day)
COMMERCIAL				
Night Club	1000 ft^2	350	1.50	0.79
Bowling/Skating	1000 ft^2	150	1.76	0.55
Club	1000 ft^2	125	0.54	0.27
Auditorium, Amusement	1000 ft^2	350	1.50	0.79
Golf Course, Camp, and Park (Structures and Improvements	1000 ft^2	100	0.43	0.23
Recreational Vehicle Park	No. of Spaces	55	0.34	0.14
Convalescent Home	Bed	125	0.54	0.28
Laundry	1000 ft^2	3,825	16.40	8.61
Mortuary/Cemetery	1000 ft^2	100	1.33	0.67
Health Spa, Gymnasium				
With Showers	1000 ft^2	600	2.58	1.35
Without Showers	1000 ft^2	300	1.29	0.68
Convention Center, Fairground, Racetrack, Sports Stadium/Arena	Average Daily Attendance	10	0.04	0.02
INSTITUTIONAL				
College/University	Student	20	0.09	0.05
Private School	1000 ft^2	200	0.86	0.45
Church	1000 ft^2	50	0.21	0.11

City of Corona
Department of Water and Power

Design Policy
November 2012

CITY OF CORONA

DEPARTMENT OF WATER AND POWER DESIGN POLICY

NOVEMBER 2012

Tom Koper

Tom Koper, PE
District Engineer
R.C.E. No. 50258

11-28-2012

Date

C. WATER RECLAMATION (SEWER) COLLECTION SYSTEM

1. GENERAL

The following sections provide criteria to be used in the design of water reclamation collection (sewer) systems. The developer and his engineer shall be responsible to ensure all design work is in conformance with this Design Policy, the City of Corona Standard Plans and Specifications for sewer systems, State of California Department of Health Services Criteria for the Separation of Water Mains and Sanitary Sewers, as shown on City Standard Drawings 419, and generally accepted standards of good engineering practice. Two sets of calculations are required with first plan check.

2. PIPELINE DESIGN CRITERIA

a. Velocity - Design Engineer shall calculate the velocity of flow under proposed conditions. Velocity shall not be less than 2 fps a minimum of once per day to provide sufficient scouring action for self-cleaning. Maximum velocity shall not be greater than 8 fps at design flow. Where 2 fps velocity cannot be provided at least once per day, the slope shall be at least 0.01 ft/ft unless approved in writing by DWP.

b. Use a Manning's "n" of 0. 013 for sewer design.

c. Maximum allowable depth of flow (d/D) at peak flows, is as follows:

   1) 10-inch and smaller 50 percent

   2) 12-inch and larger 67 percent

d. For more information refer to the engineering report in the Sewer Master Plan for Trunk Line Sewers.

e. Minimum slope(s) shall be:

   S = 0.0040 for 8-inch sewer

   S = 0.0025 for 10-inch sewer

   S = 0.0020 for 12-inch sewer

   S = 0.0012 for 15-inch sewer

   S = 0.0010 for 18-inch sewer

   S = 0.0008 for 21-inch sewer

   S = 0.0007 for 24-inch sewer

   Design slopes conforming to the minimum and maximum velocity criteria described in section 2(a) above.

   The design engineer shall submit calculation including the slopes for each segment of the pipeline. Sewer slope shall not be changed in the field unless approved by the DWP District Engineer as it could negatively impact future capacity.

f. Manholes are required at grade breaks.

g. Provide 0.10-foot drop of invert elevation through manholes. Provide 0.20-foot drop of invert elevation at right angle alignment or bends.

h. Minimum 12-inch vertical separation is required when a sewer line crosses water/reclaimed water lines. Encase the sewer line in concrete in accordance with the City of Corona Standard Detail 419.

i. If separation with a parallel water main is less than 10 feet clear (edge-to-edge), design must meet requirements of City Standard Drawings 419 and be reviewed and approved by CDPH or DWP.

j. A minimum of 1-foot vertical separation is required between potable waterlines and both non-potable waterlines and sewer lines. The potable waterline must be above the non-potable waterlines and sewer lines. Separation criteria must follow current CDPH requirements.

k. The minimum unit flow factors used shall be per the following table or as revised in subsequent Sewer Master Plan Updates:

Land Use		*Existing Unit Flow Factor (gpd/ac)	*Ultimate Unit Flow Factor (gpd/ac)	Residential Flow Factors (gpd/du)
RR1	Rural Residential 1 (0.2 to 0.5 du/ac)	150	150	300
RR2	Rural Residential 2 (1 du/ac)	300	300	300
E	Residential Estate (1-3 du/ac)	500	500	300
LDR	Residential Low Density (3-6 du/ac)	1,000	1,000	270
LMDR	Low Medium Density (6-8 du/ac)	1,200	1,200	270
MDR	Medium Density (6-15 du/ac)	1,700	1,700	240
HDR	High Density (15-36 du/ac)	2,000	2,000	200
CBD	Commercial Business District	1,000	1,050	-
C or GCC	General Community Commercial	1,000	1,050	-
CP or OP	Office Professional	1,200	1,260	-
GI	General Industrial	1,100	1,155	-
LI	Light Industrial	800	840	-
I or School	Institutional	800	800	-
OS-R or OS-P	Open Space Recreational	130	130	-
QP or MU	Quasi-Public / Mixed Use	700	700	-

**Unit flow factors based on gross acres*

3. HORIZONTAL PIPE ALIGNMENT CRITERIA

a. <u>Main Lines</u>

1) Line to be 5.0 feet off centerline of street, and located on the opposite side as the potable water main.

2) Minimum ten (10) feet clear separation from potable and non-potable water mains.

3) Minimum three (3) feet separation with other utilities.

4) Curvilinear alignment will not be permitted except when approved by DWP.

5) Manholes are required at all change of directions, both horizontal and vertical, and at 350-foot maximum spacing.

6) Clean-outs shall not be permitted except when approved by DWP.

7) Private sewer mains shall have a manhole spacing every 350 feet.

b. Sewer Laterals

1) Must clear driveway and be provided with clean-out.

2) Ten (10) feet clear separation with waterlines and water services unless otherwise approved by DWP.

