CASTLEROCK (EAST ELLIOTT) WATER, SEWER, AND RECYCLED WATER STUDY

July 2007

PBS&J Project No.: 620681.18

PADRE DAM
Municipal Water District

Prepared For:
Padre Dam Municipal Water District

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TABLE OF CONTENTS

CHAPTER **PAGE**

FIGURES **PAGE**

TABLES

APPENDICES

CHAPTER 1
INTRODUCTION

1.1 PURPOSE

The purpose of this study is to determine the on-site and off-site water, sewer, and recycled water facilities that will be required for Padre Dam Municipal Water District (District) to provide service to the proposed Castlerock Development (Development) proposed by Pardee Homes (Pardee). For the purposes of this study, it is assumed that the Development will be annexed into the City of Santee and District service areas. Based on the District planning and design criteria, this study identifies the preferred source of water supply to the development and provides recommended on-site water and recycled water distribution systems and a sewer collection system. This report is based on the Development's Tentative Map, prepared by Latitude 33.

Castlerock is a planned development currently located within the East Elliott Community Planning Area of the City of San Diego as shown in Figure 1-1. The proposed land use plan includes 424 residential units, of which 282 are single family units and 142 are multi-family units.

1.2 SCOPE

This report was prepared for the District to ensure that the recommended on-site and offsite facilities presented herein will adequately serve the ultimate needs of the Development.

The scope of this report includes the following principal elements:

- Review current development plans for Castlerock,
- Based on District planning criteria, determine potable and recycled water demands for the Development,
- Based on District planning criteria, determine sewer generation for the Development,
- Identify existing facilities which will provide water and recycled water service to the Development,
- Identify existing facilities which will provide sewer collection service to the Development
- Determine the on-site and off-site improvements needed, if any, to provide potable and recycled water service for the Development,
- Determine the on-site and off-site improvements needed to provide sewer service for the Development, and
- Prepare and submit a water, sewer, and recycled water study, with hydraulic modeling, to the District.

1.3 INTRODUCTION

The Elliott Community Plan was adopted by the City of San Diego's (San Diego) City Council in April 1977 as Resolution 202550. The planning area encompassed approximately 10,120 acres from Murphy Canyon on the west; Friar's Road, the San Diego River and Sycamore Canyon on the south and east; and the Camp Elliott military reserve on the north. Over the years, some of this area was removed from the Elliott Community Plan and incorporated into the new Tierrasanta Community and Mission Trails Regional Park Plans. The remaining portion of the Elliott Community Plan, known as East Elliott, has remained largely undeveloped.

The Elliott Community Plan was amended in 1997, Resolution R-288456 to address the East Elliott planning area. The amendment expanded the area designated for open space to correspond to the open space boundaries of the Multiple Species Conservation Program from 1339 acres to 2259 acres. The residential area was reduced from 1380 acres to 117 acres adjacent to the City of San Diego's eastern boundary because of its proximity to the City of Santee (Santee), with a maximum of 500 dwelling units. The acreage designated for the landfill was increased from 113 acres to 474 acres.

The Castlerock residences will be located within the 117 acres designated in the East Elliott Community Plan. The balance of the site (approximately 1,000 acres) will remain as permanent open space. Figure 1-1 illustrates the location of the East Elliott Community planning area within the City of San Diego and illustrates the development limits of the Castlerock Development.

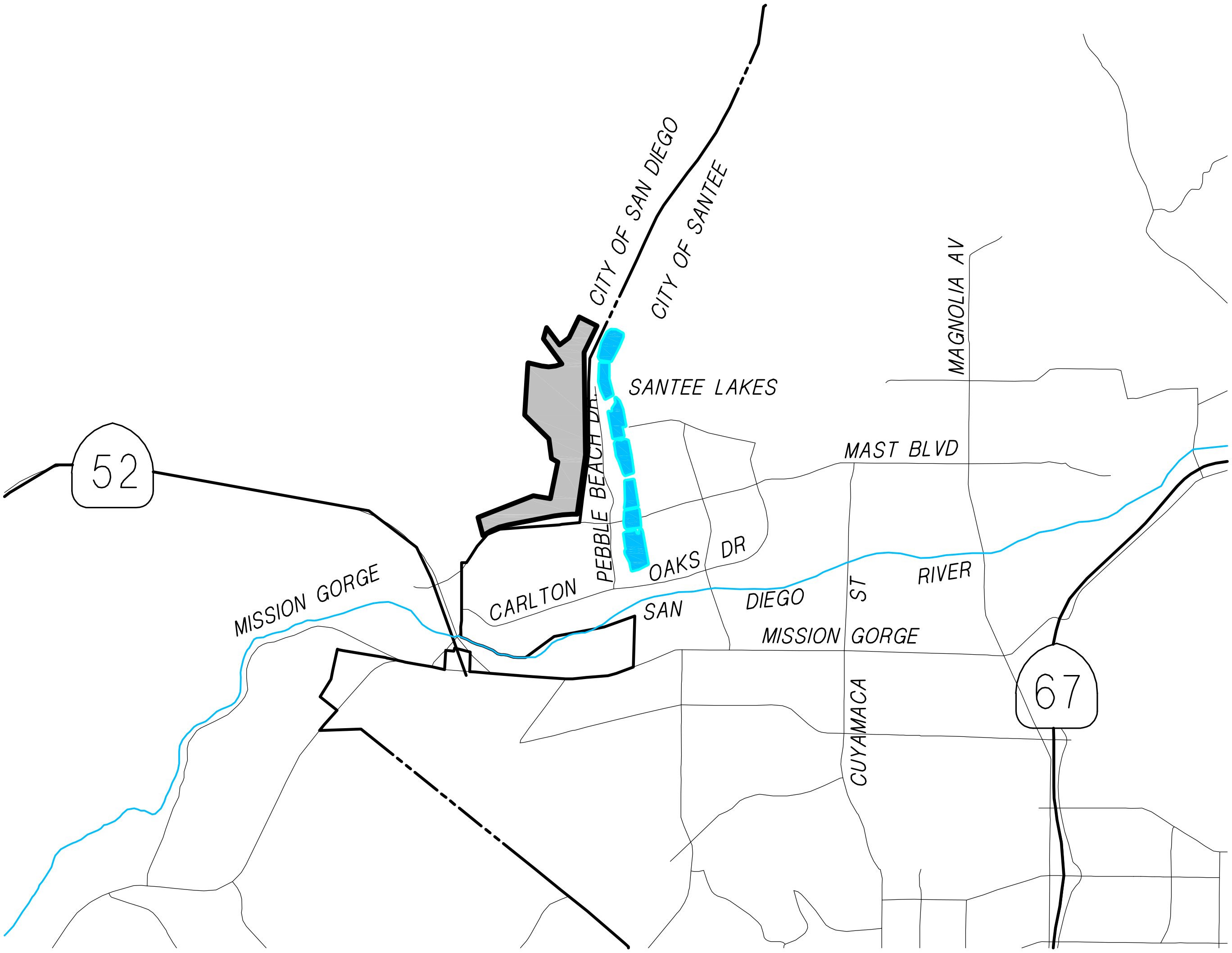
In April 2006, an Alternatives Study was prepared in support of the environmental documents. The Alternatives Study developed water and wastewater service options for both the City of San Diego and the Padre Dam Municipal Water District. Pardee is investigating both service alternatives. This study addresses service from the District.

1.4 CASTLEROCK LAND USE

Figure 2-1 illustrates the areas designated for multi-family residential. Table 1-1 contains a summary of the land use designations as well as the anticipated dwelling count. As summarized in Table 1-1, 424 dwelling units (DUs) are proposed for the Development under the current proposed Tentative Map.

Table 1-1
Castlerock Land Use

Land Use	Dwelling Units
Single Family Residential	282
Multi-Family Residential	142
TOTAL	***424***



VICINITY MAP

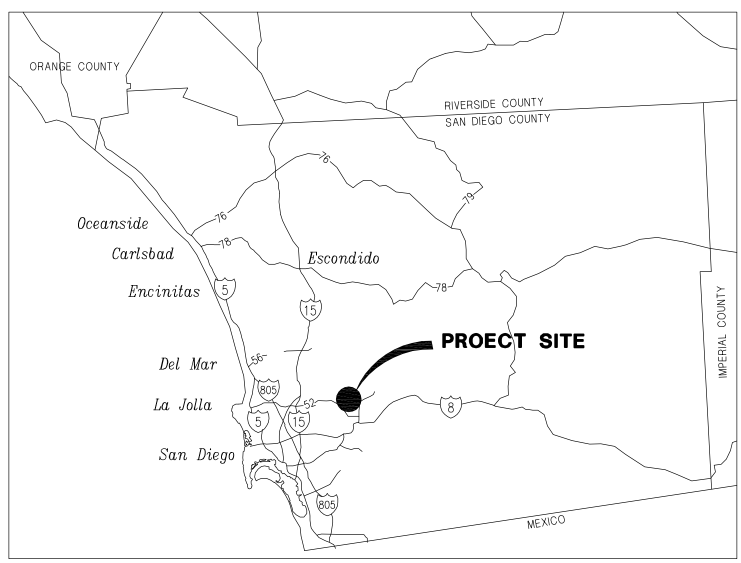
LEGEND

PROJECT SITE

CITY BOUNDARY

0 5000

SCALE IN FEET



LOCATION MAP

PROJECT LOCATION

Figure 1-1

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CHAPTER 2
POTABLE WATER SYSTEM

2.1 INTRODUCTION

Potable water service within the Castlerock Development is required for residential use and fire-fighting. Existing potable water service is located directly south and east of the Development. The Development will take supply from the existing 6.0 MG Carlton Hills Reservoir (629 HGL) just east of the Development via the existing 12-inch main in Mast Blvd.

2.2 POTABLE WATER SYSTEM AND PROJECTED DEMANDS

Based on discussions with the District and review of previous studies, the recommended unit water demands are shown in Table 2-1. Table 2-1 shows the projected demands for the Development. A potable water peaking factor of 2.0 was used to determine the maximum day demand (MDD) of 0.40 MGD (275 gpm). A peaking factor of 3.3 was used to determine the peak hour demand of 0.65 MGD (454 gpm).

Table 2-1
Projected Water Demands

Land Use	Dwelling Units	Unit Demand	Avg Annual Demand		Max Day Demand		Peak Hour Demand	
			gpm	MGD	gpm	MGD	gpm	MGD
Single Family Residential	282	480 gpd/DU	94.00	0.14	188.00	0.27	310.20	0.45
Multi-Family Condominiums	142	442 gpd/DU	43.59	0.06	87.17	0.13	143.83	0.21
TOTAL	***424***		***137.59***	***0.20***	***275.17***	***0.40***	***454.03***	***0.65***

The 2001 Integrated Facilities Plan (IFP) identified standard fire flow requirements for the various fire districts within the District's service area. Specific fire flow requirements must be determined by the local agency having jurisdiction. The City of Santee fire marshal is responsible for this area and has requested that a fire flow of 1,500 gpm be available for single-family residential and 2,500 gpm for multi-family residential developments. The latter was assumed to have a four-hour duration per the IFP.

2.3 DESIGN CRITERIA

Design criteria for the District's potable water facilities are outlined in the 2001 IFP. The design criteria are used to evaluate the existing water supply facilities and the size the new facilities based on peak flow demands and certain emergency conditions. The criteria defined in the IFP are summarized in Table 2-2.

Key analysis criteria include minimum and maximum allowable pressures within the distribution system piping under specified system operating conditions. Minimum pressure criteria are used to maintain minimum system pressures during peak domestic use periods and during fire scenarios. Maximum pressure criteria are used to set limits on the pressure to protect piping and plumbing.

Table 2-2
Design Criteria for Water Facilities

Facility	Criteria
Maximum Velocity, Peak Hour	7 fps
Maximum Velocity, MDD + Fire Flow	15 fps and meet pressure requirements (existing facilities)
	10 fps and meet pressure requirements (new facilities)
Maximum Allowable Headloss for 24-hour Max Day (no fire)	15 feet per 1000 feet of pipe (existing facilities)
	10 feet per 1000 feet of pipe (new facilities)
Hazen-Williams Roughness Coef., C	120
Maximum Desired Pressure	150 psig
Maximum Allowable Pressure	200 psig
Minimum Pressure (max day plus fire)	20 psig at fire hydrant
Minimum Pressure (peak hour)	40 psig at development pad elevation
Minimum Static Pressure	45 psig at the highest pad elevation (assuming low water level in reservoir)

Based on District pressure criteria and proposed pad elevations, the entire Development will be served by the 629 Pressure Zone. This zone has been established by the Gravity Zone connection points to the Wholesale system and the 6.0 MG Carlton Hills Reservoir. Proposed pad elevations for the Development range from 406 feet to 465 feet, resulting in static pressures ranging from 71 to 97 psi. The low water level for the Gravity Zone is set at 605 feet, which would result in a minimum static pressure of 61 to 86 psi.

2.4 EXISTING POTABLE WATER SYSTEM

The existing 6.0 MG Carlton Hills Reservoir currently serves the 629 Zone by gravity. The Castlerock Development is planned to receive service via an existing 12-inch main in Mast Blvd and 8-inch main in Pebble Beach Drive.

There is a deficit of storage in the 629 Zone at ultimate buildout. At this time the District will not require the construction of a new reservoir to serve the Development. However, water service connection fees paid will support construction of a new reservoir in the future. Detailed calculations are provided in Appendix D.

2.5 HYDRAULIC MODELING

A pipe and node network computer model using InfoWater was constructed to model the on-site water system for the Development. Presented in this section are model boundary conditions, model simulations, and results.

2.5.1 BOUNDARY CONDITIONS

Hydraulic grades were utilized from the ultimate District buildout model for peak hour and maximum day demand conditions.

