FINAL REPORT

SEWER COLLECTION SYSTEM MASTER PLAN UPDATE

Submitted to

City of Morro Bay
Department of Public Services

WALLACE GROUP

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May 2006

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EXECUTIVE SUMMARY

This report presents the Wastewater Collection System Master Plan Update for the City of Morro Bay. It is the City of Morro Bay's intent to construct wastewater system improvements consistent with the current and ultimate needs of the City. In order to facilitate this goal, and to adequately plan for the capital resources needed to meet this goal, the City has elected to update an existing Wastewater Master Plan prepared by CH2MHILL in 1986.

Preparation of a wastewater systems master plan will assist the City in prioritizing both present and future wastewater system needs and set forth a mechanism for addressing those needs through the next 20 years. The master planning process will tie the needs assessment, both existing and future, to the budgeting process.

In January 2004, the City authorized Wallace Group to prepare this comprehensive wastewater system master plan update. This plan was originally anticipated to be completed in Summer 2004. However, due to a very dry 2003/2004 winter and the need to capture additional sewer flow monitoring data, the report schedule was extended to 2005 in order to capture the 2004/2005 rainfall. This wastewater master plan is prepared in accordance with Wallace Group's proposal dated December 18, 2003, and includes analyses of the City's wastewater flows, collection system capacity, evaluation of lift stations; and a prioritized capital improvement program.

This master plan update is presented in seven chapters, summarized as follows:

- **Chapter 1, Introduction**. This chapter presents an overview of the goals of this Report, authorization and scope of work, and acknowledgment of the various staff and personnel involved in the preparation of this document.
- **Chapter 2, Demographics**. This chapter focuses on the City's General Plan Update, existing and future population projections, occupancy factors, land uses, and other considerations that will be pertinent to projecting the City's existing and future wastewater flow characteristics. Existing population is approximately 10,500. Future build-out population is projected at 12,500, with an occupancy factor of 80%. It is estimated there are approximately 200 infill lots to be developed, and approximately 300 medium to high-density new developments slated for the City. There will be minor commercial development in the future; however, it is noted that there is a potential for a number of mixed-use units (residential units on top of commercial units) in the future.
- **Chapter 3, Wastewater Flows**. This chapter provides an analysis and summary of the City's existing and future wastewater flow characteristics, based on planning/demographic information presented in Chapter 2. These wastewater flows form the basis of recommendations for recommended capital improvements in the collection system. The City's build-out wastewater flow is projected at 1.0 million gallons per day (mgd).
- **Chapter 4, Inflow and Infiltration**. This chapter presents details of the two seasons of inflow/infiltration flow monitoring conducted throughout the City's collection system, to identify problem areas of wet weather flow in the City's collection system and the joint wastewater treatment plant.
- **Chapter 5, Collection System Analysis**. This chapter presents the modeling and hydraulic analysis of the City's collection system. All gravity sewers, 8" in diameter and larger (up to 27" diameter), were hydraulically modeled and evaluated under

existing and future flow conditions, dry weather and wet weather flow conditions. The City's collection system is divided into 11 distinct drainage basins, approximately 50 miles of gravity sewers, and three sewage lift stations. These basins, the lift stations, and the 11 flow monitoring stations (from the flow monitoring investigation described in Chapter 4), is included as Figure ES-1. A graphic portrayal of how the drainage basins and lift stations relate to one another are included as Figure ES-2. Those gravity sewers, 8" diameter and larger that were included in this modeling analysis, are shown on Figure ES-3.

- **Chapter 6, Evaluation of Sewage Lift Stations**. This chapter presents the detailed evaluation of the City's three sewage lift stations. The lift stations were evaluated based on hydraulic capacity considerations, and non-hydraulic issues relating to operations.
- **Chapter 7, Capital Improvement Program**. Chapter 7 presents the capital improvement program (CIP), which ties the identified near-term and future collection system improvements for the collection system to needed capital costs. This CIP will be used by the City as a strategic planning tool to plan for and forecast needed capital budgets for anticipated collection system improvements.

CAPITAL IMPROVEMENT PROGRAM

The capital improvement program (CIP) costs were developed based on engineering judgment, confirmed bid prices for similar work in the Central Coast area, consultation with vendors and contractors, established budgetary unit prices for the work, and other reliable sources. Hard construction costs are escalated by a factor of 1.4, to allow budget for "soft costs" that include preliminary engineering, engineering, administration, construction management and inspection costs. All CIP costs are expressed in Year 2005 dollars, using an ENR Construction Cost Index of **7407**, and will need to be escalated to the year or years scheduled for the work. The unit cost for new gravity sewers includes the proposed pipelines, manholes, lateral re-connections, sewer bypassing, traffic control, etc., and all other aspects of sewer system construction.

The recommended capital improvement projects were identified based on: 1) two seasons of sewer in-line flow monitoring, to assess flow conditions during wet weather conditions; 2) detailed evaluation of the collection system by flow calculations, projections and modeling; and 3) detailed review of the City's three lift stations. Table ES-1 provides a summary of near-term and future recommended capital improvements recommended in this Report.

TIMING OF RECOMMENDED IMPROVEMENTS

The timing of recommended improvements that are triggered by future development and growth is always difficult to ascertain. For the purposes of this report, the timing of future recommendations will be expressed in terms of equivalent dwelling units (EDUs) coming on line to the City's collection system and thus inducing the need for the specified improvements. For example, a project may be recommended to be in place when an additional 100 equivalent units are developed. Based on a household density of 2.02 and a per capita wastewater demand projected at 80 gallons per capita per day (gpcd), an equivalent flow would be calculated at 16,160 gpd (11.2 gpm). Conversely, if historic trends for development indicate new housing development at 60 units per year, one might expect this development to occur with the next 18 to 24 months. Given this timing, the City would then need to anticipate this timing, and plan for the CIP accordingly, well enough in advance