4. VERTICAL PIPE ALIGNMENT CRITERIA

a. Main Lines

1) Minimum depth shall be 7.0 feet of cover over the top of the pipe, except terminal reaches may be reduced to 6.0 feet of cover, where the line will not be extended in the future.

2) Vertical curve may be permitted – prior written approval from DWP is required.

b. Sewer Laterals

1) Minimum 5.0-foot cover at property line, minimum slope S = 0.02 with a saddle unless otherwise approved.

2) Laterals shall be located below the water main with a minimum clearance of 12 inches. Where clearances are critical lateral profiles are to be detailed on the plans.

3) Laterals shall not enter a manhole.

4) A backwater sewer valve shall be installed in an inspection vault, in conformance with the plumbing code, for all buildings where the rim elevation of the upstream manhole on the sewer main is above the pad elevation of the structure.

5. SANITARY SEWER MANHOLES

a. Pre-cast concrete manholes shall be constructed and lined with polyurethane (Sancon 100™ or equal) per City Standard Drawing 302.

b. Manholes shall be 5-foot in diameter with 3-foot frame and cover for depths up to 12 feet from finish grade to sewer invert. Manholes deeper than 12 feet shall be 6-foot diameter.

c. On the center of each manhole cover, lettering shall be cast to read "CITY OF CORONA - SEWER." This lettering shall be cast into the lid.

d. Manhole rim elevations shall be lower than all pad elevations immediately downstream. If this condition cannot be met, then back water valves must be installed in accordance with the Uniform Plumbing Code, Section 710.1. A letter with the tract number and affected lots shall be prepared by the Design Engineer and submitted to the contractor/developer/owner with a copy to the Building Inspector, the Building Department, the Department of Public Works and the Department of Water and Power. Identify these lots on the grading and plumbing plans.

e. Manholes located in undeveloped land shall be surrounded by a 10-foot by 10-foot by 6-inch concrete pad reinforced with 6x6 10/10 WWF (minimum), marked with a 4-inch pipe embedded in concrete 2 feet below existing ground and 4 feet above existing ground. Manholes located in improved areas shall be located by chipping 1½-inch lettering in the curb face "MH" with dimensions to the manhole from a minimum of 2 ties points.

f. No stub-outs allowed on manholes, except where future connections are anticipated.

6. DROP MANHOLES

Drop manholes are not allowed except when specifically approved by the Department of Water and Power General Manager. If approved, the design shall be in accordance with the City of Corona Standard Detail 303.

7. PIPE MATERIAL AND SIZE

a. Main Lines

1) Main line minimum size shall be 8 inches.

2) Use SDR 26 PVC green pipe for all residential developments.

3) Use extra-strength Vitrified Clay Pipe (VCP) for all commercial and industrial developments.

4) Use ductile iron pipe with Protecto 401 ™ lining and two sheets of 8-mil polyethylene encasement, color green, where construction constraints, such as clearances with waterlines or excessive loading condition (depth of cover greater than 15 feet), warrant their use.

5) Construct all pipelines of the same material between manholes.

b. Laterals

1) Lateral minimum size for residences shall be 4 inches. Lateral minimum size for commercial and industrial units shall be 6 inches.

2) Use green PVC pipe, SDR 21, CERTA LOK ™ restrained joints for residential laterals. Call out pipe material, wall thickness, and joint style on the plans.

3) Use extra-strength vitrified clay pipe for all commercial and industrial laterals.

4) Laterals shall not be connected/manifolded together. Every individual building shall be connected to a public sewer line by a single sewer lateral. Multiple buildings on same property must have separate laterals connecting to the public sewer.

5) Use ductile iron pipe where separation criteria with water mains (i.e., clearances) cannot be met per CDPH separation requirements.

8. CORROSIVE SOIL

a. All pipeline designs shall require a geotechnical engineer to determine the existing soil corrosivity and the design engineer to recommend the appropriate cathodic protection facilities. The engineer shall specify on the plans and in the specifications the applicable corrosion control facilities.

b. All ductile iron pipes shall have at a minimum two 8-mil polyethylene protective sleeves color coded to match the contents of the pipe. Clearly denote this on each plan sheet.

c. All ductile iron pipelines crossing railroad facilities, large natural gas pipelines, or electrical facilities having impressed currents shall be provided with cathodic protection facilities.

d. All ductile iron transmission lines shall have cathodic test stations per DWP Standard Details 450 through 458. Clearly denote this on each plan sheet.

9. EASEMENTS

Where public sanitary sewers cannot be located in public roads, they shall be constructed in a prescribed easement to meet the following conditions:

a. A separate instrument is required if the easement is not shown on the Tract or Parcel Map. The instrument includes a legal description and exhibit 8½" x 11" showing the location of said easement. Private sewers are not allowed to cross adjacent/adjoining private parcels/land without special approval from DWP.

b. The easement shall be a minimum of 20 feet wide. Easements shall contain an all-weather access road in order to maintain the entire sewer system (pipeline, manholes, lift station, etc.) within the easement.

c. Easements for deep sewers shall be wide enough for sewer maintenance. The minimum width of easement shall be wide enough to accommodate a maximum of 1:1 side slopes, with a 5-foot wide by 10-foot maximum depth trench shield, and an additional 10-foot wide truck access road.

d. All legal easement descriptions and exhibits shall be prepared and stamped by a Registered Civil Engineer licensed in the State of California or Professional Land Surveyor licensed in the State of California and filed with the County Recorder if not shown on a tract or parcel map. Professional licenses must be current at the time of plan preparation.