2.5.2 METHODOLOGY

Hydraulic conditions of the proposed on-site distribution and District transmission system were evaluated using the hydraulic computer model. Analyses consisted of subjecting the proposed system to specified ultimate water demand conditions and assessing the resultant hydraulic conditions based on District design criteria. Projected peak hour demands and maximum day demands plus fire flow conditions were simulated for critical locations in the Castlerock water system. Only the most critical simulations that produced minimum pressures and maximum velocities are included herein.

Pipelines were sized to not exceed a maximum velocity of 15 fps under maximum day demands plus fire flow for existing facilities and 10 fps for new facilities. Pipeline head losses were estimated by use of the Hazen-Williams equation with a coefficient of 120 assumed for all pipelines. Table 2-3 provides a summary of critical simulations performed. Tabulated results for Castlerock and a pipe and node map are included in Appendix B. InfoWater simulation results for the entire Castlerock model are provided in Appendix C.

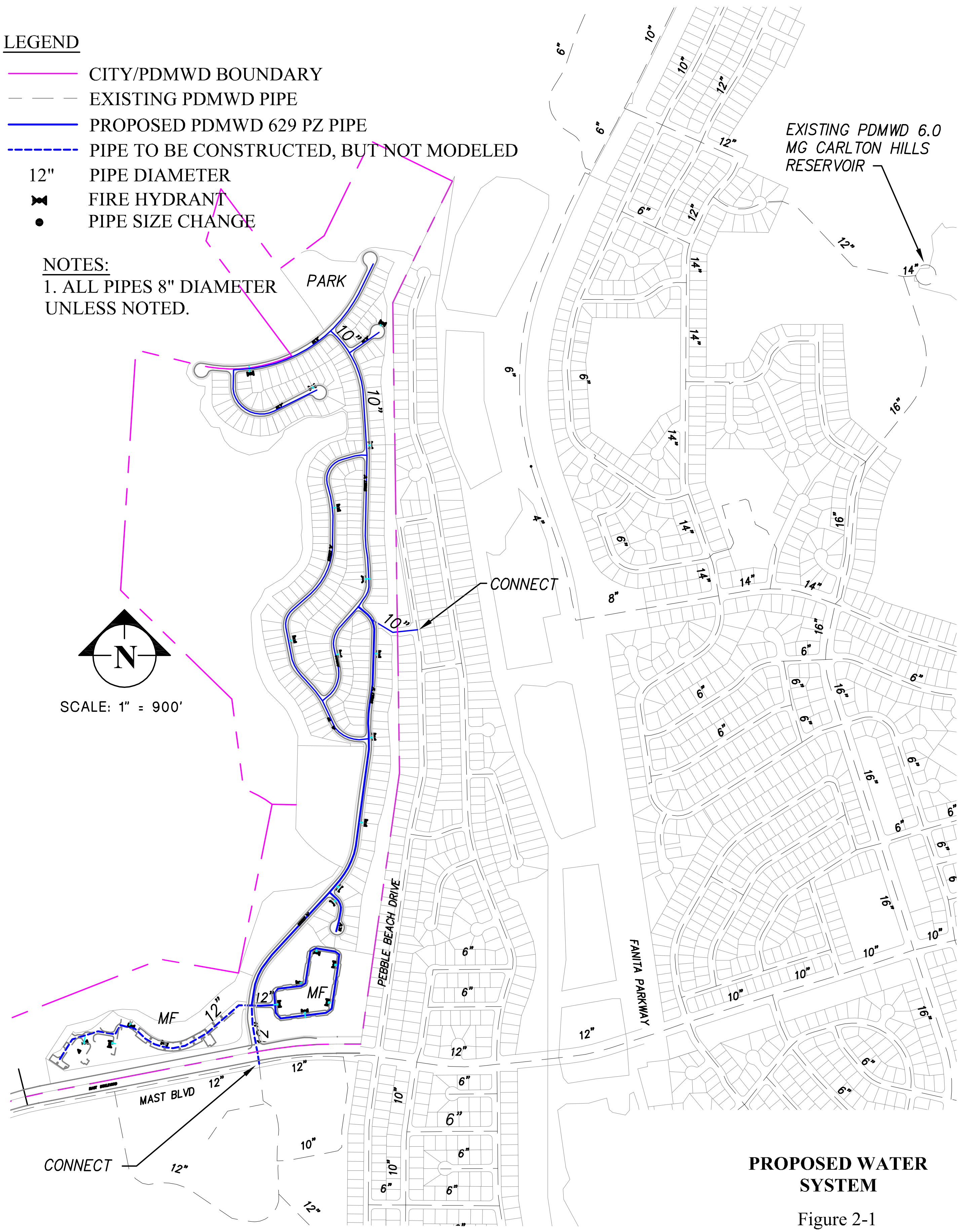
Table 2-3
Hydraulic Simulation Scenarios

Run No.	Description	Results
1	Peak Hour	Table B-1a Table B-1b
2	MDD + 1,500 gpm SF Fire at Node EE132	Table B-2a Table B-2b
3	MDD + 2,500 gpm MF Fire at Node EE104	Table B-3a Table B-3b

2.5.3 RESULTS AND CONCLUSIONS

The hydraulic analysis results indicate that the simulated minimum pressures for each phase of the Development under fire flow conditions met or exceeded the District's minimum pressure criteria of 20 psi. The simulated minimum pressure conditions met or exceeded the District's minimum pressure criteria of 40 psi during peak hour at all demand nodes. Additionally, the maximum pipeline velocities remained below the respective criteria. The proposed on-site water system shown in Figure 2-1 consists of 8-inch and 12-inch diameter mains. This system will have sufficient transmission capabilities to supply peak hour and fire flow conditions based on the District's water design criteria.

In addition, the offsite system was analyzed to determine if any significant impacts were caused by the additional demand of the Development. Results of selected locations are provided in Appendix B, Tables B-4a and B-4b. It was determined that the existing offsite system saw no significant impacts and would not require improvements.



PROPOSED WATER SYSTEM

Figure 2-1

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CHAPTER 3
WASTEWATER COLLECTION SYSTEM

3.1 INTRODUCTION

The purpose of this chapter is to present two alternative sewer options to serve the Development. This chapter also identifies recommended pipe sizes and alignments and also provides pipeline design parameters including peak depth of flow (d), d/pipe diameter (D) ratio, and pipe velocities as required per the District, for both sewer service alternatives.

3.2 BACKGROUND

The District will provide wastewater collection service to the Development. Wastewater will be collected from the Development and conveyed by gravity to the District IPS or directed via a District owned gravity pipeline to the San Diego Metropolitan Sewer System (METRO).

3.3 WASTEWATER GENERATION

Sewer flows are estimated for different land uses based on criteria provided in the IFP. Table 3-1 summarizes the wastewater generated by the Development. It is assumed that three people reside in each household. It should be noted that single family and multi-family residential were conservatively assumed to have the same generation rate.

Table 3-1
Projected Wastewater Generation

Land Use	Dwelling Units	Wastewater Generation Rate	Wastewater Generation
Single Family Residential	282	220 gpd/DU	62,040 gpd
Multi-Family Residential	142	220 gpd/DU	31,240 gpd
TOTAL	***424***		***93,280 gpd***

3.4 SEWER DESIGN CRITERIA

Sewer design criteria were based on the 2001 IFP which included dynamic modeling simulations and District's Water Agency Standards and Design Guide, summarized in Table 3-2, for steady-state calculations for sewer collection systems.

In analyzing the capacity of existing sewers, the Manning's equation and roughness coefficients from the 2001 IFP for various pipe materials were used. New sewer mains were sized using a roughness of 0.011 that is consistent with PVC pipelines. A steady-state peaking factor of 2.85 was used conservatively based on the District's 2001 IFP Criteria.

Table 3-2
Design Criteria for Sewer Pipelines

Facility	Criteria
Steady State Peaking Factors Based on Population	
Population:	Peaking Factor:
500	2.85
1,000	2.65
3,000	2.45
5,000	2.20
10,000	2.10
40,000 and greater	1.90
New Trunk Sewers	
Minimum Pipe Size	8"
Minimum Velocity	2 fps
Minimum Slope:	
6"	0.68%
8"	0.40%
10"	0.28%
12"	0.22%
15"	0.15%
Design Flow Depth at Peak Wet Weather Flow:	
6" to 12" Diameter	0.50
> 12" Diameter	0.75
Manning's Roughness Coefficient by Pipe Material:	
PVC	0.011
ABS (Plastic Composite)	0.011
TECH	0.011
VCP	0.013
Existing Trunk Sewers	
Design Flow Depth at Peak Wet Weather Flow:	
8" to 12" Diameter	0.75
> 12" Diameter	0.75
2001 IFP Sewer Criteria [1]	
Design Flow Depth at Peak Wet Weather Flow:	
8" to >12" Diameter	1.20

[1] Padre Dam Municipal Water District IFP (November 2001) Chapter 2, Planning & Design Criteria. Criteria uses dynamic hydraulic model simulation with wet weather flows.

3.5 PROPOSED WASTEWATER COLLECTION SYSTEM

This section describes the on and off-site sewerage collection system alternatives that will be evaluated as part of this study. Alternative 1 will convey flows from the Development to the District's Influent Pump Station. Alternative 2 will convey flows from the Development to the City of San Diego's Mission Gorge Trunk Sewer. The on-site collection system for both options is shown graphically on Figure 3-1. The following sub-sections describe in more detail the proposed on and off-site collection systems for each alternative.

3.5.1 Alternative 1 – Influent Pump Station

On-site Sewer Collection System

The proposed on-site sewer collection system is shown on Figure 3-1 and will consist of 8-inch diameter PVC gravity mains. Due to the topography, the development's wastewater drains toward two separate connection points of the existing District collection system. The northern portions of the Development will drain to a low point near Moana Kia Lane. The southern portion of the Development will drain to Mast Boulevard via Street "A". Development flows will be conveyed east and south to the District's Influent Pump Station.

Off-site Sewer Collection System

The off-site gravity sewer will convey flows from the Development to the Influent Pump Station. As shown on Figure 3-2, wastewater from the north portion of the development requires a new 8-inch diameter sewer pipeline along Moana Kia Lane to connect to the existing District sewer system. The wastewater would then flow north through the existing sewer system along Medina Drive, east down Grass Valley Lane, and south along Pebble Beach Drive. Wastewater from the southern portion of the development requires a new 8-inch diameter pipeline along Mast until reaching Pebble Beach Drive. 260-ft of existing 8-inch diameter VCP sewer along Pebble Beach Drive exceeds District criteria. To avoid telescoping pipelines, the existing VCP pipeline could be slip-lined or the existing pipeline could be replaced with a new 8-inch diameter PVC pipeline.

The existing 30-inch diameter VCP sewer main draining to the IPS was shown as being slightly hydraulically deficient and the District has observed surcharging in this pipeline. At this time the District is not prepared to upsize this main. The Developer may be conditioned to pay additional capacity fees to support the replacement of this main in the future.

3.5.2 Alternative 2 – Mission Gorge Trunk Sewer

Onsite Sewer Collection System

The proposed on-site sewer collection system as shown on Figure 3-1 will consist of 8-inch diameter gravity mains, dual 4-inch diameter force mains and a pump station. Due to the topography, a majority of the development's wastewater will drain to a low point distant from the proposed off-site sewer main. Therefore, a sewer pump station will be required near the center of the development to pump sewage generated in the northern portions of the Development. The southern portion of the Development will flow by gravity to Mast Boulevard via Street "A". Development flows will be conveyed westward to the Mission Gorge Trunk Sewer (MGTS) via an offsite gravity sewer using the wastewater Transportation Agreement between the District and the City of San Diego.

Sewer Pump Station and Force Mains

The topography and geographic separation of the Castlerock development from the proposed off-site sewer main will require a sewer pump station, force mains and off-site gravity main to adequately service the Development. A public sewer pump station designed in accordance with the District criteria is proposed.

The pump station will convey the Development's peak sewer flows through two parallel sewer force mains (one included as a redundant backup) to the on-site gravity sewer in Street "A", and subsequently to the off-site gravity main. At the property entrance, a single gravity line conveys the flows westward in Mast Boulevard and southward in West Hills Parkway. The alignment of the off-site gravity main lies within the District's right-of-way until West Hills Parkway, where it crosses into the City of San Diego. The District will need to obtain an encroachment permit from the City of San Diego for the off-site sewer in West Hills Parkway.

Figure 3-3 and Figure 3-4 illustrate the preliminary site lay-out and cross section of the proposed pump station. The station will be located at an elevation of approximately 410 feet and pump flows to a high point in Street "A" of approximately 475 feet. The station will be equipped with emergency storage and back-up power. A preliminary design report will be submitted to the District outlining these design requirements if this alternative is selected. An initial estimate of pump station attributes includes:

Pumps	2 (primary and backup)
Capacity	150 gallons per minute
Wet well	20,000 gallons
Force Main Size	4-inches (dual)
Force Main Velocity	3.8 feet per second
Force Main Length	1,865 feet

Pump station and force main sizing calculations are provided in Appendix C.

Offsite Sewer Collection System

The off-site gravity sewer will start at the intersection of Mast Boulevard and Street "A" and extend approximately 3,400 feet west in Mast Boulevard and then approximately 2,700 feet southward in West Hills Parkway, before terminating at the City of San Diego's 42-inch MGTS. Figure 3-5 depicts the proposed off-site gravity sewer.

Minimal utilities are anticipated to be encountered within West Hills Parkway and Mast Boulevard, as these roadways are fairly wide for construction of an 8-inch gravity sewer main. The sewer main will need to cross over the San Diego River at an existing bridge structure along West Hills Parkway. The City of San Diego appears to have anticipated utilities in this area, as the bridge was designed to accommodate future utilities. Several design concepts were developed to cross the bridge. Figure 3-6 and 3-7 illustrates the concept design.