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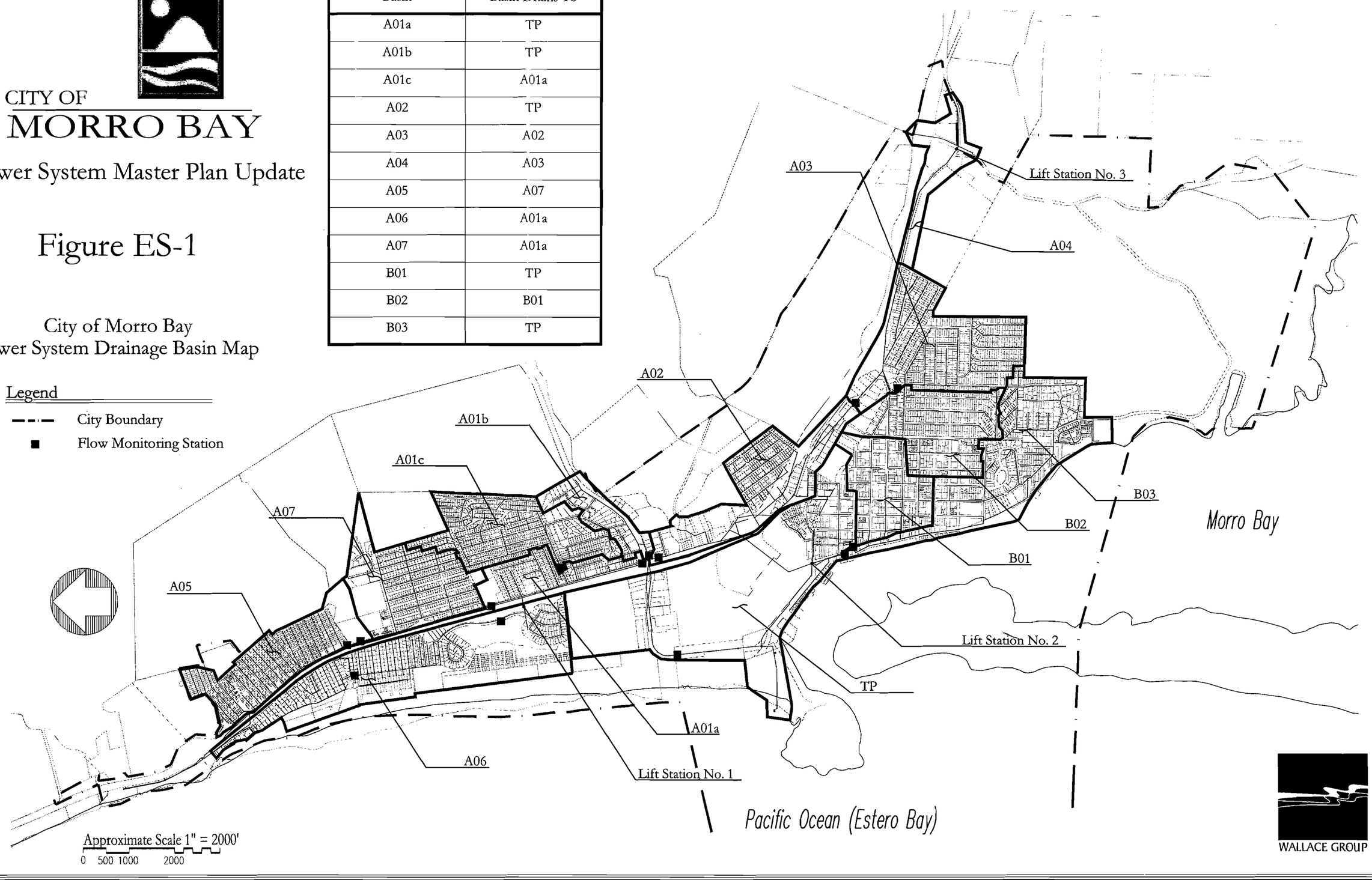
Sewer System Master Plan Update

Figure ES-1

City of Morro Bay
Sewer System Drainage Basin Map

Basin	Basin Drains To
A01a	TP
A01b	TP
A01c	A01a
A02	TP
A03	A02
A04	A03
A05	A07
A06	A01a
A07	A01a
B01	TP
B02	B01
B03	TP



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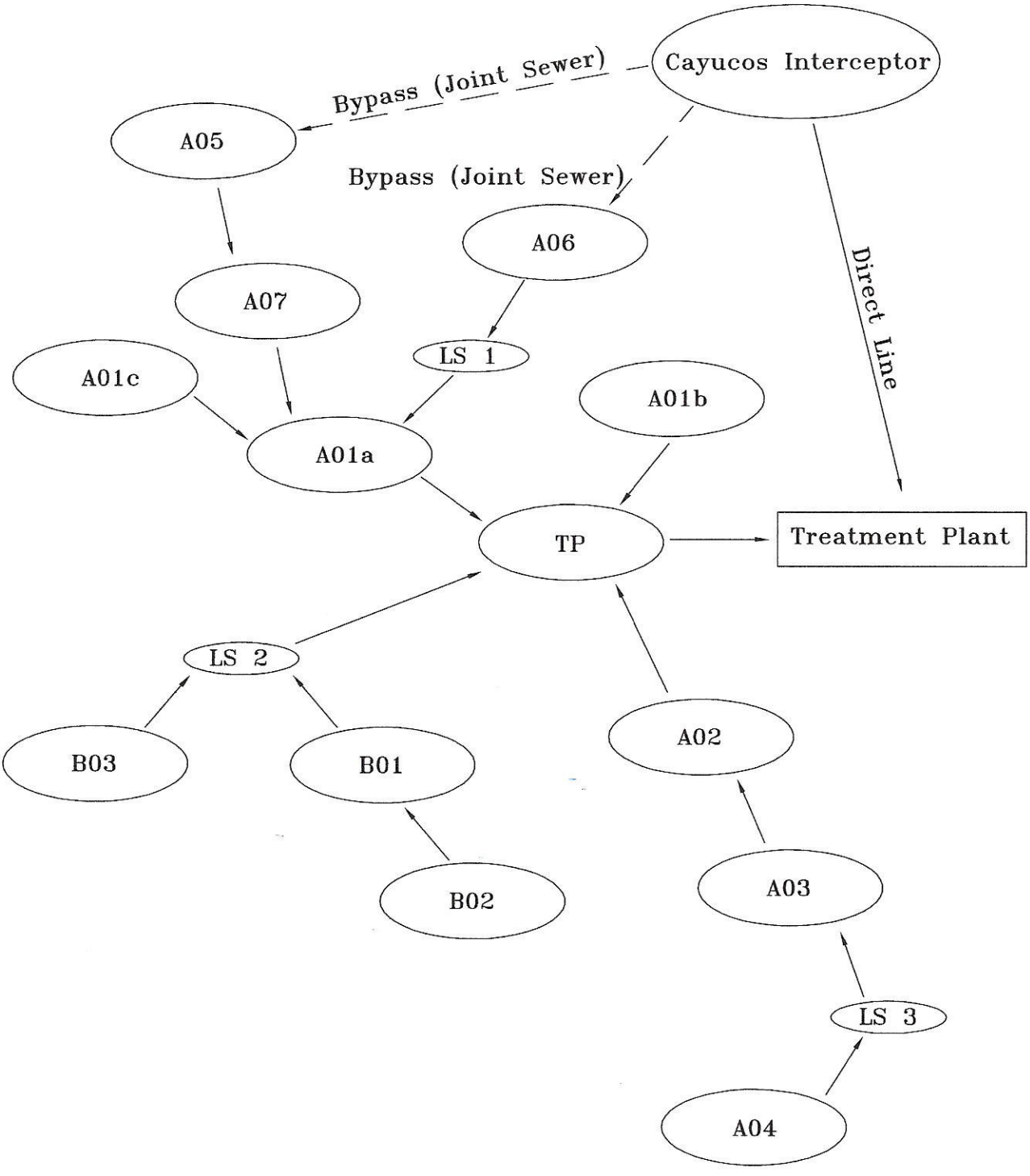