10. LIFT STATION DESIGN CRITERIA

Avoid the use of lift stations wherever possible due to the associated cost and maintenance required. Design Engineer shall utilize the following minimum sewer lift station design guidelines.

a. Design the wet well with sufficient capacity to prevent short cycles whereby the pumps frequently start and stop, yet small enough that it will regularly evacuate sewage from the wet well to prevent the wastewater from becoming septic. The desired number of pump cycles should be limited to no more than 6 per hour for motors up to 10 horsepower. Motors up to 75 horsepower should start no more than 4 times per hour. Larger motors should cycle less frequently. Lift stations should have sufficient volume to store sewage in the event of mechanical or electrical failures, until the City can respond to the failure and prevent overflows.

b. Size the pumps to efficiently handle the peak wet weather flows. Provide a minimum of two pumps sized at the peak wet weather flow to the station, so that sufficient standby capacity is available when one pump is removed for repairs or experiences a mechanical failure. Select pumps capable of passing a minimum solid size of 3-inches without clogging. Select pumps with shafts, seals and impellers constructed of wear resistant material to provide long life. Provide Tungsten Carbide seals, Ni-Hard impellers, and 316 stainless steel pump shafts. For services where aggressive agents may be found in the sewage, such as at golf courses, select pumps with complete stainless steel construction, including the pump bowl, shaft, impeller, and motor housing.

c. Dry well lift stations must be properly ventilated and provide unobstructed access to all equipment. A minimum 3-foot clearance from all obstructions should be provided. Greater clearances may be required for equipment with special maintenance needs. Provisions for equipment removal including hatches, large door openings, and hoists shall also be provided.

d. Install discharge piping, valves, and equipment at submersible pump installations above grade.

11. FORCEMAINS

Force main Systems shall be of adequate size to efficiently transmit the total ultimate peak operational flows supplied by the connected wastewater pumping station(s), to the discharge point. Coordinate capacity computations with the proposed pumping system(s), along with any future flow requirement, if applicable. In order to provide adequate pipeline cleansing, force main flow velocity shall not be less than four (4) feet per second at the minimum pumping capacity nor exceed six (6) feet per second at ultimate design pumping capacity. The force main diameter, material, coating, pressure class/thickness class, etc. shall be as required by the City Standard Details, City Design Policy, and the design engineer. Provide signed and sealed calculations to substantiate the force main diameter. Design a redundant force main or a gravity bypass line if physically possible.

Other features, provisions, and operational procedures for these systems include:

a. Auto flushing connection with an approved backflow preventer and air gap.

b. The force main shall be flushed once per month with a minimum of 1½ force main volumes.

c. Emergency shutdown provisions to mitigate sewer spills/overflows.

d. Contractor/Engineer/Private Owner shall submit an operations and maintenance procedure complete with calculations.

e. Pipe shall be Ductile Iron Pipe with Protecto 401 ™ epoxy lining and wrapped with green-colored polyethylene encasement. 4-inch minimum pipe size.

f. Provide combination sewage and air vacuum valve assemblies at high points.

12. INVERTED SIPHONS

Avoid inverted sewer siphons wherever possible. If required, utilize the following minimum design guidelines.

a. Locate the inverted siphon completely within a public right-of-way. If a location within public right-of-way is unavailable, an easement or other right-of-entry is required subject to approval by the Department of Water and Power.

b. The inverted siphon shall not impact other facilities or be impacted by other facilities. Maintain adequate clearances from other facilities.

c. Provide a minimum of two barrels. Always provide one redundant barrel for by-pass capacity, emergencies, and maintenance purposes.

d. Minimum pipe size of 6-inch and minimum velocity of 3 ft/sec shall be considered for design of each barrel.

e. Minimize bends and angle points.

f. Provide cleanouts where the length of inverted siphon exceeds 400-ft. The size of the cleanout shall be adequate to handle the debris that may accumulate, and at least as large as the size of the inverted siphon.

13. PIPE TRENCH BACKFILL

a) All pipeline trench backfill within paved areas shall be per Public Works Standard 150.

b) All pipeline trench backfill within landscaped parkway areas shall be per Public Works Standard 149

c) All pipeline bedding shall be per Department of Water and Power Standard 308.

D. <u>LOW PRESSURE SEWER SYSTEMS</u>

1. GENERAL PROVISIONS

a. Submit plans and specifications for low-pressure sewer systems to the Department of Water and Power for review and approval. Secure a permit for each low-pressure sewer installation. Approval of low-pressure wastewater systems as an alternative to conventional wastewater systems shall be in accordance with the conditions listed in Subsection 2 unless other special circumstances justifying their use are affirmatively demonstrated.

b. <u>It is not</u> the intent of this design policy to utilize low-pressure systems as a replacement for conventional gravity sewer systems. However, as a means to provide service to an individual lot or a small group of lots or buildings where conventional gravity service cannot be utilized within reason, the Department of Water and Power may consider the use of a low-pressure system; providing the Design Engineer can show reasonable justification for its use.

2. CONDITIONS OF APPROVAL

a. The use of low-pressure sewer systems will be considered where:

1) build-out has left small parcels of property in precarious locations in relation to the lay of the land,
2) shallow bedrock conditions would require extensive rock removal,
3) unstable soil conditions prohibit construction of deep sewers,
4) temporary use would provide a cost effective alternative until gravity system construction is completed,
5) the proposed sewer is located a considerable distance from existing gravity sewers, or
6) the use of a low-pressure sewer system will eliminate the need for small public lift stations.

b. The applicant is responsible to evaluate all potential alternative wastewater collection systems and justify the selection of the low-pressure sewer system based on engineering and surrounding conditions.

c. Based on the information furnished by the Developer and the Design Engineer, the Department of Water and Power will decide the acceptability, scope and extent of the low pressure sewer system to be permitted.

City of Corona
Department of Water and Power

Sewer Master Plan
Section 4

Section 4

CRITERIA

4-1 General

Establishing performance standards is an important part of developing new wastewater collection systems, and evaluating existing systems, as it forms the basis for system analysis and system improvement recommendations. These standards include methodology for estimating wastewater design flows and minimum design standards for the collection system pipes, lift stations, and force mains.

Average wastewater flows can be reasonably estimated from land use and their corresponding unit flow factors. The results are then compared to measured flows. Peaking relationships are needed for estimating peak dry weather and peak wet weather flows. Peak wet weather flows also include an allowance for inflow / infiltration (I/I).

Collection system design standards include minimum pipe size, minimum flow velocity, and depth of flow to pipe diameter ratio. Lift station criteria includes the capacity and number of pumps, wet well and force main sizes, redundancy, emergency power, remote monitoring capabilities, as well as safety and regulatory agency requirements. Finally, facility useful lives are needed for adequately scheduling replacement of the aging infrastructure.