3.6 HYDRAULIC ANALYSIS

This section of the report evaluates the on and off-site wastewater collection system to accommodate flows from the Development and verify that the on-site system layout provided in the Tentative Map will meet the District's criteria for the two sewerage options. The results of the hydraulic analysis are presented in Appendix C.

3.6.1 Alternative 1 – Influent Pump Station

On-site Sewer Collection System

The results of the hydraulic analysis are presented in Appendix C and the recommended on-site sewer collection system is shown on Figure 3-1. Manhole depth of cover and rim and invert elevations are provided in Appendix C as well.

Off-site Sewer Collection System

The existing off-site collection system was analyzed by performing steady-state calculations utilizing Manning's equation. The projected peak wastewater flows were developed by adding peak dry weather flow from the Development to the 2020-Dry weather peak flows provided by the District. Table C-2 and Figure 3-2 summarize the required facilities to adequately convey flows to the Influent Pump Station.

3.6.2 Alternative 2 – Mission Gorge Trunk Sewer

On-site Sewer Collection System

The results of the hydraulic analysis are presented in Appendix C and the recommended on-site sewer collection system is shown on Figure 3-1. Manhole depth of cover and rim and invert elevations are provided in Appendix C as well.

Off-site Sewer Collection System

The off-site gravity sewer main was sized to convey only the peak sewer flows from both the proposed sewer pump station and southern on-site gravity flows. Based on the Development peak flows an 8-inch sewer constructed at a minimum slope of 1.0% will meet the District's design criteria. Table C-1 and Figure 3-5 summarize the required facilities to adequately convey flows to the MGTS.

3.7 RESULTS AND CONCLUSIONS

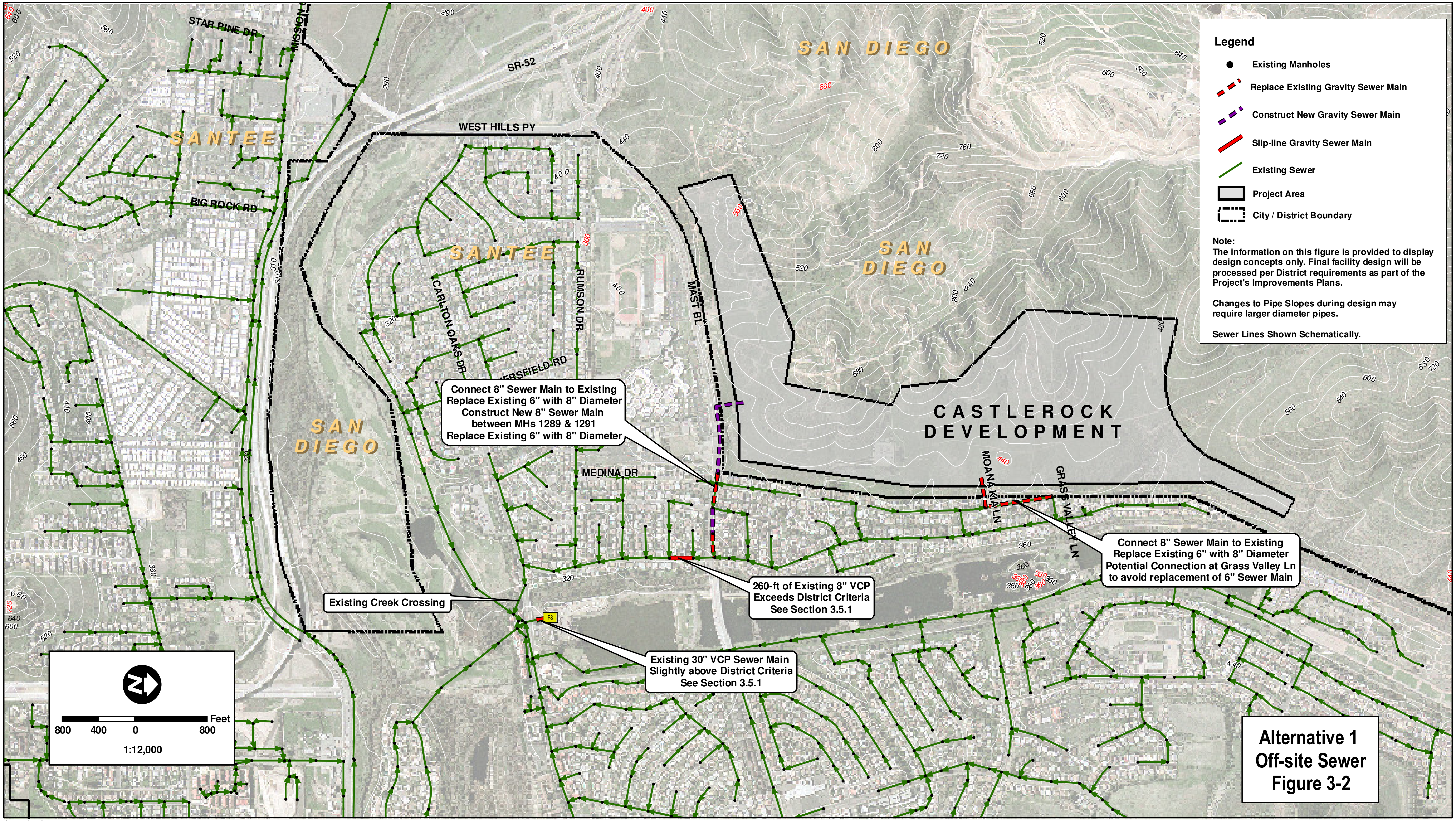
The proposed on-site sewer facilities for the Development meet the District's design requirements. The d/D ratio is less than 0.5 for all segments, and the velocity during peak flow is greater than 2.0 feet per second (fps) for both alternatives.

SCALE: 1:200

H:\CADD Design\CASTLEROCK WATER AND SEWER\FIGURES\FIGURE 3-1.dwg 08/15/07 16:41

ONSITE SEWER SYSTEM
Figure 3-1

CASTLEROCK WATER, SEWER, AND RECYCLED WATER STUDY
JULY 2007



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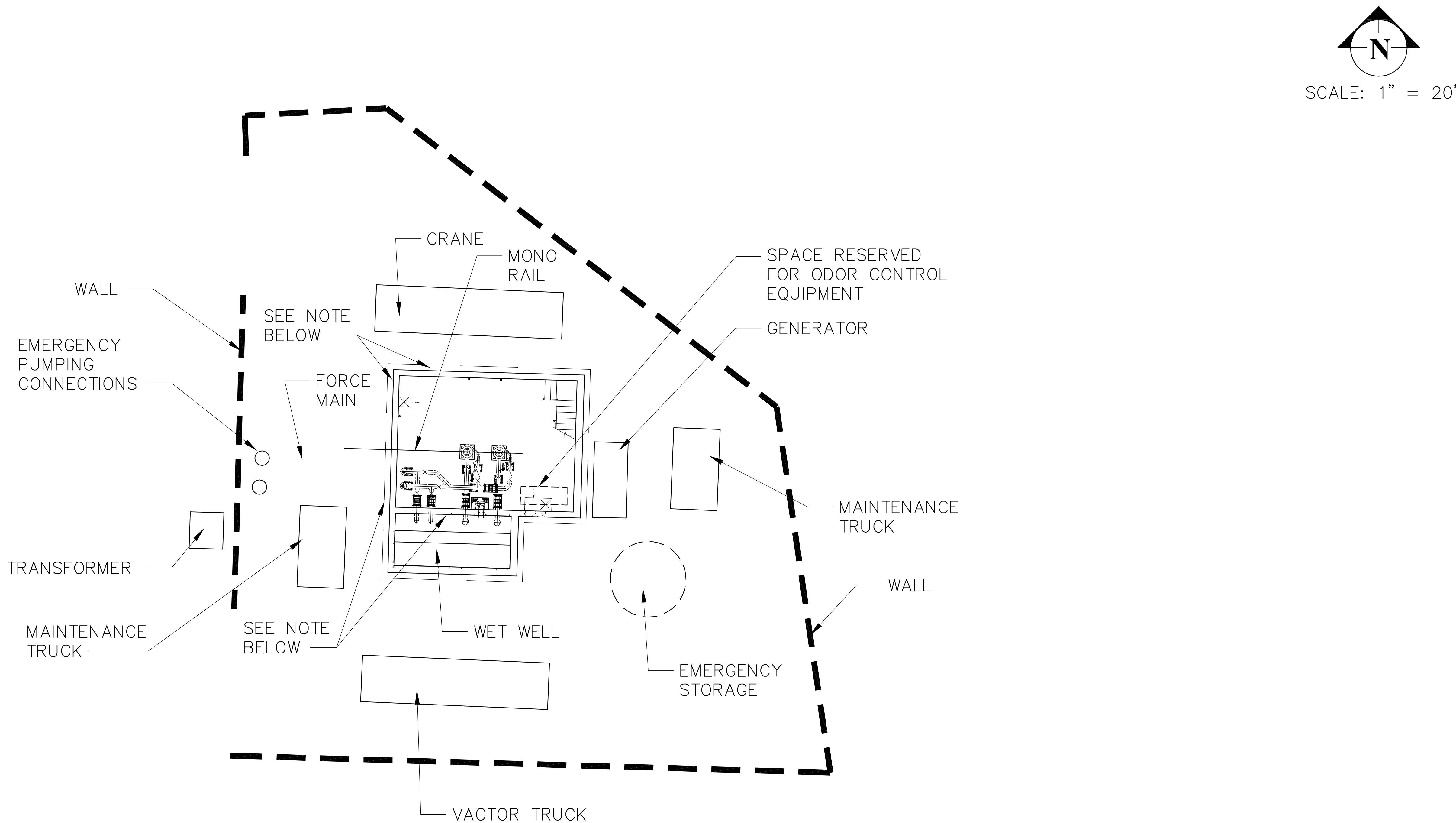

NOTE:
ARCHITECTURAL ENHANCEMENTS WILL BE INCLUDED ALONG THE FRONT & SIDES OF THE BUILDING TO BLEND THE FACILITY INTO THE RESIDENTIAL COMMUNITY.

ALTERNATIVE 2
CASTLEROCK
SEWER LIFT STATION
SITE PLAN

Figure 3-3

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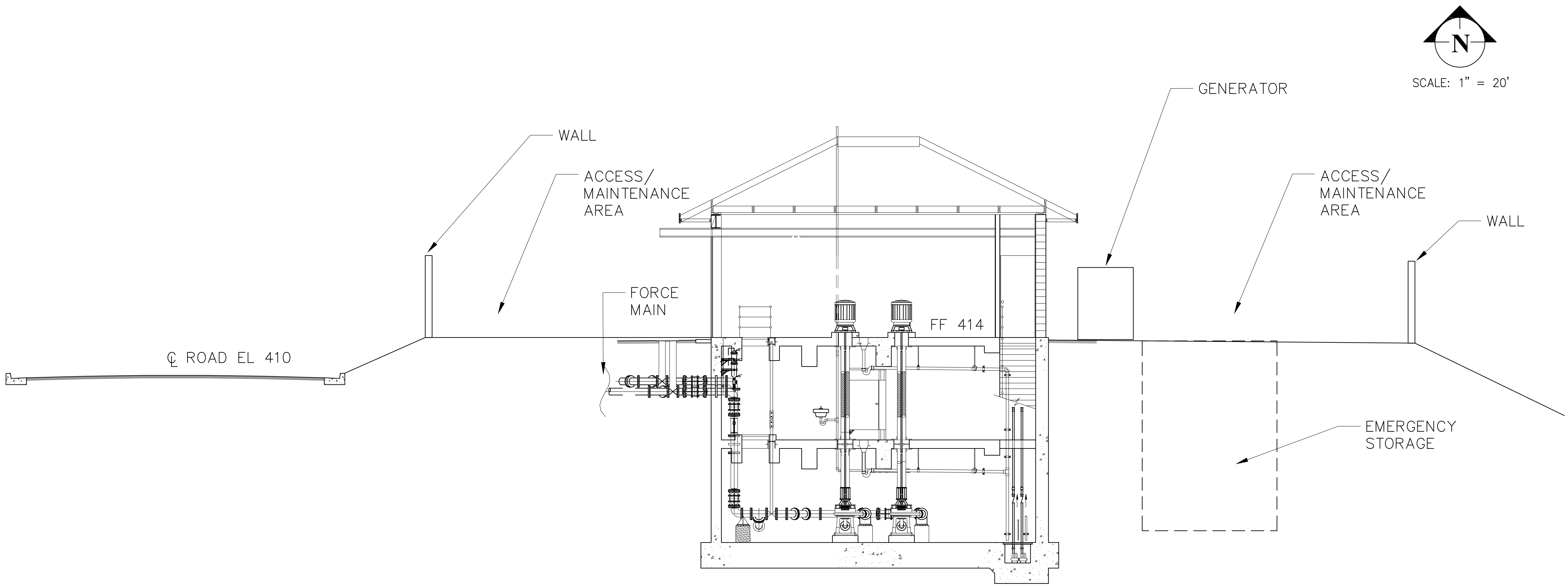