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Figure ES-2

Basin Flow Chart

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Sewer System Master Plan Update

Figure ES-3

City of Morro Bay
Modeled Sewers

Legend

- City Boundary

Modeled Pipes

- 18" to 27" Diameter
- 16" Diameter
- 15" Diameter
- 12" Diameter
- 10" Diameter
- 8" Diameter
- 6" Diameter

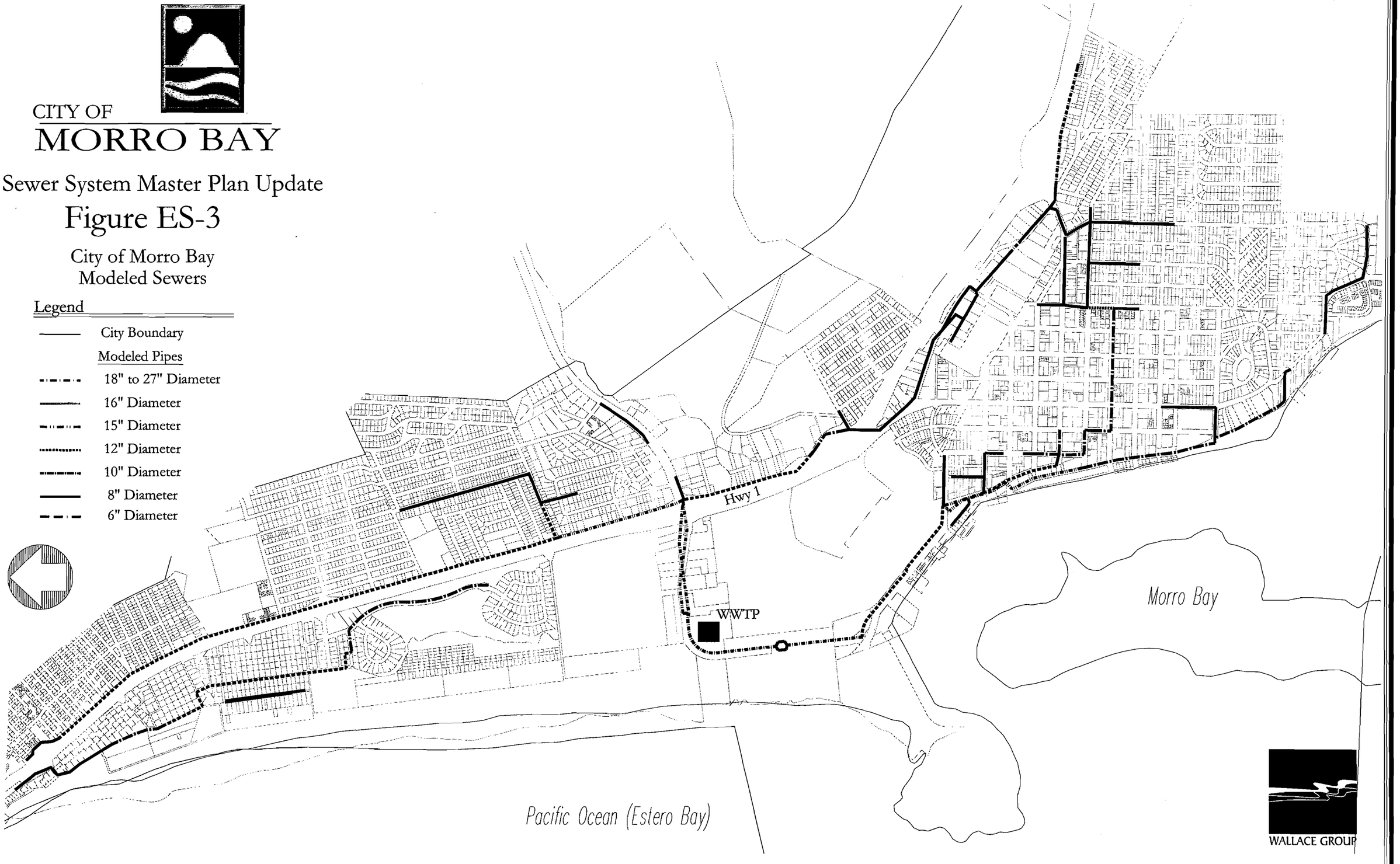

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Table ES-1. Summary of Capital Improvement Projects

Basin	Project Description/Location	City GIS/Manhole Number		Quantity			Old Dia., Inches	New Diameter, Inches[f]	Capital Improvement Cost, $		
		Upstream	Downstream	Value	Units	Unit Cost, $			Construction	Engr/Admin	Total
NEAR-TERM CAPITAL PROJECTS											
	I/I Source Reduction Program[a]	---	---	1	LS	$ 75,000	---	---		$ 75,000	$ 75,000
A02	Main St., 200' n/o Preston Lane Errol St.	N7-004	N7-002	410	LF	$ 200	12	15	$ 82,000	$ 32,800	$ 114,800
	Main St. at Hwy 1 Undercrossing	P7-007	P7-003	210	LF	$ 175	8	12	$ 36,750	$ 14,700	$ 51,450
A03	Quintana Rd., Bella Vista Dr. to Butte Ave.	S11-002	S10-001	1,750	LF	$ 150	6	8	$ 262,500	$ 105,000	$ 367,500
A06	Lift Station 1 Wetwell Washdown, Security Improvements	---	---	1	LS	$ 20,000			$ 20,000	$ 8,000	$ 28,000
TP	Lift Station 2 Wetwell Washdown, Security Improvements, Drypit Hatch and Ladder Replacement	---	---	1	LS	$ 35,000			$ 35,000	$ 14,000	$ 49,000
A04	Lift Station 3 Wetwell Washdown, Security Improvements	---	---	1	LS	$ 20,000			$ 20,000	$ 8,000	$ 28,000
A04	Lift Station 3 Electrical Service Replacement	---	---	1	LS	$ 7,500			$ 7,500	$ 3,000	$ 10,500
A05/A07	Morro Bay/Cayucos Joint Sewer Upgrade[d]	F5-009	G5-009	1,080	LF	$ 200	12	15	$ 216,000	$ 86,400	$ 302,400
	Morro Bay/Cayucos Joint Sewer Upgrade[d]	G5-009	K6-003	4,940	LF	$ 200	12	15	$ 988,000	$ 395,200	$ 1,383,200
A01	Morro Bay/Cayucos Joint Sewer Upgrade[e]	K6-003	M7-022	2,930	LF	$ 225		15	$ 659,250	$ 263,700	$ 922,950
TP	Atascadero Rd., Hwy 41 towards WWTP	M7-022	M6-006	1,010	LF	$ 375	18	27	$ 378,750	$ 151,500	$ 530,250
Subtotal: Near-term Capital Projects									$ 2,705,750	$ 1,157,300	$ 3,863,050
FUTURE CAPITAL PROJECTS											
	I/I Source Reduction Program	---	---	1	LS	$ -	---	---	$ -	$ -	TBD
A02	Main St., Errol St. to Hwy 41	N7-002	M7-022	370	LF	$ 200	12	15	$ 74,000	$ 29,600	$ 103,600
	Main St., n/o Dunbar St.	P7-003	O7-007	340	LF	$ 175	10	12	$ 59,500	$ 23,800	$ 83,300
A06	Lift Station 1 Telemetry Upgrade	---	---	1	LS	$ 5,000	---	---	$ 5,000	$ 2,000	$ 7,000
TP	Lift Station 2 Upgrade to Submersible PS[b]	---	---	1	LS	$ 300,000	---	---	$ 300,000	$ 120,000	$ 420,000
A04	Lift Station 3 Upgrade to Submersible PS[c]	---	---	1	LS	$ 250,000	---	---	$ 250,000	$ 100,000	$ 350,000
Subtotal: Future Capital Projects									$ 688,500	$ 275,400	$ 963,900