4-2 Flow Monitoring

Flow monitoring is essential in developing unit flow factors, calibrating the system model, and estimating the average and peak flows.

A temporary flow monitoring program was developed, and field flow monitoring was conducted by ADS Environmental Services. The study was conducted over a period of seven (7) days at eleven (11) locations. The selected flow monitoring locations and a summary of the results are shown on Figure 4-1 and in Table 4-1. The flow monitors were in place from February 13, 2004 through February 19, 2004.

Table 4-1
Flow Monitoring Results

Site	Model MH ID	City Atlas		Pipe Size (in)	Location	Measured Flow (cfs)		
		Sheet	MH ID			Minimum	Average	Maximum
1	10-06100	F16	1381	18	2703 Wardlow Rd	0.227	1.408	2.816
2	10-07870	G15	1573	21	2380 Railroad St (WWTP 1)	0.415	1.471	2.618
3	14-11120	H14	1840	42	2047 Railroad St (WWTP 1)	2.438	9.602	15.796
4	16-31100	J15	3199	42	Railroad St, west of N. Lincoln Av	2.245	7.741	13.813
5	16-30490	J20	3538	10	Buenta Vista Av & Ontario Av	0.076	0.393	0.789
6	22-57550	M22	8210	10	Fullerton Av & Ontario Av	0.110	0.470	0.930
7	20-58480	M15	4963	12	Parkridge Av north of Mesa Dr	0.169	0.541	0.941
8a	20-55020	M16	4982	15	300 Reed Cir (WWTP 2)	0.000	0.806	1.603
8b	20-55900	M16	4991	18	300 Reed Cir (WWTP 2)	0.605	2.288	4.123
9	20-53950	M16	5004	30	Flood Control Channel (WWTP 2)	0.636	2.596	4.766
11	14-17580	I16	2489	10	Smith Av north of Pleasant View Av	0.365	1.351	2.327

Insert Figure 4-1 (Flow Monitoring Sites)

The flow monitoring sites were strategically chosen to aid in the development of unit flow factors. Because single family residential land uses make up nearly 30 percent of the total area and generate about 70 percent of the total wastewater, it was very important to monitor several different areas with primarily residential land uses. Flow monitoring Sites 1, 5, 6, 7, and 11 were selected specifically because the existing tributary areas were dominated by single family residential uses.

Flow monitoring Site 2 was selected to aid in the development of the industrial unit flow factor. Sites 3 and 4 were selected to gauge minimum, average, and peak flows into Wastewater Treatment Plant 1 for model calibration purposes. Similarly, Sites 8A, 8B and 9 gauged the flows into Wastewater Treatment Plant 2.

4-3 Unit Flow Factors

Unit flow factors utilized in this study were developed based upon the land use data obtained from the City's existing GIS and results of the temporary flow monitoring study. Atlas maps, aerial photographs and field reviews supplemented this information.

The average daily flow recorded at each flow monitoring site and gross acreage was utilized in determining calibrated unit flow factors for each land use. The calibrated flow factors were then increased to account for vacancy rates and any inconsistencies in the flow monitoring data. The resultant flow factors represent the existing conditions. The current residential vacancy rate is 3.65 percent. The current commercial / industrial vacancy rate is approximately 6 to 7 percent. The ultimate unit flow factors are not expected to increase dramatically, because densification is not expected. The industrial and commercial flow factors were increased by 5 percent to account for any changes from the existing types of uses that may generate more wastewater.

The unit flow factors developed for this study, representing average dry weather flows, are shown in Table 4-2. The existing and ultimate unit flow factors are given in gallons per day per gross acre. Residential flow factors are also shown in gallons per day per dwelling unit. These factors are based on the fact that the existing right-of-way and railroads account for about 22 percent of the total existing developed lands (see Table 3-1) and an estimated target density for each land use category. The average density specified per the General Plan was used for residential estate, residential low density, and residential medium density lands. For the categories of residential low medium and residential high, 6 du/net acre and 15 du/net acre were implemented, respectively. This resulted in a more conservative unit flow factor but comparable in relation to the factors estimated for the other residential areas.

The residential flow factors in gallons per day per dwelling unit, also shown in Table 4-2, can be utilized by the City to make decisions regarding the revenue and rates, such as the distribution of connection fee costs associated with the improvement and expansion of wastewater treatment plants.

Table 4-2
Unit Flow Factors

Land Use		*Existing Unit Flow Factor (gpd/ac)	*Ultimate Unit Flow Factor (gpd/ac)	Residential Flow Factors (gpd/du)
RR1	Rural Residential 1 (0.2 to 0.5 du/ac)	150	150	300
RR2	Rural Residential 2 (1 du/ac)	300	300	300
E	Residential Estate (1-3 du/ac)	500	500	300
LDR	Residential Low Density (3-6 du/ac)	1,000	1,000	270
LMDR	Low Medium Density (6-8 du/ac)	1,200	1,200	270
MDR	Medium Density (6-15 du/ac)	1,700	1,700	240
HDR	High Density (15-36 du/ac)	2,000	2,000	200
CBD	Commercial Business District	1,000	1,050	-
C or GCC	General Community Commercial	1,000	1,050	-
CP or OP	Office Professional	1,200	1,260	-
GI	General Industrial	1,100	1,155	-
LI	Light Industrial	800	840	-
I or School	Institutional	800	800	-
OS-R or OS-P	Open Space Recreational	130	130	-
QP or MU	Quasi-Public / Mixed Use	700	700	-

**Unit flow factors based on gross acres*

4-4 Peaking Factors

The wastewater unit flow factors discussed in Sub-section 4-3 are used to generate average dry weather flows (ADWF) entering the collection system. However, the adequacy of a sewage collection system is based upon its ability to convey the peak flows. At any individual point in the system, peak dry weather flow (PDWF) is estimated by converting the total average flow upstream of the point in question to peak dry weather flow by an empirical peak-to-average relationship.