ALTERNATIVE 2
CASTLEROCK
SEWER LIFT STATION
SITE PLAN

Figure 3-4

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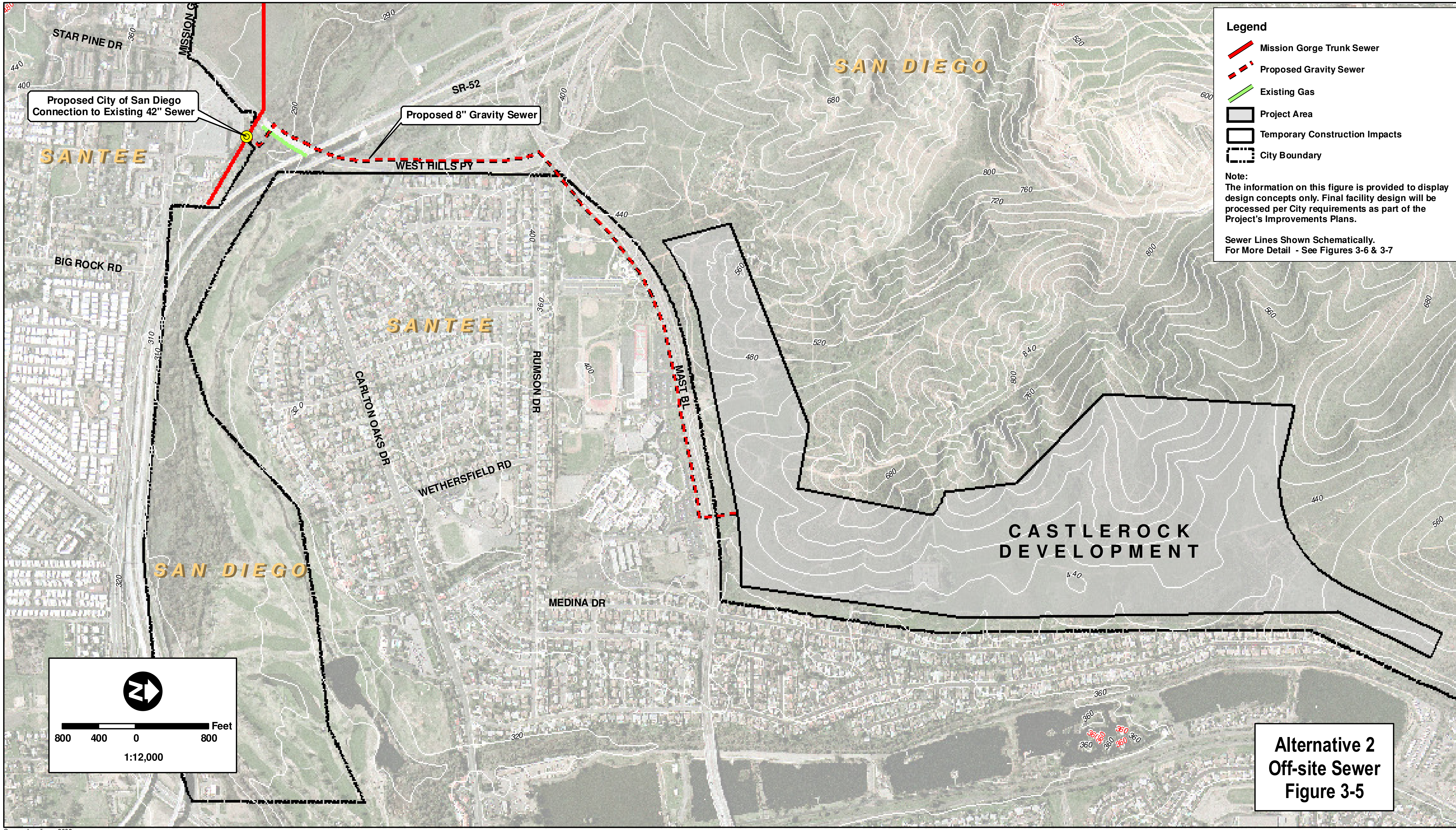

Source: Landiscor, 2003

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CHAPTER 4
RECYCLED WATER

4.1 INTRODUCTION

Regional recycled water facilities required to serve this area of the District were identified and sized in the 2001 IFP. These facilities include the Water Reclamation Facility (WRF) and the 1.5 MG Fanita Terrace Reservoir located south of Mission Gorge Road. The current capacity of the WRF is 2.0 MGD, of which 1.0 MGD is used to refill the Santee Lakes directly east of the Castlerock Development. The District may also supplement the recycled water system with potable water or other sources during peak summer periods prior to the proposed treatment plant expansion. The hydraulic grade of the District's recycled water system is 629-feet.

4.2 DESIGN CRITERIA

Design criteria for the recycled water system are presented in Table 4-1.

Table 4-1
Recycled Water Design Criteria

Facility	Criteria
Demand Factor	2.5 ac-ft/year/ac (1.55 gpm/ac)
Delivery Period	8 hours
Peak Day Peaking Factor[1]	6.36
Minimum Pressure	30 psi
Maximum Velocity	7 fps
Maximum Allowable Headloss	15 feet per 1000 feet of pipe (existing facilities) 10 feet per 1000 feet of pipe (new facilities)
Hazen-Williams Roughness Coef., C	120

1. Peaking factor determined by normalizing the IFP peaking factor of 2.12 into an 8-hour irrigation window. The 2.12 factor assumed a 24-hour window for irrigation.

The recycled zone has the same HGL as the potable water system at 629-feet. Therefore, the range of static pressures would remain the same at 97 to 71 psi. The low water level for the Gravity Zone is set at 605 feet, which would result in a minimum static pressure of 61 to 86 psi.

4.3 RECYCLED WATER DEMANDS

Recycled water will be used for the permanent irrigation of roadway medians, slopes, and parks. There are several temporary irrigation areas that may be supplied by recycled water or potable water. A summary of the irrigation areas and estimated recycled demands are shown in Table 4-2.

Table 4-2
Projected Recycled Water Demands

Permanent Irrigation		Temporary Irrigation		Total		
Area (ac)	ADD (gpm)	Area (ac)	ADD (gpm)	Area (ac)	ADD (gpm)	Peak (gpm)
26.0	46.7	18.6	33.5	44.6	80.2	510.2

Slope stabilization and other temporary recycled water uses will be needed during the first few years of development. These uses account for an additional 33.5 gpm – ADD as shown in Table 4-2, for a total temporary irrigation of 80.2 gpm - ADD. Ultimately the recycled water demand for the Development will be 46.7 gpm – ADD, or 297.1 gpm – peak.

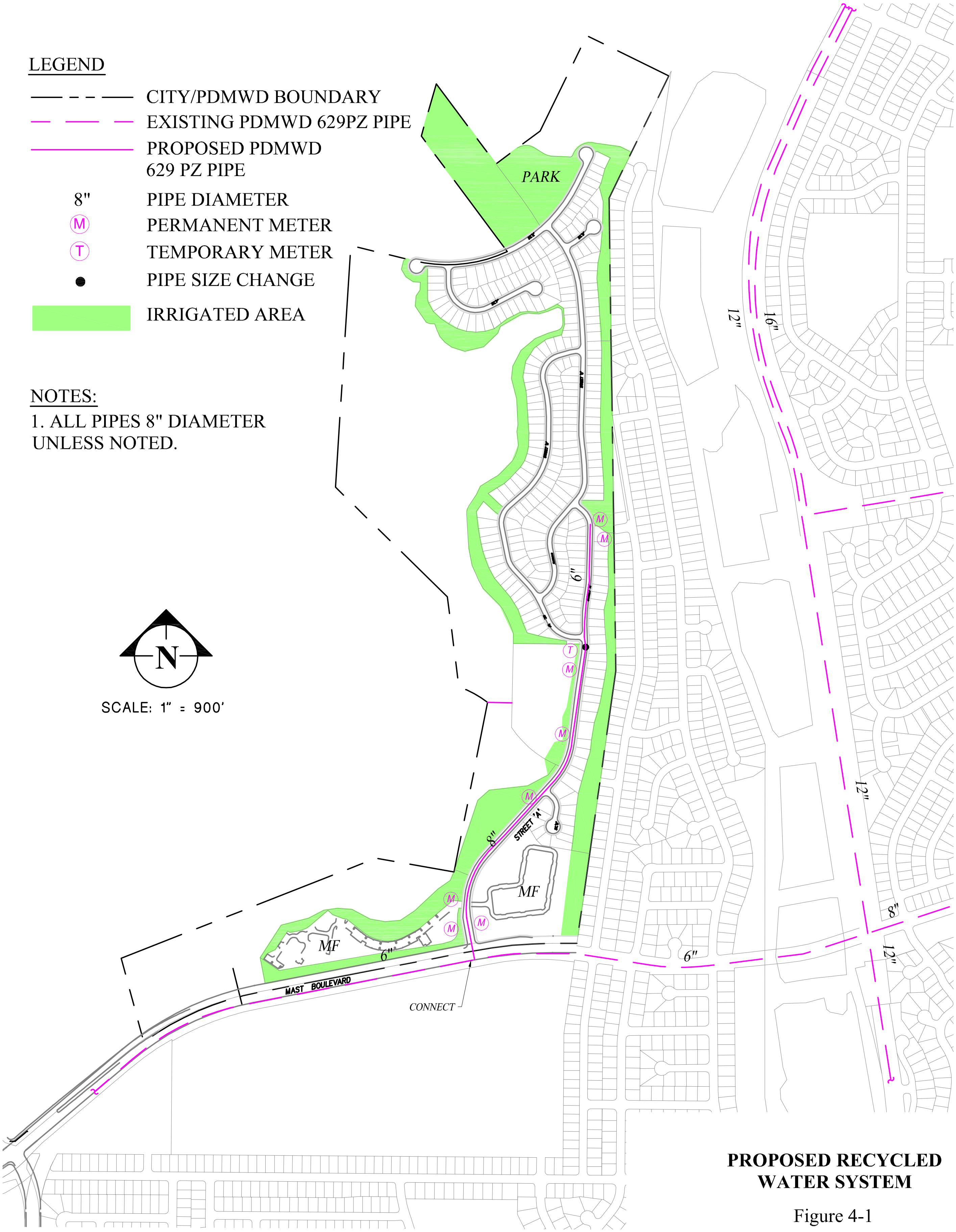
4.4 SYSTEM ANALYSIS

The Castlerock Development is planned to receive service via an existing 6-inch main in Mast Blvd. A single 8-inch/6-inch main will serve the Development along Street A based on the analysis summary presented in Table 4-3. The recycled water system and proposed meter locations are shown on Figure 4-1. The meter locations were coordinated with the Development Landscape Architect, KTU&A. The Landscape Architect located the meters to minimize pipelines in the Development roads. Further coordination with the Landscape Architect will be required once Development plans are finalized to maximize the use of recycled water.

Table 4-3
Recycled Water Analysis Summary

Condition	Peak Flow (gpm)	Diameter	Velocity[1]	Headloss (ft/1000-ft)[2]
Tempoary	510.2	8"	2.1	6.1
		6"	5.8	24.7
		[3]ex 6"	6.6	31.8
Ultimate	297.1	8"	1.9	2.2
		6"	3.4	9.1
		[3]ex 6"	4.2	13.7

Notes:
1. Velocities must be less than 7 fps per District criteria.
2. Headloss per 1000-ft must be less than 10-ft per District Criteria for new pipes and 15-ft for existing pipes.
3. Existing and projected flows in the 6” main in Mast Blvd were estimated at 73.9 gpm during peak flow conditions in addition to the demands of the Castlerock Development.



PROPOSED RECYCLED WATER SYSTEM

Figure 4-1

APPENDIX A
CORRESPONDENCE

An employee-owned company

July 31, 2007

Courtney Mael
Padre Dam Municipal Water District
9300 Fanita Parkway
Santee, CA 92071

SUBJECT: CASTLEROCK WATER, SEWER, & RECYCLED WATER STUDY – Response to Comments

Dear Mr. Mael:

Below are our responses to the District's comments on the Castlerock Water, Sewer, and Recycled Water Study.

General Comments

1. Include the Recycled Water System in the table of contents.

   The Recycled Water System has been included in the table of contents.

2. Include Figure 4-1 for Recycled Water under figures page ii.

   Figure 4-1 has been added to page ii.

3. Remove 2-4 and 2-5 from the tables there were not included in the study for Padre Dam. Add 4-1 for Recycled Water, page ii.

   Revised per comment.

4. Add any appendices required for Recycled water.

   No Appendix was required.

5. 1-1 Purpose – This section lists the number of units as 359 single family and 120 multi-family this is not consistent with several tables including table 1-1 which lists 272 single family, 87 and 120 multi-family. The difference between the 87 and 120 multi-family units is not defined. Please revise through out.

   The total number of units has been revised to 424 DUs. The text and tables match.

6. Be consistent with the terms through out the report. Refer to Padre Dam as the District, refer to the Castlerock Development as Development. The term project has been inserted instead of development and Padre Dam instead of District in a few places.

   All terms are consistent. All references to Padre Dam and called District, and all references to the Castlerock development are called Development.

Potable Water System

1. Use Table 2-1 from the 2001 IFP.

   Table 2-1 from the IFP is used in the report.

2. Use a potable water peaking factor of 2.0.

Maximum Day peaking factor was changed to 2.0 per comment.

3. Table 2-1 – Use a unit demand of 442 gpd/DU for condos or town homes and 398 gpd/DU for Apartments.

   The unit demand factors used in the study are consistent with previous planning studies performed by PBS&J for the District. However, the type of product anticipated for this Project would use the 442 gpd/DU. Demands have been revised per comment.

4. Methodology – The results should be based on District build out and that should be stated under methodology.

   Revised per comment, but stated under Boundary Conditions.

5. Show impact to existing facilities. Please provide the back up to the analysis done on the existing facilities and it no change is required than provide a paragraph summarizing that.

   Tables B-4a and B-4b have been added to Appendix B to show impacts to existing offsite facilities.

6. Please revise the paragraph below table 2-2 to include a reference to the low water lever of 605 and modify the pressure ranges to be 60-71 and 86-97 psi.

   Revised per comment.

7. A paragraph may need to be added to address the District storage and the impact of this Development within section 2-4.

   A paragraph has been added to this section, along with detailed calculations in Appendix D.

<u>Wastewater Collection System</u>

1. Typically we use d not dn to define peak depth of flow (d/D).

   The analysis was revised to use d instead of d_n.

2. Revise section 3.2 to read: The District will provide wastewater collection service to the Development. Wastewater will be collection from the Development and conveyed by gravity to the District IPS or directed via a District owned gravity pipeline to the San Diego Metropolitan Sewer System.

   This section was revised as commented.