Notes:

[a]Annual budget (recurring cost) of $25,000 per year for 3 years, for I/I source reduction.

[b]Includes telemetry upgrade.

[c]Includes telemetry upgrade, water service, and site work to reduce flooding potential.

[d]Segment G5-009/K6-003, Island Street to Las Vegas Street, will need to be implemented prior to F5-009/G5-009, Mindoro Street to Island Street.

[e]Parallel 15" relief sewer.

[f]All sewer replacements/upgrades are modeled as PVC pipe (n=0.010), unless otherwise indicated.

TBD=To be determined. Capital cost to be determined after source of I/I identified.

to anticipate the anticipated growth needs. Where applicable, details of timing of future improvements are expressed in terms of units of growth as opposed to specific timeline.

ALLOCATION OF CAPITAL COSTS TO EXISTING CUSTOMER BASE AND FUTURE DEVELOPMENT

It is recommended that the City conduct a rate study by a qualified firm to determine recommended updates to the City's rate structure, for sewer hook-up fees, development impact fees, and operation and maintenance costs. This section describes a general recommendation for how the City should allocate capital costs of improvements to both the existing customer and future development (impact fees).

Impact fees, or future increased connection fees, for future development are typically calculated based on the development's percentage increase in flow or impact to the infrastructure that will support the development. This potential impact could be based on a basin by basin review, or more globally based on City-wide services. For the City of Morro Bay, it is recommended that any allocation of impact fees be based on the overall service area as a whole. It is felt this approach would be most equitable to all parties concerned. As an example, a developer could build a substantial development in one area of the City, within a collection system area that will require no future improvements to support this development. If impact fees are assessed based on basin-specific needs, this Developer would need only pay the ordinary sewer connection fee per unit. Conversely, one could build the same number of homes or fewer, in another area of the City, and could trigger collection system improvements that would be more costly, thus having to pay for a larger incremental share of the improvement.

NEAR-TERM CIP RECOMMENDATIONS

The following are "immediate" or near-term recommendations for improvements to the City's sewer collection system:

- Implement a program to identify sources of inflow and infiltration throughout the collection system, and implement measures to mitigate the I/I sources once identified. Refer to Chapter 3 for a definition of inflow and infiltration. The priority for targeting I/I identification and source removal programs within the City's collection system are summarized as follows (1 being the most severe I/I problem):

1. Basin A07
2. Basin A06 Upper
3. Basin A01c
4. Basin A05
5. Basin A06 Lower
6. Basin A04
7. Basin B03
8. Basins B01 and B02
9. Basin A03
10. Basin A01b
11. Basin A02
12. Basin A01a
13. Basin TP

As mentioned in Chapter 3 and 4, it is also recommended that the overall I/I problem be addressed, and thus the Cayucos SD should also consider actively pursuing means of reducing and controlling I/I to the joint wastewater treatment plant.

Budgeting for I/I investigation and source reduction can be difficult. Until the source(s) of infiltration and inflow are identified, the magnitude of cost to reduce the sources of wet weather flow cannot be determined. However, the City should reserve an annual budget on an on-going basis to implement an I/I reduction program. With over 50 miles of gravity sewers throughout the City, video inspection alone could cost the City $130,000 or more. Recent video costs noted by Wallace Group for other local agencies were approximately $0.50/LF.

The City's I/I source identification program should consist of some or all of the following:

- Additional focused I/I wet weather monitoring in the "hot spots", to further pinpoint sources of I/I, Winter 2005/2006.

- Smoke test suspect areas of high inflow, including the northern portion of Basin A06, and Basin A01c, Winter/Spring 2006.

- Video inspect sewers that are of high potential for inflow and infiltration, including the northern portion of Basin A06 (particularly in the backyard sewers), and collection system components within Basin A04 (suspected infiltration), Winter 2005/2006.

- Continue program to maintain manhole gasket/water tight lids already installed in areas prone to flooding, especially in Basins A05 and A07, and particularly on Main Street where manholes are situated in the flowline of the street. It is recognized that the has already placed covers on over 50% of the existing manholes, with the majority of these installations in North Morro Bay. Based on this, significant further reduction of I/I from this action alone, is not anticipated.

NEAR-TERM COLLECTION SYSTEM TRUNK LINE UPGRADES

Figure ES-4 depicts the near-term collection system upgrades. Refer to Figures ES-5, ES-6 and ES-7 for locations of the specific pipe reaches with recommended upgrades. Refer to Table 7-1, Chapter 7, for details of other pipelines recommended for close monitoring.

Morro Bay/Cayucos SD Joint Sewer. It is recommended that the City and Cayucos SD clarify their contractual understanding of specific hydraulic capacities before considering future expansion of this sewer. If the 40% capacity is not needed, the 12" reach of joint sewer does not likely need any future upgrades, so long as the I/I component can be reduced. Upgrade of the 12" diameter joint sewer, from Vashon Street to Las Vegas Street, however, should be anticipated in the future should the 40% capacity allocation remain intact. Figures ES-4 and ES-5 portray the modeled hydraulic deficiencies under the flow scenarios including and excluding Cayucos SD capacity, under existing and future dry weather and wet weather flow conditions. As indicated in Chapter 5, near-term efforts for this trunk sewer should focus on reduction of I/I as a first order of business.

The existing 18" gravity sewer, from Manhole M7-022 to M6-001 (approximately 1,000 LF) needs to be upgraded to a 27" sewer (smooth-wall PVC, n=0.010), to hydraulically match the other portion of sewer recently upgraded to 27" diameter.