The peaking formula commonly used in sewerage studies is of the following form:

$Q_{peak} = a\, Q_{ave}{}^{b}$

where Q_{peak} = Peak Dry Weather Flow

Q_{ave} = Average Dry Weather Flow

a, b = Peaking Formula Coefficients

The temporary flow monitoring data was reviewed to develop peaking relationships at each site. As expected, these relationships varied from site to site depending upon the makeup and size of the tributary land use. Several diurnal curves were developed for each of the (11) flow monitoring sites. Average hourly flows for each hour of the day at each of the sites were used to produce diurnal curves for each day of the week as well as composite diurnal curves for the weekend and weekday. Figures 4-2, 4-3 and 4-4 illustrate the citywide flow monitoring site and composite weekday, weekend, and overall diurnal curves. These figures all indicate the expected peaks and epitomize a typical diurnal curve. The highest peak occurs during the morning hours with another peak occurring in the evening. While peaking factors at the different sites ranged from 1.5 to 2.2 the overall composite factors were between 1.8 and 1.9.

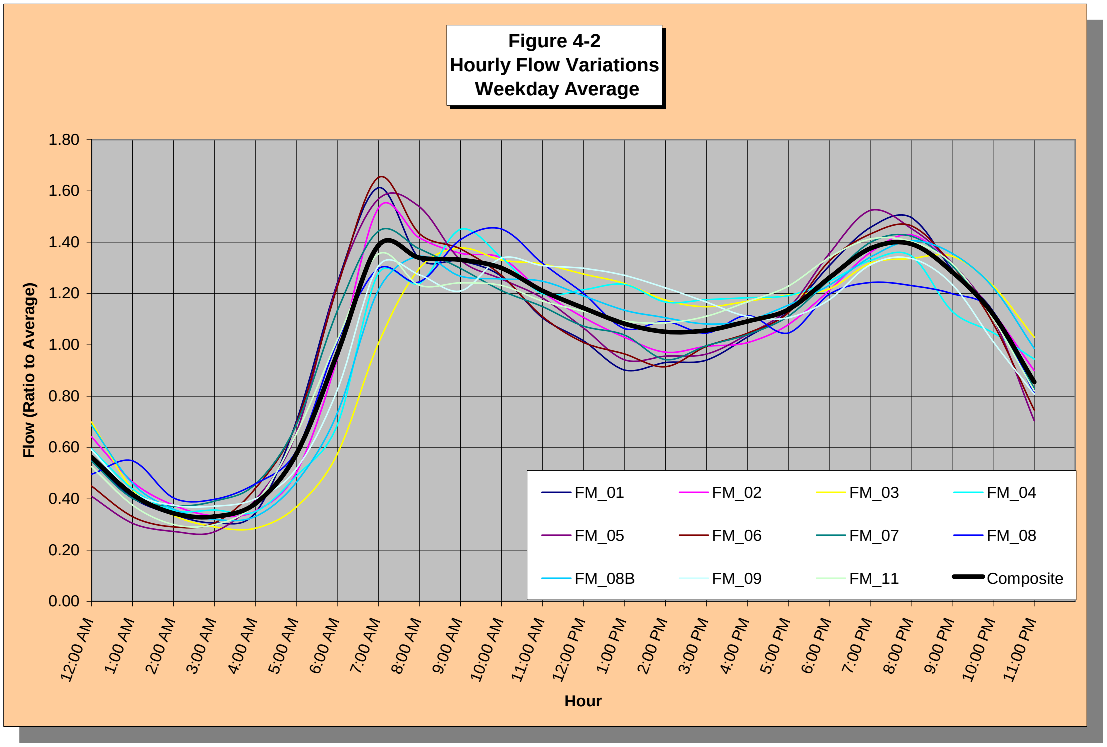


Figure 4-2
Hourly Flow Variations
Weekday Average
Flow (Ratio to Average)
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
12:00 AM
1:00 AM
2:00 AM
3:00 AM
4:00 AM
5:00 AM
6:00 AM
7:00 AM
8:00 AM
9:00 AM
10:00 AM
11:00 AM
12:00 PM
1:00 PM
2:00 PM
3:00 PM
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Hour
FM_01
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FM_04
FM_05
FM_06
FM_07
FM_08
FM_08B
FM_09
FM_11
Composite