3. Section 3.4 – District subdivision design criteria are the Water Agency Standards and Design Guide, please reference in the first paragraph.

   This section was revised as commented.

4. Where are the steady state peaking factors derived from? Please define.

   Peaking factors are shown in Table 3-2 and are derived from District Criteria and 2001 IFP. Additional language was added to the text to describing the peaking factor that was used.

5. d/D for existing and new pipe should be based on peak dry weather flow values.

   The analysis has been revised to include the 2020 peak dry weather flows, provided by the district.

6. New sewer main should us roughness of 0.011.

   The analysis has been revised to include a roughness of 0.011 for existing ABS, Tech, and PVC as well as new PVC sewer mains and 0.013 for existing VCP sewer mains.

7. No need to list ABS or Tech pipe.

There are existing ABS pipelines along Mast Boulevard and existing Tech pipelines along Pebble Beach Drive and as such, they have been left in Table 3-2.

8. Indicate the direction in the street the sewer flows, ie south in Pebble Beach.

   Flow direction is shown on the drawings and described in the text.

9. Flow in Medina Drive from Mona Kia Land heads North. The pipe in discontinuous in Medina Drive south of Mona Kia.

   Sewer flow direction has been revised and is shown on the figures.

10. The District does not allow the upsizing of an upstream pipe with upsizing all pipes downstream to equal or greater sizes. Please do not list the upsize of the pipe in Pepper Drive as optional.

    No optional sewer replacement in Pebble Beach Drive is included as a recommended project due to the use of 2020 peak dry weather flows per the Districts request.

11. Briefly describe and show how the existing Sycamore Creek Crossing is accomplished.

    Due to the use of 2020 peak dry weather flows per the Districts request, the existing Sycamore Creek Crossing is no longer a capacity constraint, and therefore the photographs were taken out of the Study.

12. It is the District's intent to follow the existing alignment of the creek crossing.

    The existing alignment of the creek crossing will be followed and no capacity constraints were identified.

13. The Development may impact the existing 30" pipe leading in the IPS and may cause additional surcharging. Please examine the impact and note it in the review.

    The existing 30" pipe was included in the analysis.

14. Show the alignment of the existing creek crossing in the photographs.

    Due to the use of 2020 peak dry weather flows per the Districts request, the existing Sycamore Creek Crossing is no longer a capacity constraint, and therefore the photographs were taken out of the Study.

15. Section 3.6.1 Please clarify how the projected peak wastewater flows were developed. Were they done by summing (peak?) development loading to 2020 WW flows?

    This section has been revised for clarity.

16. Appendix C-2 Indicate District designated MH ids rather than model node ids. Conversion table is attached.

    All MH IDs have been revised.

<u>Recycled Water</u>

1. Table 4-1 Demand factor – 2.9 for turf or 1.5 for drought tolerant planting.

   A unit demand factor of 2.9 AFY was used per comment.

2. Peak Day Peaking Factor – Where is this number from?

   The peak month factor per the 2001 IFP is 2.12. However, this assumes a 24-hour irrigation period. The District has requested that an 8-hour window for irrigation be considered for the Project, which is only one third of a day. Therefore, the 2.12 factor was multiplied by 3 to normalize the peak flow over an 8-hour window, which resulted in a peaking factor of 6.36.

3. Use the same change from the water section regarding the static range of the recycled water system.

   Revised per comment.

4. Label Street A on figure 4-1.

   Revised per comment.

5. Revise figure 1 to include irrigation areas and park locations.

   Proposed irrigated areas are shown on Figure 4-1.

We feel that the attached comments adequately address the City's comments. If you have any comments, please feel free to contact me at (858) 874-1810.

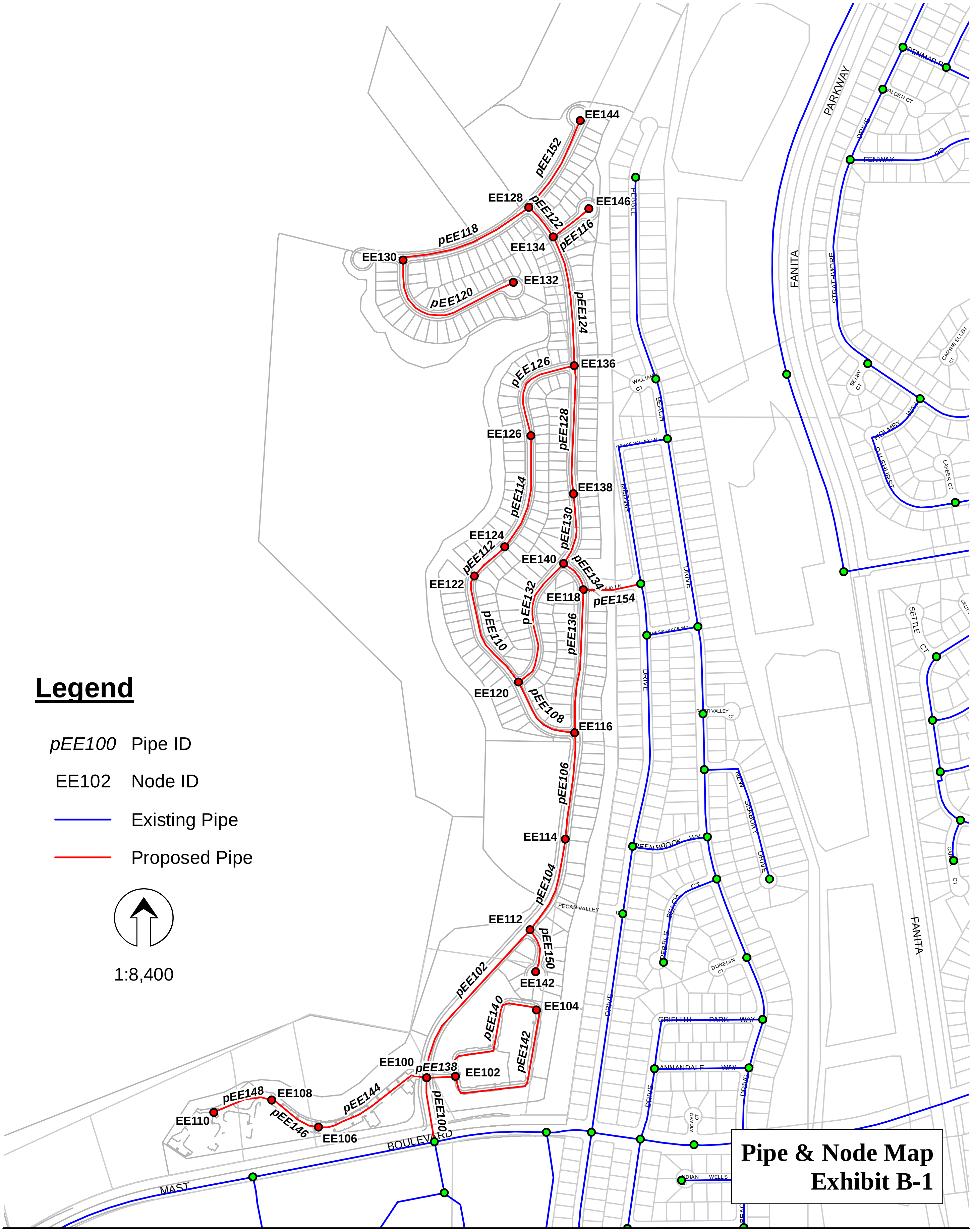
Respectfully,

Jennifer R. Bileck

Jennifer R. Bileck, P.E.
Senior Engineer

cc: Allen Kashani, Pardee Homes
John Eardensen, Latitude 33
Mark B. Elliott, PBS&J
James J. Strayer, PBS&J
Kyle O. McCarty, PBS&J
File *H:\Waterres\072 Pardee\491023 - Castlerock Wtr Swr\Corr\cm073107 response.doc*

APPENDIX B
INFOWATER OUTPUT



Pipe & Node Map
Exhibit B-1

PBSJ

Castlerock Water, Sewer, & Recycled Water Study
July 2007

8/10/07 EC KM H:\Waterres\072 Pardee\491023 - Castlerock Wtr Swr\Model\pipe node water July07.mxd

TABLE B-1a

PEAK HOUR

NODE ID	DEMAND (gpm)	ELEVATION (ft)	ZONE (ft)	STATIC PRESSURE (psi)	MODEL PRESSURE (psi)
EE100	0	446.0	629	79	76
EE102	43.79	446.0	629	79	76
EE104	43.79	452.0	629	77	73
EE106	18.43	453.5	629	76	73
EE108	18.47	452.5	629	76	73
EE110	18.47	453.0	629	76	73
EE112	0	469.5	629	69	65
EE114	13.12	450.0	629	77	74
EE116	14.2	426.0	629	88	84
EE118	18.6	409.0	629	95	91
EE120	25.12	449.0	629	78	74
EE122	18.56	461.0	629	73	69
EE124	24.04	431.0	629	86	82
EE126	25.16	430.0	629	86	82
EE128	10.92	426.0	629	88	84
EE130	18.56	441.0	629	81	77
EE132	28.44	434.0	629	84	80
EE134	15.28	420.0	629	90	86
EE136	26.24	425.0	629	88	84
EE138	24.04	415.0	629	93	89
EE140	17.48	413.0	629	94	90
EE142	14.2	454.5	629	76	72
EE144	7.64	445.0	629	80	76
EE146	0	427.0	629	87	83

TABLE B-1b

MDD + SF FIRE @ NODE EE132

PIPE ID	UPSTREAM NODE	DOWNSTREAM NODE	DIAMETER (in)	FLOW (gpm)	HEADLOSS PER 1000' (ft/kft)	VELOCITY (fps)
EE100	1252	EE100	12	314.12	0.34	0.89
EE102	EE100	EE112	8	171.18	0.81	1.09
EE104	EE112	EE114	8	156.98	0.69	1.00
EE106	EE114	EE116	8	143.86	0.58	0.92
EE108	EE116	EE120	8	100.91	0.30	0.64
EE110	EE120	EE122	8	91.99	0.26	0.59
EE112	EE122	EE124	8	73.43	0.17	0.47
EE114	EE124	EE126	8	49.38	0.08	0.32
EE116	EE134	EE146	8	0.00	0.00	0.00
EE118	EE128	EE130	8	47.00	0.07	0.30
EE120	EE130	EE132	8	28.44	0.03	0.18
EE122	EE134	EE128	10	65.57	0.05	0.27
EE124	EE136	EE134	10	80.85	0.07	0.33
EE126	EE126	EE136	8	24.23	0.02	0.15
EE128	EE136	EE138	8	-82.87	0.21	0.53
EE130	EE138	EE140	8	-106.91	0.34	0.68
EE132	EE140	EE120	8	16.21	0.01	0.10
EE134	EE140	EE118	8	-140.60	0.56	0.90
EE136	EE118	EE116	8	-28.74	0.03	0.18
EE138	EE100	EE102	12	87.58	0.03	0.25
EE140	EE102	EE104	8	22.69	0.02	0.14
EE142	EE104	EE102	8	-21.10	0.02	0.13
EE144	EE100	EE106	12	55.37	0.01	0.16
EE146	EE106	EE108	12	36.93	0.01	0.10
EE148	EE108	EE110	12	18.47	0.00	0.05
EE150	EE112	EE142	8	14.20	0.01	0.09
EE152	EE128	EE144	8	7.64	0.00	0.05
EE154	EE118	1210	10	-130.45	0.16	0.53

TABLE B-2a

MDD + SF FIRE @ NODE EE132

NODE ID	DEMAND (gpm)	ELEVATION (ft)	ZONE (ft)	STATIC PRESSURE (psi)	MODEL PRESSURE (psi)
EE100	0.0	446.0	629	79	74
EE102	27.5	446.0	629	79	74
EE104	27.5	452.0	629	77	72
EE106	11.6	453.5	629	76	71
EE108	11.6	452.5	629	76	71
EE110	11.6	453.0	629	76	71
EE112	0.0	469.5	629	69	60
EE114	8.2	450.0	629	77	66
EE116	8.9	426.0	629	88	74
EE118	11.7	409.0	629	95	81
EE120	15.8	449.0	629	78	62
EE122	11.7	461.0	629	73	54
EE124	15.1	431.0	629	86	66
EE126	15.8	430.0	629	86	64
EE128	6.9	426.0	629	88	57
EE130	11.7	441.0	629	81	35
EE132	1517.9	434.0	629	84	21
EE134	9.6	420.0	629	90	61
EE136	16.5	425.0	629	88	64
EE138	15.1	415.0	629	93	73
EE140	11.0	413.0	629	94	78
EE142	8.9	454.5	629	76	66
EE144	4.8	445.0	629	80	48
EE146	0.0	427	629	87	58

TABLE B-2b

MDD + SF FIRE @ NODE EE132

PIPE ID	UPSTREAM NODE	DOWNSTREAM NODE	DIAMETER (in)	FLOW (gpm)	HEADLOSS PER 1000' (ft/kft)	VELOCITY (fps)
EE100	1252	EE100	12	763.25	1.78	2.17
EE102	EE100	EE112	8	673.48	10.19	4.30
EE104	EE112	EE114	8	664.56	9.94	4.24
EE106	EE114	EE116	8	656.32	9.71	4.19
EE108	EE116	EE120	8	638.84	9.24	4.08
EE110	EE120	EE122	8	677.06	10.29	4.32
EE112	EE122	EE124	8	665.40	9.96	4.25
EE114	EE124	EE126	8	650.30	9.55	4.15
EE116	EE134	EE146	8	0.00	0.00	0.00
EE118	EE128	EE130	8	1529.52	46.54	9.76
EE120	EE130	EE132	8	1517.86	45.88	9.69
EE122	EE134	EE128	10	1541.18	15.92	6.30
EE124	EE136	EE134	10	1550.78	16.10	6.33
EE126	EE126	EE136	8	634.50	9.12	4.05
EE128	EE136	EE138	8	-932.76	18.62	5.95
EE130	EE138	EE140	8	-947.86	19.18	6.05
EE132	EE140	EE120	8	54.00	0.10	0.34
EE134	EE140	EE118	8	-1012.84	21.69	6.46
EE136	EE118	EE116	8	-8.56	0.00	0.05
EE138	EE100	EE102	12	55.00	0.01	0.16
EE140	EE102	EE104	8	14.25	0.01	0.09
EE142	EE104	EE102	8	-13.25	0.01	0.08
EE144	EE100	EE106	12	34.77	0.01	0.10
EE146	EE106	EE108	12	23.20	0.00	0.07
EE148	EE108	EE110	12	11.60	0.00	0.03
EE150	EE112	EE142	8	8.92	0.00	0.06
EE152	EE128	EE144	8	4.80	0.00	0.03
EE154	EE118	1210	10	-1015.96	7.36	4.15