Figure ES-5

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Figure ES-4

Summary of Gravity Sewer Capital Improvements

Label	Pipe	Location	
1	N7-002 to M7-022	Main St., Errol St., to Atascadero Rd.	
2	N7-004 to N7-002	Main St., 200' N/O Preston Ln. to Errol St.	
3	N7-007 to N7-004	Main St., Preston Ln. to 200' N/O Preston Ln.	M
4	O7-002 to N7-007	Main St., Frontage Rd. to Preston Ln.	M
5	O7-005 to O7-003	Main St. at Frontage Rd.	M
6	P7-003 to O7-007	Quintana Rd., 100' to 300' N/O Dunbar St.	
7	P7-007 to P7-003	Quintana Rd. at Dunbar St.	
8	Q8-007 to P7-008	Quintana Rd. at Main St.	M
9	R9-003 to R9-002	Kennedy Way, 200' N/O Quintana Rd.	M
10	S9-001 to R9-010	Quintana Rd. at Morro Bay Blvd. to 300' Morro Bay Blvd.	M
11	R9-010 to R9-008	Quintana Rd. near McDonalds/Burger King	M
12	S10-001 to S9-003	Quintana Rd., 200' E/O Butte Ave. to Morro Bay Blvd.	M
13	S11-002 to S10-001	Quintana Rd., 200' E/O Bella Vista Dr. to Butte Ave.	

Legend

- Deficient Pipes - Existing Flow and Build-Out Flow
- Deficient Pipes - Build-Out Flow Only
- Marginal Capacity - To be Monitored
- M Monitor

*Refer to Table 7-1 for full descriptions of improvements

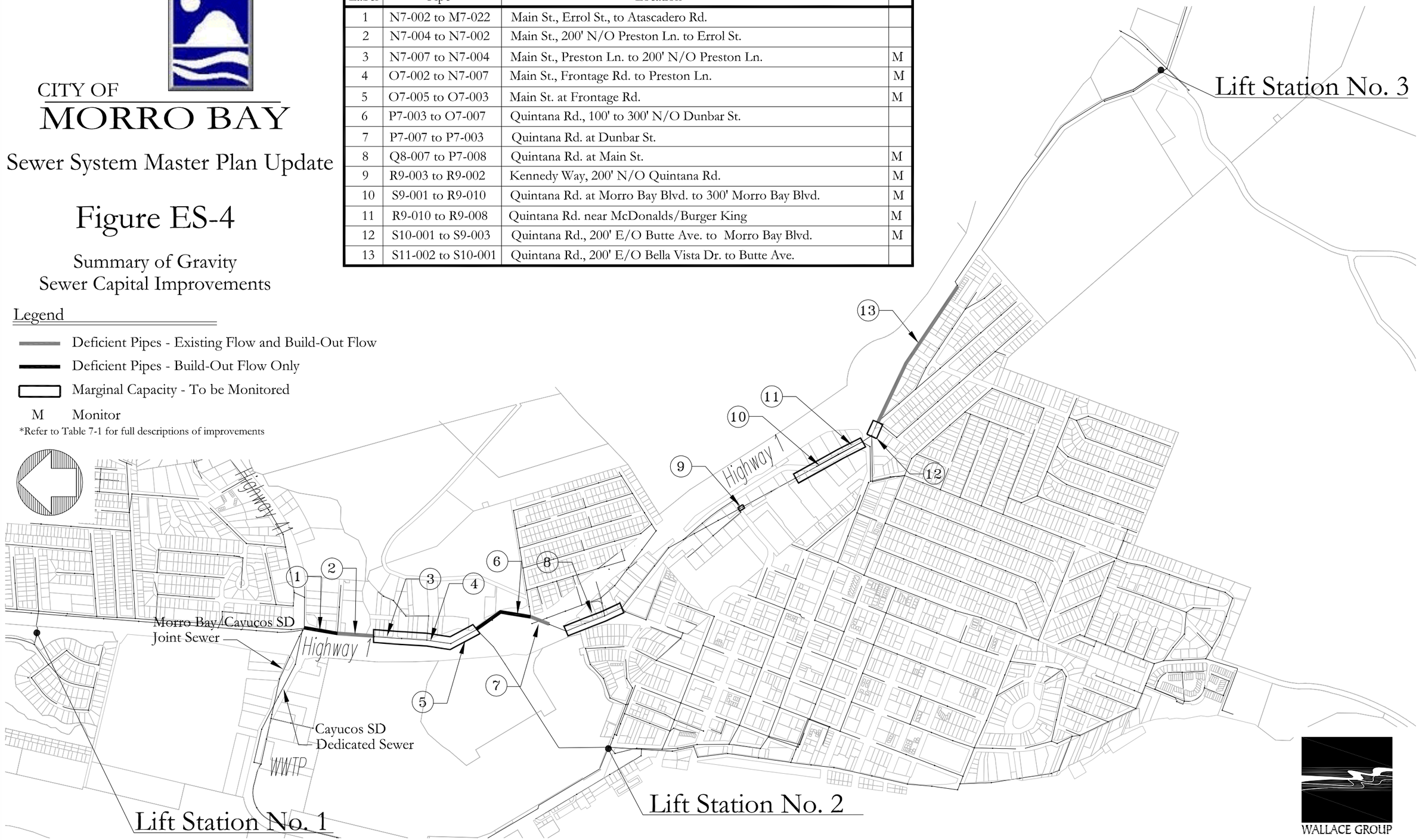

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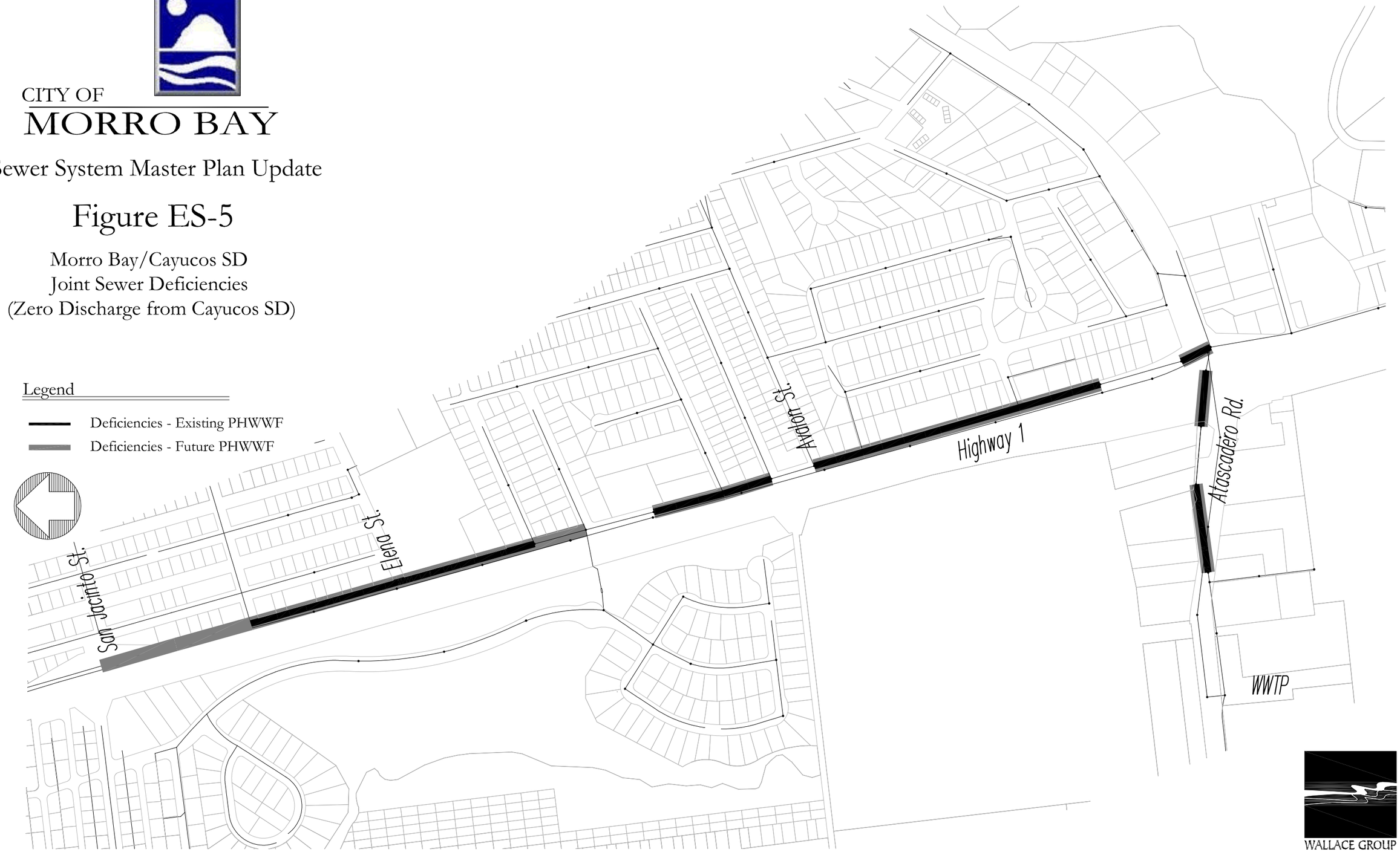
Figure ES-5