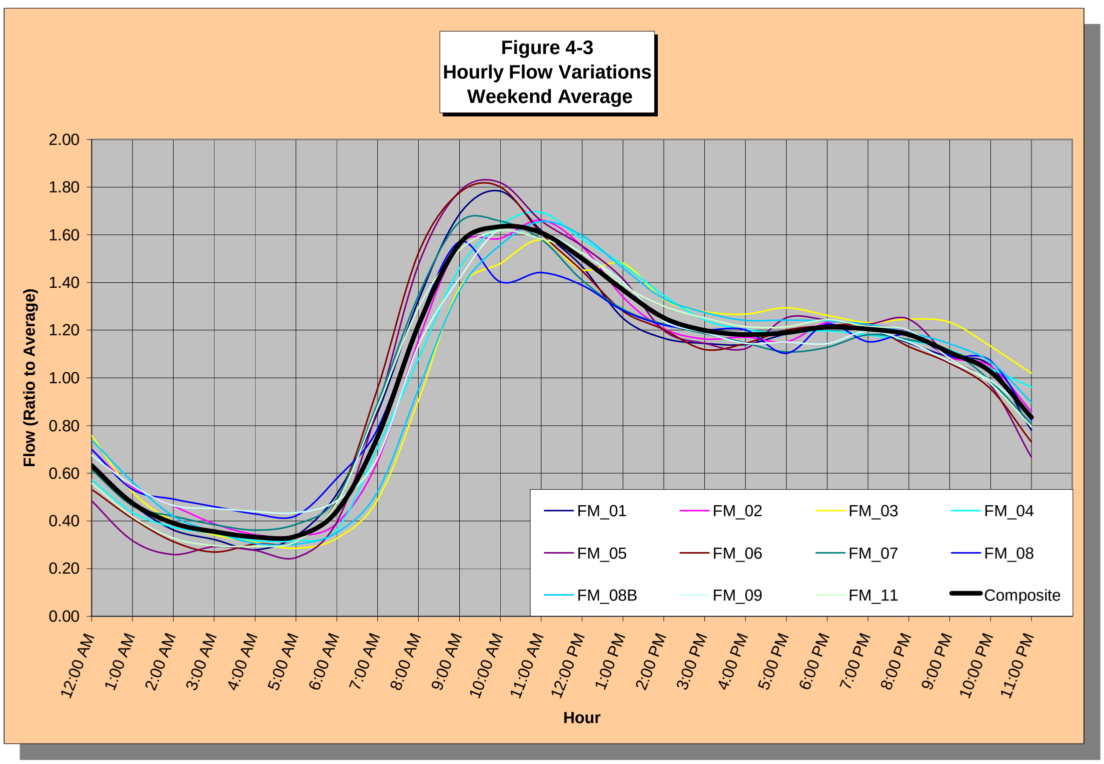
Figure 4-3
Hourly Flow Variations
Weekend Average
Flow (Ratio to Average)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
12:00 AM
1:00 AM
2:00 AM
3:00 AM
4:00 AM
5:00 AM
6:00 AM
7:00 AM
8:00 AM
9:00 AM
10:00 AM
11:00 AM
12:00 PM
1:00 PM
2:00 PM
3:00 PM
4:00 PM
5:00 PM
6:00 PM
7:00 PM
8:00 PM
9:00 PM
10:00 PM
11:00 PM
Hour
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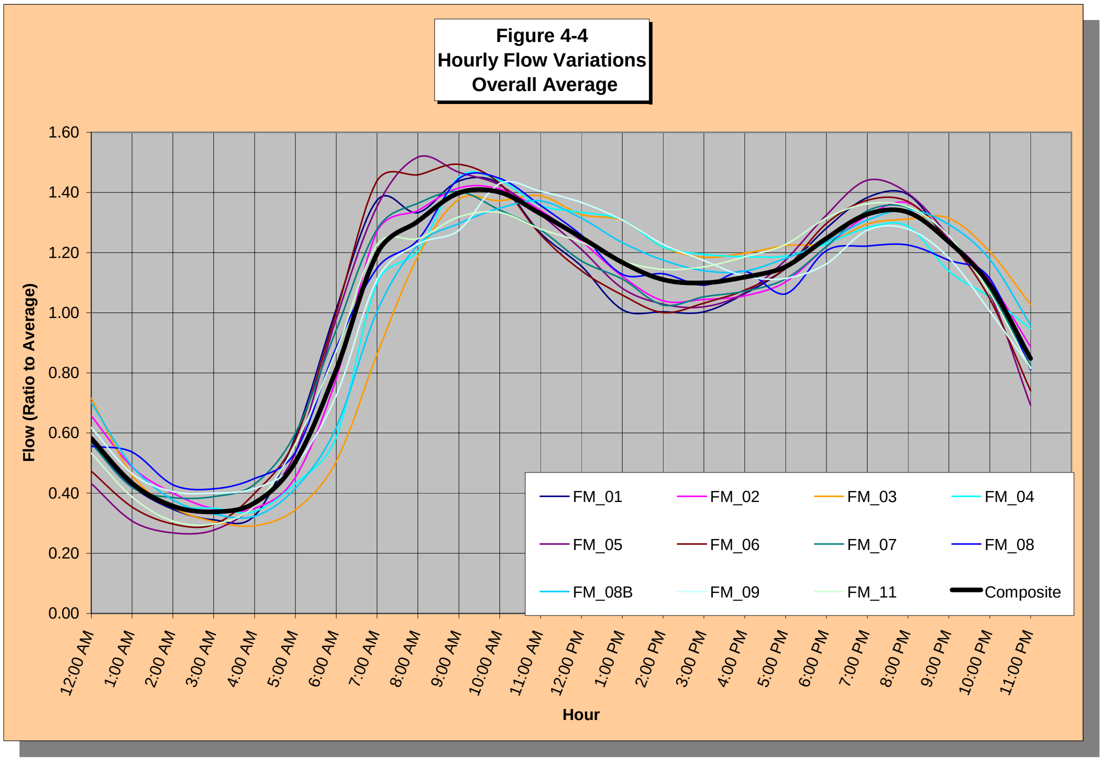
FM_09
FM_11
Composite



Based on the flow monitoring data, and our previous experience, the following peaking relationship was selected for this study:

$$Q_{peak}\ (cfs) = 1.95 \times Q_{ave}(cfs)^{0.92}$$

The peak wet weather flow (PWWF) has two components: peak dry weather flow (PDWF) and rainfall dependent inflow/infiltration (I/I). Therefore, the following equation applies:

$$PWWF = PDWF + I/I$$

Inflow and infiltration is discussed further in Sub-section 4-5.

4-5 Inflow and Infiltration

Inflow is the surface water that typically gains entry to the sewer system through perforated or unsealed manhole covers and illegal connections, such as roof and yard drains, during rainfall events. Infiltration is defined as water entering the collection system from the ground through defective pipes, pipe joint connections, or manhole walls. The sewer system design capacity must include allowances for these extraneous components, which inevitably become a part of the total flow. The amount of inflow and infiltration (I/I) that enters the system typically depends upon the availability and location of the stormwater drainage facilities, age of structures, materials and methods of construction, the location of the groundwater table, and the characteristics of the soil. In absence of flow monitoring data, many regulating agencies implement commonly accepted practices for estimating I/I. For example, I/I is often estimated based on the diameter and length of pipeline (100 to 400 gpd/ in. dia/ mile) or as a percentage of the peak flow or pipeline capacity.

Our experience from other master planning studies and review of limited flow monitoring information available during severe rainfall events indicate that the peak wet weather flow can vary from 10 percent of average dry weather flows in steeper areas with adequate drainage facilities, to over 400 percent of average dry weather flows in flat areas that lack significant drainage facilities.