TABLE B-3a

MDD + MF FIRE @ NODE EE104

NODE ID	DEMAND (gpm)	ELEVATION (ft)	ZONE (ft)	STATIC PRESSURE (psi)	MODEL PRESSURE (psi)
EE100	0.0	446.0	629	79	68
EE102	27.5	446.0	629	79	66
EE104	2527.5	452.0	629	77	52
EE106	11.6	453.5	629	76	64
EE108	11.6	452.5	629	76	65
EE110	11.6	453.0	629	76	64
EE112	0.0	469.5	629	69	58
EE114	8.2	450.0	629	77	67
EE116	8.9	426.0	629	88	77
EE118	11.7	409.0	629	95	85
EE120	15.8	449.0	629	78	67
EE122	11.7	461.0	629	73	62
EE124	15.1	431.0	629	86	75
EE126	15.8	430.0	629	86	76
EE128	6.9	426.0	629	88	77
EE130	11.7	441.0	629	81	71
EE132	17.9	434.0	629	84	74
EE134	9.6	420.0	629	90	80
EE136	16.5	425.0	629	88	78
EE138	15.1	415.0	629	93	82
EE140	11.0	413.0	629	94	83
EE142	8.9	454.5	629	76	64
EE144	4.8	445	630	80	69
EE146	0.0	427	631	88	77

TABLE B-3b

MDD + MF FIRE @ NODE EE104

PIPE ID	UPSTREAM NODE	DOWNSTREAM NODE	DIAMETER (in)	FLOW (gpm)	HEADLOSS PER 1000' (ft/kft)	VELOCITY (fps)
EE100	1252	EE100	12	2383.31	14.68	6.76
EE102	EE100	EE112	8	-206.47	1.14	1.32
EE104	EE112	EE114	8	-215.39	1.23	1.37
EE106	EE114	EE116	8	-223.63	1.32	1.43
EE108	EE116	EE120	8	-69.35	0.15	0.44
EE110	EE120	EE122	8	24.35	0.02	0.16
EE112	EE122	EE124	8	12.69	0.01	0.08
EE114	EE124	EE126	8	-2.41	0.00	0.02
EE116	EE134	EE146	8	0.00	0.00	0.00
EE118	EE128	EE130	8	29.52	0.03	0.19
EE120	EE130	EE132	8	17.86	0.01	0.11
EE122	EE134	EE128	10	41.18	0.02	0.17
EE124	EE136	EE134	10	50.78	0.03	0.21
EE126	EE126	EE136	8	-18.21	0.01	0.12
EE128	EE136	EE138	8	-85.47	0.22	0.55
EE130	EE138	EE140	8	-100.57	0.30	0.64
EE132	EE140	EE120	8	109.49	0.35	0.70
EE134	EE140	EE118	8	-221.03	1.29	1.41
EE136	EE118	EE116	8	163.20	0.74	1.04
EE138	EE100	EE102	12	2555.00	16.70	7.25
EE140	EE102	EE104	8	1309.73	34.92	8.36
EE142	EE104	EE102	8	-1217.77	30.51	7.77
EE144	EE100	EE106	12	34.77	0.01	0.10
EE146	EE106	EE108	12	23.20	0.00	0.07
EE148	EE108	EE110	12	11.60	0.00	0.03
EE150	EE112	EE142	8	8.92	0.00	0.06
EE152	EE128	EE144	8	4.80	0.00	0.03
EE154	EE118	1210	10	-395.91	1.28	1.62

TABLE B-4a

Impacts to Offsite Pressure

Node ID	Location	Peak Hour Pressure[1]	
		Without Castlerock	With Castlerock
1204	End of Pebble Beach Drive	101.93	100.33
1252	Connection to Mast Blvd	82.21	81.02
1334	End of Cecilwood Drive	47.65	47.41
1948	Intersection of Siwanoy Court & Pebble Beach Drive	125.19	124.13
1268	Intersection of Lake Canyon Road & Fanita Parkway	114.87	114.63

TABLE B-4b

Impacts to Offsite Piping Velocities

Pipe ID	Diameter & Location	Peak Hour Velocity[1]	
		Without Castlerock	With Castlerock
1341	14" Connection to Carlton Hills Res.	2.31	2.78
1317	12" in Mast Blvd east of connection	0.11	0.30
1343	16" from Carlton Hills Res. in Carlton Hills Blvd.	0.86	1.11
1327	12" in Mast Blvd bridge	1.07	1.78
1265	8" in Pebble Beach Drive between Cypress Lakes Way & Moana Kia Lane	0.27	0.89

1. Peak Hour conditions were taken from the District's buildout model.

APPENDIX C
SEWER SYSTEM ANALYSIS

TABLE C-1

ON-SITE SEWER STUDY SUMMARY
CASTLEROCK ALTERNATIVE 1 SEWER STUDY

Job No.: 620681.18

For: PDMWD
By: PBS&J

11/18/2008

Line No.	From MH	To MH	In-Line DU	Total DU	Average Flow (mgd)	Average Flow (cfs)	Peak/Ave Ratio	Peak Design Flow (mgd)	Peak Design Flow (cfs)	Line Size (inches)	Design Slope (%)	d (feet)	d/D	Velocity (fps)	Comments
P1	A4	A5	17	17	0.004	0.006	2.85	0.011	0.016	8	2.8	0.04	0.06	1.94	
P2	A5	A6	2	19	0.004	0.006	2.85	0.012	0.018	8	1.1	0.05	0.08	1.45	
P3	A6	A7	3	22	0.005	0.007	2.85	0.014	0.021	8	1.0	0.06	0.09	1.48	
P4	A7	G1		22	0.005	0.007	2.85	0.014	0.021	8	10.2	0.03	0.05	3.31	
P5	A3	A2	5	5	0.001	0.002	2.85	0.003	0.005	8	1.0	0.03	0.04	0.96	
P6	A2	A1	7	12	0.003	0.004	2.85	0.008	0.012	8	1.0	0.04	0.07	1.24	
P7	A1	B1	3	15	0.003	0.005	2.85	0.009	0.015	8	2.5	0.04	0.06	1.83	
P8	B1	B2	4	19	0.004	0.006	2.85	0.012	0.018	8	3.8	0.04	0.06	2.23	
P9	B2	C2	8	27	0.006	0.009	2.85	0.017	0.026	8	1.0	0.06	0.09	1.57	
P10	C1	C2	11	11	0.002	0.004	2.85	0.007	0.011	8	4.2	0.03	0.05	2.00	
P11	C2	F1	2	40	0.009	0.014	2.85	0.025	0.039	8	1.0	0.08	0.11	1.78	
P12	E1	F1	11	11	0.002	0.004	2.85	0.007	0.011	8	2.4	0.03	0.05	1.64	
P13	F1	G1	6	57	0.013	0.019	2.85	0.036	0.055	8	1.0	0.09	0.13	1.97	
P14	G1	G2	5	84	0.018	0.029	2.85	0.053	0.081	8	0.87	0.11	0.17	2.11	
P15	G2	H1	12	96	0.021	0.033	2.85	0.060	0.093	8	0.79	0.12	0.18	2.12	
P16	D1	D2	13	13	0.003	0.004	2.85	0.008	0.013	8	6.0	0.03	0.04	2.39	
P17	D2	D3	8	21	0.005	0.007	2.85	0.013	0.020	8	2.6	0.04	0.07	2.02	
P18	D3	D4	13	34	0.007	0.012	2.85	0.021	0.033	8	1.1	0.07	0.10	1.75	
P19	D4	D5	6	40	0.009	0.014	2.85	0.025	0.039	8	1.1	0.07	0.11	1.84	
P20	D5	H1	10	50	0.011	0.017	2.85	0.031	0.049	8	4.0	0.06	0.09	3.09	
P21	H1	I1	8	154	0.034	0.052	2.85	0.097	0.149	8	0.59	0.17	0.25	2.20	
P22	I1	I2	13	167	0.037	0.057	2.85	0.105	0.162	8	0.59	0.17	0.26	2.25	
P23	I2	K1	13	180	0.040	0.061	2.85	0.113	0.175	8	0.57	0.18	0.27	2.27	
P24	J1	J2	12	12	0.003	0.004	2.85	0.008	0.012	8	3.1	0.03	0.05	1.85	
P25	J2	K1	10	22	0.005	0.007	2.85	0.014	0.021	8	6.9	0.04	0.05	2.89	
P26	K1	O2	2	204	0.045	0.069	2.85	0.128	0.198	8	0.62	0.19	0.28	2.43	
P27	L1	L2	15	15	0.003	0.005	2.85	0.009	0.015	8	2.0	0.04	0.06	1.69	
P28	L2	L3	6	21	0.005	0.007	2.85	0.013	0.020	8	5.0	0.04	0.06	2.54	
P29	L3	L4	7	28	0.006	0.010	2.85	0.018	0.027	8	5.0	0.04	0.07	2.77	
P30	L4	N1		28	0.006	0.010	2.85	0.018	0.027	8	4.4	0.05	0.07	2.65	
P31	M1	M2	4	4	0.001	0.001	2.85	0.003	0.004	8	7.7	0.02	0.02	1.83	
P32	M2	M3	5	9	0.002	0.003	2.85	0.006	0.009	8	2.8	0.03	0.05	1.64	
P33	M3	N1	6	15	0.003	0.005	2.85	0.009	0.015	8	1.0	0.05	0.07	1.33	
P34	N1	N2	12	55	0.012	0.019	2.85	0.034	0.053	8	1.0	0.09	0.13	1.95	

PAGE 1 TOTAL **259 DUs**

TABLE C-1

ON-SITE SEWER STUDY SUMMARY
CASTLEROCK ALTERNATIVE 1 SEWER STUDY

For: PDMWD
By: PBS&J

Job No.: 620681.18

11/18/2008

Line	From	To	In-Line	Total	Average Flow		Peak/Ave	Peak Design Flow		Line Size	Design Slope	d	d/D	Velocity	Comments
No.	MH	MH	DU	DU	(mgd)	(cfs)	Ratio	(mgd)	(cfs)	(inches)	(%)	(feet)		(fps)	
P35	N2	O1	11	66	0.015	0.022	2.85	0.041	0.064	8	1.7	0.08	0.13	2.49	
P36	O1	O2		270	0.059	0.092	2.85	0.169	0.262	8	9.2	0.11	0.17	6.85	
P37	O2	OSN1		270	0.059	0.092	2.85	0.169	0.262	8	1.0	0.19	0.29	3.11	
P38	R1	R3	12	12	0.003	0.004	2.85	0.008	0.012	8	1.7	0.04	0.06	1.49	
P39	R3	R4		12	0.003	0.004	2.85	0.008	0.012	8	1.1	0.04	0.06	1.28	
P40	R4	R5		12	0.003	0.004	2.85	0.008	0.012	8	1.0	0.04	0.07	1.24	
P41	R5	R6		12	0.003	0.004	2.85	0.008	0.012	8	2.4	0.04	0.05	1.69	
P42	R6	R7	16	28	0.006	0.010	2.85	0.018	0.027	8	1.0	0.06	0.10	1.60	
P43	R7	R8	14	42	0.009	0.014	2.85	0.026	0.041	8	1.1	0.08	0.11	1.87	
P44	R8	T1	5	47	0.010	0.016	2.85	0.029	0.046	8	1.1	0.08	0.12	1.94	
P45	T1	V1		47	0.010	0.016	2.85	0.029	0.046	8	1.7	0.07	0.11	2.25	
P46	X1	X2	8	8	0.002	0.003	2.85	0.005	0.008	8	1.0	0.04	0.05	1.10	
P47	X2	X3	8	16	0.004	0.005	2.85	0.010	0.016	8	1.0	0.05	0.08	1.35	
P48	X3	X4	8	24	0.005	0.008	2.85	0.015	0.023	8	1.0	0.06	0.09	1.52	
P49	X4	X4A	8	32	0.007	0.011	2.85	0.020	0.031	8	1.0	0.07	0.10	1.66	
P50	X4A	X5	8	40	0.009	0.014	2.85	0.025	0.039	8	1.0	0.08	0.11	1.78	
P51	X5	X6		40	0.009	0.014	2.85	0.025	0.039	8	1.0	0.08	0.11	1.78	
P52	X6	X7		40	0.009	0.014	2.85	0.025	0.039	8	1.0	0.08	0.11	1.78	
P53	X7	X8	8	48	0.011	0.016	2.85	0.030	0.047	8	1.0	0.08	0.12	1.88	
P54	X8	X9	7	55	0.012	0.019	2.85	0.034	0.053	8	1.0	0.09	0.13	1.95	
P55	X9	V1		55	0.012	0.019	2.85	0.034	0.053	8	1.0	0.09	0.13	1.95	
P56	V1	V3		102	0.022	0.035	2.85	0.064	0.099	8	3.2	0.09	0.13	3.54	
P57	S1	S2	22	22	0.005	0.007	2.85	0.014	0.021	8	3.0	0.04	0.07	2.15	Private Sewer
P58	S2	S3	21	43	0.009	0.015	2.85	0.027	0.042	8	1.0	0.08	0.12	1.82	Private Sewer
P59	S3	S4	7	50	0.011	0.017	2.85	0.031	0.049	8	1.0	0.08	0.13	1.91	Private Sewer
P60	S4	S5	2	52	0.011	0.018	2.85	0.033	0.050	8	1.0	0.09	0.13	1.92	Private Sewer
P61	S5	V3		52	0.011	0.018	2.85	0.033	0.050	8	1.0	0.09	0.13	1.92	Private Sewer
P62	V3	OS1		154	0.034	0.052	2.85	0.097	0.149	8	7.5	0.09	0.13	5.39	