Morro Bay/Cayucos SD
Joint Sewer Deficiencies
(Zero Discharge from Cayucos SD)

Legend

- Deficiencies - Existing PHWWF
- Deficiencies - Future PHWWF



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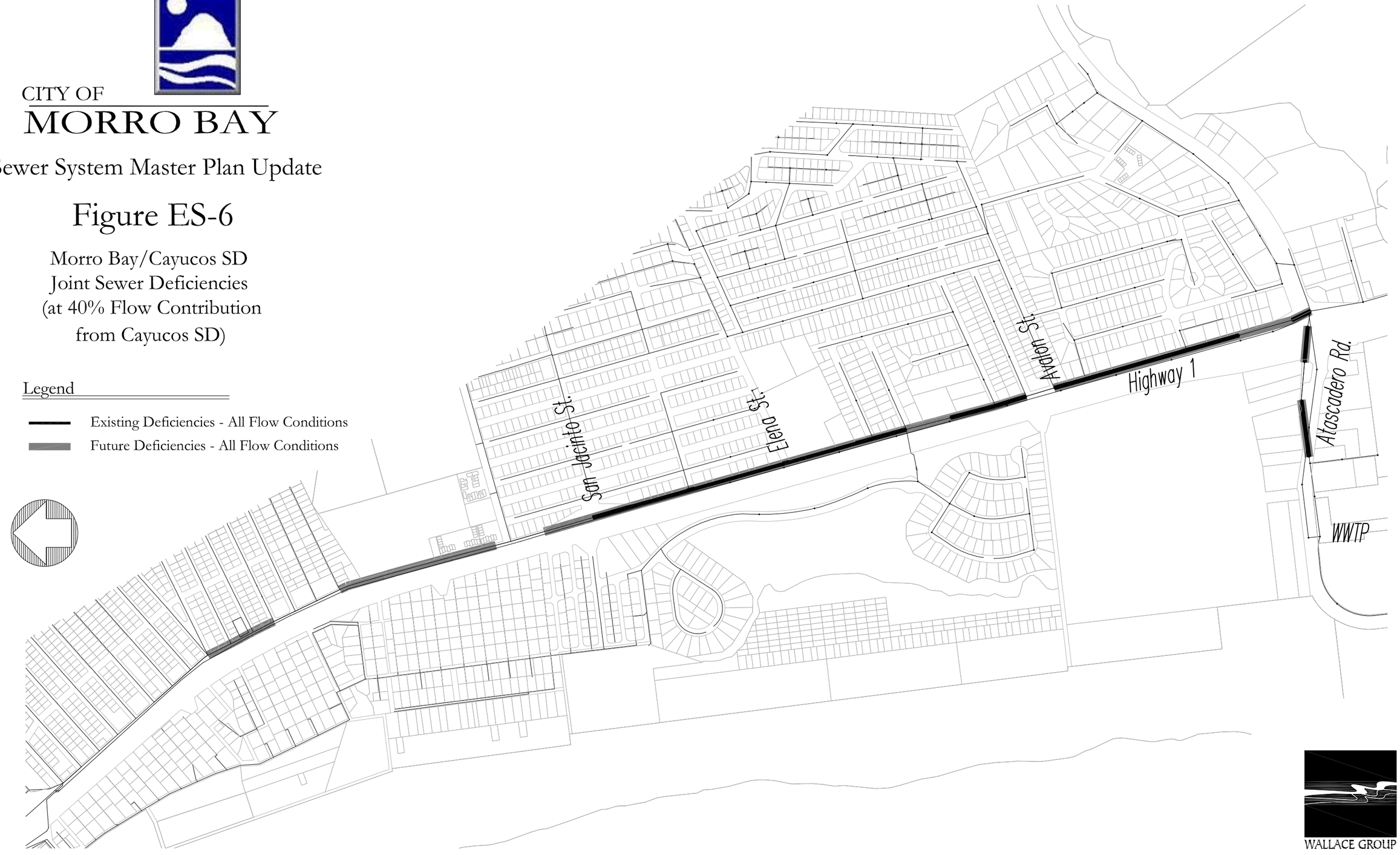
Sewer System Master Plan Update

Figure ES-6

Morro Bay/Cayucos SD
Joint Sewer Deficiencies
(at 40% Flow Contribution
from Cayucos SD)

Legend

- Existing Deficiencies - All Flow Conditions
- Future Deficiencies - All Flow Conditions



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NEAR-TERM LIFT STATION IMPROVEMENTS

Implement the following projects at all three lift stations:

- Provide wetwell washdown system. If concrete attack continues, provide wetwell concrete lining to safeguard sulfuric acid attack.
- Provide improved security features to minimize tampering potential.

Other specific lift station improvements for the near-term are summarized as follows:

Lift Station 2:

- Replace drypit hatch/lid, and access ladder.

Lift Station 3:

- Wetwell concrete lining to safeguard against sulfuric acid attack.
- Replace/upgrade existing electrical service.

FUTURE COLLECTION SYSTEM IMPROVEMENTS

Refer to Figures ES-4, ES-5 and ES-6 for locations of the specific pipe reaches recommended for future upgrade. Refer to Table 7-2, Chapter 7, for details of those sewers that are recommended for further observation and monitoring in the future.