For this master plan, several of the more significant and recent storm events that occurred during the past 10-years were studied. Rainfall information as well as treatment plant flows was used to evaluate the extent of the inflow and infiltration in the study area. Rain data was collected at two rain gage stations: Prado Dam (Army Corp of Engineers) and Chase and Taylor (Riverside County Flood Control and Water Conservation District). The studied rain events occurred on January 4, 1995; December 6, 1997; January 10, 2001; February 25, 2003; and March 16, 2003. Hydrographs were created for each of these rainfall events. Based on the Riverside County Hydrology Manual criterion, each event was assigned a storm frequency.

The average dry weather flows before and after the rainfall were compared to the peak flows seen at Wastewater Treatment Plants 1 and 2 during the wet weather events. A ratio of peak wet weather flow to average dry weather flow was then calculated. This factor was then plotted versus the frequency of the event and extrapolated. For a wet weather event with a 10-year frequency, the peak wet weather flow to average dry weather flow ratio is estimated at 2.5. Per conversations with City staff, the recent design of Wastewater Treatment Plant 3 utilized a factor of 2.6. Therefore, a factor of 2.6 will also be used for this Master Plan.

4-6 Sewer Design Criteria

Design criteria are established to ensure that the wastewater collection system can operate effectively under all flow conditions. Each pipe segment must be capable of carrying the peak flows without surcharging the system. Low flows must be conveyed at a velocity that will prevent solids from settling and blocking the system.

The City of Corona's minimum allowed slope of an 8-inch gravity sewer pipe is 0.004 feet per foot. Where this slope cannot be attained due to existing constraints, a more in-depth analysis must be performed and submitted to the City for approval. At a minimum, all pipes must be 8 inches or larger in diameter and the velocity of flow should be greater than 2 feet per second at average flow. This velocity will prevent deposition of solids in the sewer. A velocity of 3 feet per second is desired at peak dry weather flow, to resuspend any materials that may have already settled in the pipe. The corresponding minimum slopes for the various pipe sizes are as shown in Table 4-3.

It is important to note that the sewer sizes and slopes in Table 4-3 are provided for information only and assume the depth of flow in the pipe is half or two-thirds full (see footnote below Table 4-3). If there is insufficient flow to create this condition, greater slopes than those shown are necessary to acquire the desired minimum velocities.

Table 4-3
Minimum Sewer Sizes

Sewer Size	2 ft/sec Velocity Slope	3 ft/sec Velocity Slope
8" (1)	0.0040	0.0074
10" (1)	0.0025	0.0055
12" (1)	0.0020	0.0044
15" (2)	0.0012	0.0027
18" (2)	0.0010	0.0021
21" (2)	0.0008	0.0017
24" (2)	0.0007	0.0015

(1) *Assuming sewer is flowing 1/2 full and n = 0.013*
(2) *Assuming sewer is flowing 2/3 full and n = 0.013*

The design and analysis of gravity sewer systems are typically based upon the depth to diameter ratio (d/D). Pipes should meet the City's established criterion as follows:

- ***New*** pipes 12-inches in diameter and smaller: **d/D = 0.50** at peak dry weather flow.
- ***New*** pipes 15-inches in diameter and larger: **d/D = 0.67** at peak dry weather flow.
- ***Existing*** pipes 12-inches in diameter and smaller: **d/D = 0.64** at peak dry weather flow.
- ***Existing*** pipes 15-inches in diameter and larger: **d/D = 0.67** at peak dry weather flow.

All pipes should be designed with a d/D less than or equal to 0.82 at peak wet weather flows. This reserves a minimum of 50 percent (12-inch and smaller pipes) to 26.5 percent (larger pipes) of the pipe's full flow capacity for inflow and infiltration.

Considering the City's I/I study, it is recommended that the peak wet weather flow be estimated as the larger of the two following equations:

1. Peak Wet Weather Flow (PWWF) = 1.265 x Peak Dry Weather Flow (PDWF)
2. Peak Wet Weather Flow (PWWF) = 2.60 x Average Dry Weather Flow (ADWF)

The extra pipeline capacity also allows for the possibility that actual wastewater flows may be slightly higher than anticipated, especially during the hours when instantaneous or intermittent peak flows may occur. The peak dry weather flows are generally observed between the hours of 6:00 a.m. and 9:00 a.m. and 7:00 p.m. and 9:00 p.m. in residential areas. Industrial areas usually produce peaks later in the day. Peak flows may also be observed during rainfall events due to inflow. In addition to providing peak wet weather flow capacity, the area above the peak daily water surface helps to keep the sewage aerated, reducing the possibility of septic conditions and odors.

The design capacity of a gravity pipeline is the calculated capacity of the pipeline based on the Manning formula:

$$Q = 1.486 \; A \, R^{2/3} S^{1/2} / n$$

Q = flow in cubic feet per second

R = hydraulic radius in feet = A / P

A = cross-sectional area of the pipe in square feet

P = wetted perimeter in feet

S = slope of pipe in feet of rise per foot of length

n = Manning's friction factor

Sewer system capacity is established using a Manning's friction factor of 0.013 for vitrified clay pipe. A summary of sewer system design criteria is listed in Table 4-4.

Table 4-4
Sewer System Criteria

Collection System		
Minimum Pipe Size		8-inch
Minimum Velocity		2.0 ft/sec at average flow
		3.0 ft/sec at peak dry weather flow
Pipe Depth to		0.50 for pipes 12-inches and smaller at peak dry weather flow
Diameter Ratio		0.67 for pipes 15-inches and larger at peak dry weather flow
Lift Station		
Pumps	*	Minimum 2 each sized at peak flow
	*	Minimum solids handling capacity 3"
Wet Wells	*	Sized to limit pump cycling to less than 4 to 5 times/hr
	*	Provide sufficient storage at peak flow to allow response to a failure
	*	Equipment to be maintained must be accessible without entering structure
Ventilation	*	12 air changes/hr minimum in dry well and as required by NFPA 820
	*	30 air changes/hr minimum in wet well if not operated continuously
	*	12 air changes/hr minimum in wet well if operated continuously
Controls		Redundant system. Float operated back-up controls.
Emergency Power		Stationary source with automatic transfer switch
Telemetry		Dialer system (minimum) at all pump stations to alert personnel in the event of a station failure
Forcemains	*	Minimum velocity 3.0 ft/sec
	*	Minimum size 4-inches
	*	Air Vacs installed in vaults

4-7 Lift Station Design Criteria

It is desirable to develop a sewer collection system with as few lift stations as possible due to the associated cost and maintenance required. This master plan recommends replacement and upgrading of 9 lift stations. Further analyses, evaluation, and recommendations regarding the City's lift stations are found in Section 7. No new lift stations are currently recommended. However, in the case that a lift station is necessary or when upgrading or replacing an existing lift station, it must be designed to be reliable and sized with sufficient capacity. It must contain redundant equipment, an emergency power supply, sufficient storage, and be able to notify the appropriate personnel in the event of failure.