PAGE 2 TOTAL **165 DUs**

PAGE 1 TOTAL **259 DUs**

DEVELOPMENT TOTAL **424 DUs**

TABLE C-2

EXISTING OFF-SITE SEWER STUDY SUMMARY
CASTLEROCK ALTERNATIVE 1 SEWER STUDY

For: PDMWD
By: PBS&J

Job No.: 620681.18

Line No.	From MH	To MH	Castlerock Peak Flow (mgd)	Castlerock Peak Flow (cfs)	2020 DW Peak Flow (mgd)	2020 DW Peak Flow (cfs)	Material Type	Peak Design Flow (mgd)	Peak Design Flow (cfs)	Line Size (inches)	Design Slope (%)	d (feet)	d/D	Velocity (fps)	Comments
P1	OSN1	1703	0.169	0.261	-	-	PVC	0.169	0.261	8	1.52%	0.17	0.26	3.62	New 8"PVC
P2	1703	1702	0.169	0.261	0.004	0.006	PVC	0.173	0.268	8	1.00%	0.20	0.29	3.13	Replace Exist 6" VCP w/ 8"PVC
P3	1702	1701	0.169	0.261	0.007	0.011	PVC	0.176	0.272	8	0.70%	0.22	0.32	2.77	Replace Exist 6" VCP w/ 8"PVC
P4	1701	1697	0.169	0.261	0.009	0.014	VCP	0.178	0.275	8	3.68%	0.16	0.23	4.46	
P5	1697	1698	0.169	0.261	0.032	0.050	VCP	0.201	0.311	8	0.40%	0.30	0.44	2.08	
P6	1698	1699	0.169	0.261	0.038	0.059	VCP	0.207	0.320	8	0.40%	0.30	0.45	2.09	
P7	1699	1700	0.169	0.261	0.042	0.065	VCP	0.211	0.326	8	1.11%	0.23	0.35	3.05	
P8	1700	1624	0.169	0.261	0.045	0.070	VCP	0.214	0.331	8	0.40%	0.31	0.46	2.11	
P9	1624	1625	0.169	0.261	0.063	0.097	VCP	0.232	0.359	8	0.40%	0.32	0.48	2.16	
P10	1625	1626	0.169	0.261	0.066	0.102	VCP	0.235	0.364	8	0.40%	0.32	0.49	2.16	
P11	1626	1627	0.169	0.261	0.071	0.110	TECH	0.240	0.371	8	0.40%	0.30	0.45	2.46	
P12	1627	1558	0.169	0.261	0.076	0.118	TECH	0.245	0.379	8	0.40%	0.30	0.45	2.48	
P13	1558	1559	0.169	0.261	0.089	0.138	TECH	0.258	0.399	8	0.40%	0.31	0.47	2.51	
P14	1559	1296	0.169	0.261	0.099	0.153	TECH	0.268	0.415	8	0.40%	0.32	0.48	2.53	
P15	1296	1295	0.169	0.261	0.112	0.173	TECH	0.281	0.435	8	0.40%	0.33	0.49	2.56	
P16	1295	1294	0.169	0.261	0.118	0.183	TECH	0.287	0.444	8	0.40%	0.33	0.50	2.58	
P17	1294	1293	0.169	0.261	0.127	0.197	VCP	0.296	0.458	8	0.40%	0.37	0.56	2.29	
P18	1293	1292	0.169	0.261	0.138	0.214	VCP	0.307	0.475	8	0.40%	0.38	0.57	2.31	
P19	1292	1291	0.169	0.261	0.140	0.217	VCP	0.309	0.478	8	0.29%	0.42	0.64	2.04	
P20	OS1	2172	0.097	0.150	-	-	PVC	0.097	0.150	8	1.34%	0.14	0.20	2.94	New 8"PVC
P21	2172	1424	0.097	0.150	0.017	0.026	PVC	0.114	0.176	8	11.79%	0.09	0.13	6.64	Replace Exist 6" VCP w/ 8"PVC
P22	1424	1289	0.097	0.150	0.037	0.057	PVC	0.134	0.207	8	2.29%	0.14	0.21	3.91	Replace Exist 6" VCP w/ 8"PVC
P23	1289	1290	0.097	0.150	0.037	0.057	PVC	0.134	0.207	8	4.75%	0.12	0.18	5.06	New 8"PVC
P24	1290	1291	0.097	0.150	0.039	0.060	PVC	0.136	0.210	8	4.85%	0.12	0.18	5.12	Replace Exist 6" VCP w/ 8"PVC
P25	1291	746	0.266	0.412	0.179	0.277	VCP	0.445	0.689	8	0.48%	0.46	0.69	2.68	
P26	746	744	0.266	0.412	0.184	0.285	VCP	0.450	0.696	8	0.34%	0.54	0.81	2.30	d/D > 0.75
P26	746	744					LINE			7.52		0.46	0.74	2.85	Slip-lined
P26	746	744					PVC			8		0.47	0.70	2.67	Replace w/ 8" PVC
P27	744	741	0.266	0.412	0.198	0.306	VCP	0.464	0.718	8	0.43%	0.50	0.75	2.57	
P28	741	739	0.266	0.412	0.205	0.317	VCP	0.471	0.729	8	2.47%	0.29	0.43	5.08	
P29	739	734	0.266	0.412	0.230	0.356	VCP	0.496	0.767	8	1.46%	0.34	0.52	4.23	
P30	734	457	0.266	0.412	0.237	0.367	VCP	0.503	0.778	8	2.01%	0.32	0.47	4.79	
P31	457	456	0.266	0.412	0.286	0.443	VCP	0.552	0.854	8	1.20%	0.39	0.59	4.03	
P32	456	444	0.266	0.412	0.289	0.447	VCP	0.555	0.859	8	1.44%	0.37	0.55	4.33	
P33	444	445	0.266	0.412	0.326	0.504	VCP	0.592	0.916	8	2.08%	0.34	0.52	5.05	
P34	445	50	0.266	0.412	0.328	0.508	VCP	0.594	0.919	8	2.18%	0.34	0.51	5.15	
P35	50	583	0.266	0.412	0.711	1.100	VCP	0.977	1.512	15	0.10%	0.80	0.64	1.82	
P36	583	3001	0.266	0.412	0.717	1.109	VCP	0.983	1.521	15	0.20%	0.64	0.52	2.38	
P37	3001	2987	0.266	0.412	3.593	5.560	VCP	3.859	5.971	30	0.04%	1.58	0.63	1.82	
P38	2987	2687	0.266	0.412	7.500	11.605	VCP	7.766	12.017	30	0.24%	1.39	0.56	4.28	
P39	2687	IPS	0.266	0.412	7.500	11.605	VCP	7.766	12.017	30	0.10%	1.90	0.76	3.00	d/D > 0.75

Table C-3
Alternative 1 Manhole Elevations

Manhole ID	RIM Elevation (Feet)	Invert Elevation (Feet)	Depth of Cover (Feet)	Notes
A4	444.0	439.0	5.0	
A5	432.0	427.0	5.0	
A6	432.0	426.5	5.5	
A7	434.5	425.2	9.3	
A3	443.7	438.7	5.0	
A2	443.1	438.0	5.1	
A1	441.2	436.0	5.2	
B1	438.0	433.0	5.0	
B2	424.0	419.0	5.0	
C1	444.0	439.0	5.0	
C2	422.0	415.1	6.9	
E1	424.0	419.0	5.0	
F1	418.0	413.0	5.0	
G1	419.7	410.9	8.8	
G2	421.4	409.7	11.7	
D1	457.5	452.5	5.0	
D2	436.8	431.8	5.0	
D3	429.7	424.7	5.0	
D4	426.4	421.0	5.4	
D5	425.5	419.3	6.2	
H1	424.1	406.6	17.5	
I1	420.7	404.8	15.9	
I2	416.3	402.4	13.9	
J1	439.4	434.4	5.0	
J2	430.4	425.4	5.0	
K1	411.7	400.1	11.6	
L1	461.5	456.5	5.0	
L2	453.8	448.8	5.0	
L3	439.0	434.0	5.0	
L4	428.5	423.5	5.0	
M1	449.6	444.6	5.0	
M2	433.4	428.4	5.0	
M3	422.6	417.6	5.0	
N1	418.9	413.9	5.0	
N2	418.7	410.2	8.5	
O1	410.6	404.6	6.0	
O2	412.7	399.7	13.0	
OSN1	388.0	380.8	7.2	North Off-site
R1	460.8	455.3	5.5	
R3	454.8	449.8	5.0	
R4	454.7	448.4	6.3	
R5	454.3	448.0	6.3	
R6	447.0	442.0	5.0	
R7	448.8	440.1	8.7	
R8	448.9	439.5	9.4	
T1	447.5	438.7	8.8	
X1	456.0	450.4	5.6	
X2	455.5	449.3	6.2	
X3	454.5	448.6	5.9	
X4	454.0	447.2	6.8	
X4A	454.5	446.6	7.9	
X5	455.0	445.9	9.1	
X6	457.0	443.7	13.3	
X7	458.0	442.0	16.0	
X8	456.0	439.5	16.5	
X9	448.0	436.7	11.3	
V1	445.9	435.9	10.0	
S1	452.9	447.9	5.0	Private
S2	440.0	436.0	4.0	Private
S3	445.0	432.6	12.4	Private
S4	443.9	432.2	11.7	Private
S5	443.9	431.7	12.2	Private
V3	443.9	430.3	13.6	
OS1	427.0	420.0	7.0	SOUTH ON-SITE

TABLE C-4

SEWER STUDY SUMMARY
CASTLEROCK ALTERNATIVE 2 SEWER STUDY

For: PDMWD
By: PBS&J

Job No.: 620681.18

Line	From	To	In-Line	Total	Average Flow		Peak/Ave	Peak Design Flow		Line Size	Design Slope	d	d/D	Velocity	Comments
No.	MH	MH	DU	DU	(mgd)	(cfs)	Ratio	(mgd)	(cfs)	(inches)	(%)	(feet)		(fps)	
P1	A4	A5	17	17	0.004	0.006	2.85	0.011	0.016	8	2.8	0.04	0.06	1.97	
P2	A5	A6	2	19	0.004	0.006	2.85	0.012	0.018	8	1.1	0.05	0.08	1.46	
P3	A6	A7	3	22	0.005	0.007	2.85	0.014	0.021	8	1.0	0.06	0.09	1.49	
P4	A7	G1		22	0.005	0.007	2.85	0.014	0.021	8	10.2	0.03	0.05	3.34	
P5	A3	A2	5	5	0.001	0.002	2.85	0.003	0.005	8	1.0	0.03	0.04	0.95	
P6	A2	A1	7	12	0.003	0.004	2.85	0.008	0.012	8	1.0	0.04	0.07	1.25	
P7	A1	B1	3	15	0.003	0.005	2.85	0.009	0.015	8	2.5	0.04	0.06	1.79	
P8	B1	B2	4	19	0.004	0.006	2.85	0.012	0.018	8	3.8	0.04	0.06	2.25	
P9	B2	C2	8	27	0.006	0.009	2.85	0.017	0.026	8	1.0	0.06	0.09	1.57	
P10	C1	C2	11	11	0.002	0.004	2.85	0.007	0.011	8	4.2	0.03	0.04	2.00	
P11	C2	F1	2	40	0.009	0.014	2.85	0.025	0.039	8	1.0	0.08	0.11	1.77	
P12	E1	F1	11	11	0.002	0.004	2.85	0.007	0.011	8	2.4	0.03	0.05	1.64	
P13	F1	G1	6	57	0.013	0.019	2.85	0.036	0.055	8	1.0	0.09	0.13	1.98	
P14	G1	G2	5	84	0.018	0.029	2.85	0.053	0.081	8	0.87	0.11	0.17	2.12	
P15	G2	H1	12	96	0.021	0.033	2.85	0.060	0.093	8	0.79	0.12	0.18	2.12	
P16	D1	D2	13	13	0.003	0.004	2.85	0.008	0.013	8	6.0	0.03	0.04	2.36	
P17	D2	D3	8	21	0.005	0.007	2.85	0.013	0.020	8	2.6	0.04	0.07	2.02	
P18	D3	D4	13	34	0.007	0.012	2.85	0.021	0.033	8	1.1	0.07	0.10	1.74	
P19	D4	D5	6	40	0.009	0.014	2.85	0.025	0.039	8	1.1	0.07	0.11	1.83	
P20	D5	H1	10	50	0.011	0.017	2.85	0.031	0.049	8	4.0	0.06	0.09	3.07	
P21	H1	I1	8	154	0.034	0.052	2.85	0.097	0.149	8	0.59	0.17	0.25	2.20	
P22	I1	I2	13	167	0.037	0.057	2.85	0.105	0.162	8	0.59	0.17	0.26	2.25	
P23	I2	K1	13	180	0.040	0.061	2.85	0.113	0.175	8	0.57	0.18	0.27	2.27	
P24	J1	J2	12	12	0.003	0.004	2.85	0.008	0.012	8	3.1	0.03	0.05	1.86	
P25	J2	K1	10	22	0.005	0.007	2.85	0.014	0.021	8	6.9	0.04	0.05	2.91	
P26	K1	O1	2	204	0.045	0.069	2.85	0.128	0.198	8	0.5	0.20	0.30	2.25	
P27	L1	L2	15	15	0.003	0.005	2.85	0.009	0.015	8	2.0	0.04	0.06	1.65	
P28	L2	L3	6	21	0.005	0.007	2.85	0.013	0.020	8	5.0	0.04	0.06	2.54	
P29	L3	L4	7	28	0.006	0.010	2.85	0.018	0.027	8	5.0	0.04	0.07	2.80	
P30	L4	N1		28	0.006	0.010	2.85	0.018	0.027	8	4.4	0.05	0.07	2.68	
P31	M1	M2	4	4	0.001	0.001	2.85	0.003	0.004	8	7.7	0.02	0.03	1.91	
P32	M2	M3	5	9	0.002	0.003	2.85	0.006	0.009	8	2.8	0.03	0.05	1.65	
P33	M3	N1	6	15	0.003	0.005	2.85	0.009	0.015	8	1.0	0.05	0.07	1.30	
P34	N1	N2	12	55	0.012	0.019	2.85	0.034	0.053	8	1.0	0.09	0.13	1.95	
P35	N2	O1	11	66	0.015	0.022	2.85	0.041	0.064	8	3.5	0.07	0.11	3.19	