Basin A02. All sewer upgrades for Basin A02 (Figure ES-6) are recommended after approximately 130 equivalent residential units of development occur. Thus, the City should begin planning/design for these upgrades approximately two years prior. Based on what is envisioned to occur in Basins A02, A03 and A04, the development of 130 equivalent residential units could take a number of years, or may not even be reached. Thus, further monitoring in future years is recommended to verify if the upgrade is necessary in the future or not.

FUTURE LIFT STATION IMPROVEMENTS

The following are lift stations recommendations for the future. The identified future upgrades are more safety and operational oriented, and are not necessarily triggered by future growth. These recommendations for Lift Station 2 and 3, however, will include significant capital investment, and should be scheduled for in conjunction with other factors, including anticipation of when next major pump upgrades might be required, end of useful life of facilities, and funding opportunities from grants, low interest loans, or major development whose capital connection/impact fees can help fund such projects.

Implement the following projects at all three lift stations:

- Provide auto-dialer system to telephone/page designated City staff to respond to alarm conditions.

Other specific lift station improvements for the future are summarized as follows:

Lift Station 1:

- Continue assessing I/I in the future, and reduce I/I in Basin A06 to the extent practicable. Verify actual pumping capacity relative to existing and future anticipated peak flows.

Lift Station 2:

- Upgrade lift station to submersible pump station with self-cleaning wetwell, with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs), and with sufficient hydraulic capacity/redundancy to meet future peak flows in the simplex mode of operation (if confirmed that peak pumping capacity is inadequate).

Lift Station 3:

- Upgrade lift station to submersible pump station with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs).
- In conjunction with lift station upgrade to submersible station, provide flood protection (flood wall or berm) to safeguard vaults from inundation.
- Provide potable water for sanitary and washdown purposes.

OTHER IMPORTANT CONSIDERATIONS

There are many details presented throughout this master plan update report. Other critical considerations that need to be emphasized are outlined in this section.

CAYUCOS SANITARY DISTRICT

The City of Morro Bay and the Cayucos Sanitary District own and operate the treatment plant under a joint powers agreement. The Cayucos SD portion of flow to the treatment plant is delivered directly through a dedicated trunk line from Cayucos. A lift station immediately north of Morro Bay City limits pumps Cayucos wastewater to this interceptor.

The City and Cayucos SD also jointly own a 12" and 15" gravity sewer, located on Main Street from Vashon Street (north Morro Bay) to Atascadero Road (Hwy. 41), and on Atascadero Road to the WWTP (18" to 27" diameter). The Cayucos SD and Morro Bay have joint ownership in this trunk (60% Morro Bay and 40% Cayucos SD). The Agreement does not stipulate specifics as to actual flow deliveries, but simply states capacities based on gallons per day.

EXISTING WW FLOWS

Flow to the treatment plant is monitored at the entrance to the treatment plant. Cayucos flows account for approximately 26% of the total flow to the treatment plant. During the winter months, the joint wastewater treatment plant experiences a substantial increase in wastewater flows due to inflow/infiltration (I/I). The flow peaks in the summer months due to

summer tourist demands. In 2003, the City retained Wallace Group to conduct flow monitoring of each of the City's basins during the winter of 2004 and 2005 to establish the magnitude of I/I coming from each basin. The results of the flow monitoring are reported in Chapter 4. Inflow and infiltration are defined as follows:

> *Inflow* is the water discharged into a sewer system and service connections from such sources as roof drains, cellar, yard and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers and pick holes, cross connections from storm sewers, catch basins, storm water, surface runoff, or drainage. Inflow varies rapidly with rainfall conditions, with flows rising and falling within minutes or hours of a severe storm event.

Both the City of Morro Bay and Cayucos SD collection systems show significant signs of wet weather inflow. This is evidenced by the flow monitoring conducted in 2004 and 2005 in the City of Morro Bay, by the plant flow records, and Cayucos SD reporting of total daily flows. The increase in wet weather flows to the WWTP is substantial. It is noted that the winter flows to the treatment plant increase over 3-fold during wet weather. In contrast, the summer maximum day is approximately 1.5 times average flows.

FLOW COMPARISON TO 1986 SEWER MASTER PLAN

In 1985, the average dry weather flow (ADWF) for the City of Morro Bay (excluding Cayucos SD) was reported at 1.31 mgd. The 1986 report described a "planning" population of approximately 10,660 at that time; it is believed this referenced population included summer tourist influence. The actual permanent residential population in 1986 was likely around 9,000 to 9,500 people. This would translate into a per capita wastewater flow of approximately 140 gpcd at that time.

In 2005, the City's population has grown to approximately 10,500 people, or a 10 percent increase from 1986. Current flow data shows a dry weather flow of 0.835 mgd, or approximately 80 gpcd overall (including all components of flow). Thus, the City's overall population increased 10 percent in the last 2 decades, while decreasing wastewater flows by 40 percent or more. This reduction in flows is likely due in part, to the aggressive water conservation measures that have taken place over the recent years. Another contributing factor to flow reduction at the plant was the replacement of the WWTP influent flow meter in August 2001, which reduced measured plant flows by 25%. It is likely that a number of the pipeline upgrade recommendations in the 1986 report will not be necessary due to this reduction in wastewater flows.

INFLOW AND INFILTRATION MONITORING

During the winter months, the wastewater treatment plant experiences a substantial increase in wastewater flows, both from the City's collection system and the Cayucos SD system. In 2003, the City retained Wallace Group to conduct flow monitoring of each of the City's basins during the winter of 2004 to establish the magnitude of I/I coming from each basin. Unfortunately, the 2003-2004 rainy season was very mild, which resulted in only one significant rain event during the two-month flow monitoring period. Wallace Group recommended to complete additional flow monitoring on specific basins during the following 2004-2005 rain season.

The results of this program identified the "target" areas that the City needs to focus attention on, to further assess and reduce I/I to the collection system and WWTP. The program also confirmed that wet weather flows are a major flow contributor to the WWTP and collection system during the wet season. Priorities for addressing the I/I problem, by Basin, is presented in Chapter 4 and summarized in this Executive Summary.

COLLECTION SYSTEM MODELING AND ANALYSIS

Overall, the City's gravity sewer collection system is in good condition relative to hydraulic capacity. The sewer model results showed that in general, the majority of 8" and larger sewers are of sufficient capacity to serve the City's existing and future build-out population. Details of the analysis is in Chapter 5. It is also noted that there were a number of sewer segments that required additional monitoring in the future. Monitoring was recommended in lieu of pipeline replacement, given the understanding that some flow parameters in the hydraulic model are conservative in nature, and that in coming years the City will be able to provide a significant overall reduction in infiltration/inflow, and that replacement of gravity sewers should occur only if warranted and confirmed. It is also noted that the a number of sewers recommended for upgrades in the 1986 sewer master plan, will no longer be required due mainly to a significant overall flow reduction based on water conservation throughout the City.