The primary components of a typical lift station are the wet well, dry well, pumps, motors, valves, ventilation and electrical equipment, controls and the force main. The following general criteria are recommended.

The wet well stores the incoming wastewater until a pump is activated to discharge it to a gravity facility for further conveyance to a treatment facility. It should be designed with sufficient capacity to prevent short cycles whereby the pumps frequently start and stop, yet small enough that it will regularly evacuate sewage from the wet well to prevent the wastewater from becoming septic. Generally, the desired number of pump cycles should be limited to no more that 6 per hour for motors up to 10 horsepower. Motors up to 75 horsepower should start no more than 4 times per hour. Larger motors should cycle less frequently. Lift stations should also have sufficient volume to store sewage in the event of mechanical or electrical failures, until the City can respond to the failure and prevent overflows.

The pumps should be sized to efficiently handle the peak wet weather flows. A minimum of two pumps sized at the peak wet weather flow to the station should be provided so that sufficient standby capacity is available when one pump is removed for repairs or experiences a mechanical failure. The pumps should be able to pass a minimum solid size of 3-inches without clogging. The shafts, seals and impellers should be constructed of wear resistant material to provide long life. Tungsten Carbide seals, Ni-Hard impellers, and 316 stainless steel pump shafts are recommended. For services where aggressive agents may be found in the sewage, such as at golf courses, complete stainless steel construction is recommended. This includes the pump bowl, shaft, impeller, and motor housing.

The dry well houses the valves, pumps, motors and electrical equipment and controls. It must be properly ventilated and provide unobstructed access to all equipment. A minimum 3-foot clearance from all obstructions should be provided. Greater clearances may be required for equipment with special maintenance needs. Provisions for equipment removal including hatches, large door openings, and hoists should also be provided.

The force mains should be selected to operate within a 3 feet per second to 5 feet per second velocity range, but should not be smaller than 4-inches in diameter.

While submersible lift stations may be utilized for the small flows, the larger lift stations should be the wet well/dry well type. They should be designed with easy access to all equipment. The National Electric Code classifies the wet wells of wastewater pumping stations as Class I, Group D, Division 1 facilities if continuously ventilated at less than 12 air changes per hour, and Division 2 if continuously ventilated at 12 or more air changes per hour. Dry wells, which are physically separated from wet wells, if ventilated at less than 12 air changes per hour, are classified as Class I, Group D, Division 2 locations. Wet wells, and under certain circumstances dry wells, are considered confined spaces and should be entered in accordance with the corresponding requirements of Occupational Safety and Health Administration (OSHA).

All lift stations should incorporate redundant control systems for operation of the pumps. A float system should be used as a backup for a primary control system that utilizes an ultrasonic device or a bubbler system for level measurement and pump operation.

Stations in which have an ultimate peak wet weather flow 500-gpm or greater shall be wet well/dry type pump stations. Stations with less than 500-gpm ultimate peak wet weather flow should be submersible type pump stations.

Telemetry equipment is required in all stations. The City's standard telemetry equipment consists of a radio, a telemetry panel, a RTU/PLC, and an antenna. Depending upon the complexity of the installation the City may also employ ethernet equipment and switches so that programming, data acquisition, monitoring of other sites can be accomplished with a laptop computer in the field.

An emergency power source is required to operate the pump station during outages of the primary power source. A standby generator with an automatic transfer switch is the preferred type of emergency power source.

While lift stations may be necessary to serve portions of the City's service area because of topographic requirements, all feasible efforts should be made to eliminate their use. In evaluating the feasibility of constructing a lift station, a detailed comparison with a gravity alternative should be made. The service lives of each facility, the cost of operation and maintenance, as well as the many problems associated with the development of flows during the first several years should be carefully considered. Service criteria for sewer lift stations are summarized in Table 4-4.

4-8 Service Life of Pipe and Lift Station Equipment

In addition to the design criteria discussed in previous sections, the useful lives for which relatively trouble-free service can be expected are also of great importance when assessing an existing or future sewer system. Once the service life of a facility is exceeded, it becomes subject to failure and is often expensive to maintain. The determination of useful life can be difficult and depends on many different considerations including the following:

- Type of materials used and recorded performance of similar installations
- Velocities and flow rates expected in the system
- Chemical and biological conditions of the wastewater
- Construction methods and installation quality
- Frequency, thoroughness, and types of Maintenance

The useful life values listed in Table 4-5 are generally accepted as prudent planning criteria and are used as benchmarks for replacement recommendations in this study. The majority of the pump stations' piping, valves, mechanical, and electrical components have exceeded the recommended useful lives listed in Table 4-5 due in large part to the City's fine maintenance practices.

Table 4-5
Planning Criteria for Facility Useful Life

Facility	Description	Useful Life (Years)
Gravity Sewers:	Asbestos Cement Pipe (ACP)	50
	Cast Iron Pipe (cip)	50
	Ductile Iron Pipe (DIP)	50
	Plastic Pipe	50
	Truss Pipe	50
	Vitrified Clay Pipe (VCP)	50
	Manholes	50
Forcemains:	Asbestos Cement Pipe (ACP)	50
	Cast Iron Pipe (cip)	50
	Ductile Iron Pipe (DIP)	50
	Plastic Pipe	50
Lift Stations:	Structure	40
	Piping	15
	Valving	15
	Mechanical	15
	Electrical	20

Values listed above are from the State Controller's Uniform Utility Accounting System for the State of California, Appendix A, Suggested Useful Lives of Fixed Assets