PAGE 1 TOTAL **270 DUs**

TABLE C-4

SEWER STUDY SUMMARY
CASTLEROCK ALTERNATIVE 2 SEWER STUDY

For: PDMWD
By: PBS&J

Job No.: 620681.18

Line No.	From MH	To MH	In-Line DU	Total DU	Average Flow (mgd)	Average Flow (cfs)	Peak/Ave Ratio	Peak Design Flow (mgd)	Peak Design Flow (cfs)	Line Size (inches)	Design Slope (%)	d (feet)	d/D	Velocity (fps)	Comments
P36	O1	PS		270	0.059	0.092	2.85	0.169	0.262	8	0.57	0.22	0.34	2.55	
P37	PS	Q1		270	0.059	0.092	2.85	**0.220**	0.340	8	N/A	0.3	0.4	2.4	SPS - see Force Main Calc
P38	Q1	R1		270	0.059	0.092	2.85	0.220	0.340	8	1.0	0.22	0.33	3.36	Includes Pumped Flow
P39	R1	R3	12	282	0.062	0.096	2.85	0.228	0.352	8	1.7	0.20	0.30	4.10	Includes Pumped Flow
P40	R3	R4		282	0.062	0.096	2.85	0.228	0.352	8	1.1	0.22	0.33	3.51	Includes Pumped Flow
P41	R4	R5		282	0.062	0.096	2.85	0.228	0.352	8	1.0	0.23	0.34	3.39	Includes Pumped Flow
P42	R5	R6		282	0.062	0.096	2.85	0.228	0.352	8	2.4	0.18	0.27	4.64	Includes Pumped Flow
P43	R6	R7	16	298	0.066	0.101	2.85	0.238	0.368	8	1.0	0.23	0.35	3.43	Includes Pumped Flow
P44	R7	R8	14	312	0.069	0.106	2.85	0.246	0.381	8	1.1	0.23	0.34	3.58	Includes Pumped Flow
P45	R8	T1	5	317	0.070	0.108	2.85	0.249	0.386	8	1.4	0.22	0.32	3.92	Includes Pumped Flow
P46	S1	S2	22	22	0.005	0.007	2.85	0.014	0.021	8	3.0	0.04	0.07	2.17	Private Sewer
P47	S2	S3	21	43	0.009	0.015	2.85	0.027	0.042	8	1.0	0.08	0.12	1.82	Private Sewer
P48	S3	S4	7	50	0.011	0.017	2.85	0.031	0.049	8	1.0	0.08	0.13	1.90	Private Sewer
P49	S4	S5	2	52	0.011	0.018	2.85	0.033	0.050	8	1.0	0.09	0.13	1.93	Private Sewer
P50	S5	V3		52	0.011	0.018	2.85	0.033	0.050	8	1.0	0.09	0.13	1.93	Private Sewer
P51	T1	V1		317	0.070	0.108	2.85	0.249	0.386	8	1.0	0.24	0.36	3.47	Includes Pumped Flow
P52	V1	V3		317	0.070	0.108	2.85	0.249	0.386	8	3.7	0.17	0.25	5.55	Includes Pumped Flow
P53	V3	OS1		369	0.081	0.126	2.85	0.282	0.436	8	0.52	0.30	0.45	2.83	Includes Pumped Flow
P54	OS1	OS2	14	383	0.084	0.130	2.85	0.291	0.450	8	0.53	0.31	0.46	2.87	Includes Pumped Flow
P55	OS2	OS3		383	0.084	0.130	2.85	0.291	0.450	8	1.3	0.24	0.36	3.98	Includes Pumped Flow
P56	OS3	OS4	27	410	0.090	0.140	2.85	0.308	0.476	8	1.6	0.23	0.35	4.36	Includes Pumped Flow
P57	OS4	OS5	14	424	0.093	0.144	2.85	0.317	0.490	8	1.0	0.27	0.40	3.71	Includes Pumped Flow
P58	OS5	OS6		424	0.093	0.144	2.85	0.317	0.490	8	0.65	0.30	0.46	3.17	Includes Pumped Flow
P59	OS6	OS7		424	0.093	0.144	2.85	0.317	0.490	8	0.50	0.33	0.49	2.87	Includes Pumped Flow
P60	OS7	OS8		424	0.093	0.144	2.85	0.317	0.490	8	0.50	0.33	0.49	2.87	Includes Pumped Flow
P61	OS8	OS9		424	0.093	0.144	2.85	0.317	0.490	8	3.7	0.19	0.29	5.96	Includes Pumped Flow
P62	OS9	OS10		424	0.093	0.144	2.85	0.317	0.490	8	5.3	0.17	0.26	6.76	Includes Pumped Flow
P63	OS10	MGTS		424	0.093	0.144	2.85	0.317	0.490	8	0.5	0.33	0.49	2.87	Includes Pumped Flow

PAGE 2 TOTAL **154 DUs**

PAGE 1 TOTAL **270 DUs**

DEVELOPMENT TOTAL **424 DUs**

Table C-5
Alternative 2 Manhole Elevations

Manhole ID	RIM Elevation (Feet)	Invert Elevation (Feet)	Depth of Cover (Feet)	Notes
A4	444.0	439.0	5.0	
A5	432.0	427.0	5.0	
A6	432.0	426.5	5.5	
A7	434.5	425.2	9.3	
A3	443.7	438.7	5.0	
A2	443.1	438.0	5.1	
A1	441.2	436.0	5.2	
B1	438.0	433.0	5.0	
B2	424.0	419.0	5.0	
C1	444.0	439.0	5.0	
C2	422.0	415.1	6.9	
E1	424.0	419.0	5.0	
F1	418.0	413.0	5.0	
G1	419.7	410.9	8.8	
G2	421.4	409.7	11.7	
D1	457.5	452.5	5.0	
D2	436.8	431.8	5.0	
D3	429.7	424.7	5.0	
D4	426.4	421.0	5.4	
D5	425.5	419.3	6.2	
H1	424.1	406.6	17.5	
I1	420.7	404.8	15.9	
I2	416.3	402.4	13.9	
J1	439.4	434.4	5.0	
J2	430.4	425.4	5.0	
K1	411.7	400.1	11.6	
L1	461.5	456.5	5.0	
L2	453.8	448.8	5.0	
L3	439.0	434.0	5.0	
L4	428.5	423.5	5.0	
M1	449.6	444.6	5.0	
M2	433.4	428.4	5.0	
M3	422.6	417.6	5.0	
N1	418.9	413.9	5.0	
N2	418.7	410.2	8.5	
O1	410.6	398.7	11.9	
PS	414	398.4	15.6	Pump Station
Q1	461.9	456.9	5.0	
R1	460.8	455.3	5.5	
R3	454.8	449.8	5.0	
R4	454.7	448.4	6.3	
R5	454.3	448.0	6.3	
R6	447.0	442.0	5.0	
R7	448.8	440.1	8.7	
R8	448.9	439.5	9.4	
S1	452.9	447.9	5.0	Private
S2	440.0	436.0	4.0	Private
S3	445.0	432.6	12.4	Private
S4	443.9	432.2	11.7	Private
S5	443.9	431.7	12.2	Private
T1	447.5	438.5	9.0	
V1	445.9	436.8	9.1	
V3	437.8	430.3	7.5	
OS1	435.2	429.8	5.4	
OS2	435.7	427.7	8.0	
OS3	427.5	422.5	5.0	
OS4	421.3	416.3	5.0	
OS5	416.3	412.3	4.0	
OS6	413.7	409.7	4.0	
OS7	420.9	407.7	13.2	
OS8	415.8	405.7	10.1	
OS9	395.8	390.8	5.0	
OS10	374.8	369.8	5.0	

Calculated by:	KM
Checked by:	JS
Date:	7/11/2007

Castlerock - Sewer Pump Station
Design Flow Rate Calculation

PURPOSE

Determine the Required Design Pump Flow Rate for the Proposed Sewer Pump Station.

ABBREVIATIONS

DU - Dwelling Unit
EDU - Equivalent Dwelling Unit
fps - feet per second
gpd - gallons per day
mgd - million gallons per day
gpm - gallons per minute
Pop - Population
Pop Gen Rate - Average Sewage generation for 1 person
EDU Gen Rate - Average Sewage generation for 1 EDU

CRITERIA

Equivalent Dwelling Units (EDU) =	270 DU	Per Tentative Map, Latitude 33
EDU Generation Rate =	220 gpd/EDU	District's IFP
Dry Weather Peaking Factor (DWPF) =	2.85	District Criteria for Population = 500, Conservative
Wet Weather Peaking Factor (WWPF) =	1.30	Conservative for PVC pipe oustide of a floodplain
Pump Station Reserve Capacity Factor (PSRCF) =	1.00	2 hours of emergency storage will be provided

PROCEDURE

1. Calculate the Average Daily Flow (ADF)
2. Calculate the Peak Dry Weather Flow (PDWF)
3. Calculate the Peak Wet Weather Flow (PWWF)
4. Calculate te Pump Design Flow Rate (Q_D)

EQUATIONS

1. ADF = EDU x EDU Gen Rate
2. PDWF = ADF x DWPF
3. PWWF = PDWF x WWPF
4. Q_D = PWWF x PSRCF

CALCULATIONS

1. ADF = 270 DU x 220 gpd/EDU =	59,400 gpd
2. PDWF = 59,400 gpd x 2.85 =	169,290 gpd
3. PWWF = 169,290 gpd x 1.3 =	220,077 gpd
4. Q_D = 220,077 gpd x 1.0 / 10^6 gpd/mgd =	0.22 mgd
4. Q_D = 220,077 gpd x 1.0 / 1440 gpd/gpm =	153 gpm

USE - 155 gpm (0.22 mgd) as Design Point

PBS&J

Calculated by:	KM
Checked by:	JS
Date:	7/11/2007

Castlerock Sewer Study - Sewer Pump Station
Force Main Sizing Calculation

PURPOSE

Select the force main diameter for the Proposed Sewer Pump Station.

ABBREVIATIONS

EDU - Equivalent Dwelling Unit
fps - feet per second
gpd - gallons per day
mgd - million gallons per day
cfs - cubic feet per second

CRITERIA

Design Pump Rate (Q_D) =	0.22 mgd	Taken from Design Pump Rate Calculation
Minimum Diameter for Solids Handiling Pump =	4 inch	Minimum Diameter for Solids handeling capability
Inside Diameter of 4-inch Class 200 PVC Pipe (D) =	4.07 inch	JM Pipe Catalog
Hazen Williams Roughness Coefficient (C) =	130	Conservative for Class 200 PVC
Length of Force main (L) =	1,865.0 feet	Taken from schematic layout
Minimum Velocity for Sewer Force main =	3.5 fps	Conservative
Maximum Velocity for Sewer Force main =	8.0 fps	Conservative

PROCEDURE

1. Calculate the Force main Velocity for a 4-inch Diameter Pipe
2. Confirm that the Velocity meets City Criteria
3. Calculate the Head Loss for 4-inch Diameter Pipe

EQUATIONS

1. $V = Q_D$ / ((PI/4)(4.07"/12"/ft)^2)
2. H_L = 3.02 L (D ^ -1.167) (V/C) ^ 1.85

CALCULATIONS

1. V = (0.25 mgd x 1.547 cfs/mgd) / ((PI/4)(4.07"/12"/ft)^2) =	3.8 fps	Conforms
2. H_L = 3.02 x 1,865 ft X ((4.07"/12"/ft) ^ -1.167) (4.28 fps/130) ^ 1.85 =	28.4 feet	

APPENDIX D
WATER STORAGE CALCULATIONS

PBSJ

9275 Sky Park Court, Suite 200
San Diego, CA 92123

PROJECT Castlerock - Water
CLIENT
CALCULATED BY J Bileck DATE 7·25·07
CHECKED BY DATE
PROJECT NO SHT 1 OF 1

Per 2001 IFP Table 5-11:

Ultimate 2020 Storage Required = 19.31 MG
Existing Storage = 17.6 MG

$\Big\}$ 1.71 MG Deficit

CIP #W1014 would build a 1.71 MG Res @ Mesa Rd.

Old criteria included full fire storage:
3,500 gpm x 5 hrs = 1.05 MG

New criteria only requires ½ : 1.05/2 = 0.525 MG extra

1.71 − 0.525 = 1.185 MG Revised Mesa Res

Castlerock Storage Required:

ADD x 200% = 0.20 MG x 200% = 0.40 MG

1.185 + 0.40 = $\boxed{1.585 \text{ MG}}$

* New Mesa Reservoir should have a minimum volume of 1.585 MG. Timing of this facility will be determined by Padre Dam based on the acceleration of growth in the Gravity Zone.