LIFT STATION EVALUATION

The lift station evaluation covered hydraulic and non-hydraulic parameters, and details of the analysis are presented in Chapter 6 of this Report. Some of the major considerations noted are as follows:

- Lift Stations 2 and 3, with wet pit/dry pit designs, require confined space entry procedures for routine and emergency maintenance. Consideration should be given in future years to upgrade these lift stations to eliminate this safety issue.

- In evaluating Lift Station 3 pump run times, it was discovered there may be a significant infiltration problem in Basin A04, near the trailer parks, convalescent home and mortuary. This trend was not detectable during the sewer flow monitoring efforts.

- Lift Stations 1 and 2 may be marginal with respect to pumping future flows coupled with large peaks of infiltration/inflow. Efforts to reduce I/I should be a priority, and further assessment of the capacity of these lift stations should be conducted in later years as the I/I program progresses.

CHAPTER 1

INTRODUCTION

This report presents the Wastewater Collection System Master Plan Update for the City of Morro Bay. The City of Morro Bay provides its customers with sewage collection and treatment service, domestic water service, fire protection, and other services. The current population within the City limits is approximately 10,500 and is expected to increase to 12,500 at build-out. The City is not facing significant growth potential within the City limits. It is the City of Morro Bay's intent to construct wastewater system improvements consistent with the current and ultimate needs of the City. In order to facilitate this goal, and to adequately plan for the capital resources needed to meet this goal, the City has elected to update an existing Wastewater Master Plan prepared by CH2MHILL in 1986.

Preparation of a wastewater systems master plan will assist the City in prioritizing both present and future wastewater system needs and set forth a mechanism for addressing those needs. Present needs addressed in the wastewater system master plan will include the "three R's": Repair, Rehabilitation, and Replacement. Future needs will address those capital improvements required to support the anticipated growth of Morro Bay through the next twenty years. The master planning process will also tie the needs assessment, both existing and future, to the budgeting process.

ENVIRONMENTAL REVIEW

In accordance with Title 14, California Code of Regulations, Chapter 3, Article 18 (Statutory Exemptions), this wastewater collection system master plan update is considered a planning study and therefore adoption of this document is exempt from the requirements to prepare Environmental Impact Reports (EIR) or Negative Declarations (ND). However, on a project-specific basis, CEQA must be satisfied for any major capital improvement projects described in this report that will be implemented by the City in the future, through the preparation of an appropriate EIR or Negative Declaration.

AUTHORIZATION AND SCOPE OF WORK

In January 2004, the City of Morro Bay authorized Wallace Group to prepare a comprehensive wastewater system master plan update. This plan was originally anticipated to be completed in Summer 2004. However, due to a very dry winter and the need to capture additional sewer flow monitoring data, the report schedule was extended to 2005. This wastewater master plan is prepared in accordance with Wallace Group's proposal dated December 18, 2003, and includes analyses of the City's wastewater flows, collection system capacity, evaluation of lift stations and force mains; and a prioritized capital improvement program.

ACKNOWLEDGEMENTS

Wallace Group thanks and acknowledges the following City representatives for their efforts, involvement, input and assistance in preparing this engineering report and analysis:

Bill Boucher, Capital Projects Manager
Dave Phillips, Maintenance Supervisor
Frank Cunningham, City Engineer
Greg Cummings, Planning Manager
Mike Prater, Planning Manager
Bruce Keogh, Wastewater Division Manager

The following Wallace Group key team members were involved in the preparation of this water master plan report:

Steven G. Tanaka, P.E., Director of Water Resources
Kari Wagner, P.E., Professional Engineer
Beth Montanez, P.E., Professional Engineer
Ray Dienzo, Associate Engineer

CHAPTER 2

DEMOGRAPHICS

This chapter presents the demographics, and existing and future population forecasts for the City of Morro Bay. Demographic information was obtained from the proposed General Plan/Local Coastal Plan of the City of Morro Bay, dated February 2004.

INTRODUCTION

The City of Morro Bay is located in San Luis Obispo County. The planning area includes the area within the city limits. The General Plan of Morro Bay lists the current population as approximately 10,510, with a corresponding household density of 2.02.

The 1986 Master Plan used a 1985 planning population of 10,660 based on the number of dwelling units and a household density of 2.2. In 1986 the estimated build-out population was stated as 14,190, estimated to occur around 2010.

LAND USE

The City of Morro Bay Service Area covers approximately 1,650 acres. The land use zones are Low Density Residential (LDR), Moderate Density Residential (MDR), Medium Density Residential (MDR), High Density Residential (HDR), Mixed Use (MU), Service Commercial (SC), Visitor Serving Commercial (VSC), Community Commercial (CC), Commercial/Recreational Fishing (CRF), General (Light) Industrial (GLI), Public Facility (PF), Agriculture (AG), Open Space/Recreation (OSR). The Residential land use zones and density allowed are illustrated in Table 2-1. Existing land use/zoning maps are available in the City's current General Plan, and future potential land use/zoning is provided on the City's proposed General Plan Update (February 2004). This proposed zoning map is provided as Figure 2-1.

Table 2-1. Land Use Category Acreage Breakdown And Density

Land Use Category	Density
Low Density Residential (LDR)	Up to 4 DU/acre
Moderate Density Residential (MDR)	4 to 7 DU/ acre
Medium Density Residential (MDR)	7 to 15 DU/ acre
High Density Residential (HDR)	15 to 27 DU/ acre
Mixed Use (MU)	15 to 27 DU/acre
Service Commercial (SC)	---
Visitor Serving Commercial (VSC)	---
Community Commercial (CC)	7 to 15 DU/acre
Commercial/Recreational Fishing (CRF)	---
General (Light) Industrial (GLI)	---
Public Facility (PF)	---
Agriculture (AG)	Up to 4 DU/acre
Open Space/Recreation (OSR)	---

CITY OF MORRO BAY PROPOSED GENERAL PLAN/LCP LAND USE

City Limits
Coastal Zone Boundary
Parcel Boundary

Low Density Residential
Moderate Density Residential
Medium Density Residential
High Density Residential
Mixed Use
Service Commercial
Visitor Serving Commercial
Community Commercial
Commercial / Recreational Fishing
General (Light) Industrial
Coastal Dependant Industrial
Public Facilities
Agriculture
Open Space / Recreation

Environmentally Sensitive Habitat

SP-A Special Planning Area

0 875 1,750 3,500
FEET

Prepared by Dyett & Bhatia, August 17, 2004

Figure 2-1

GROWTH

The population projections in the 1986 Master Plan were based on the limited water availability and the General Plan at the time. Today, future population growth is not as restricted by water availability because of conservation efforts, the desalination plant and state water delivered to the City. Despite this, the population has not grown at the rates expected in the 1986 Master Plan.

In 1984, Measure F was passed that limited the population of Morro Bay to 12,200. It was assumed that the city would meet this cap in 2000, but the city has only grown slightly since the measure was passed.

There are approximately 200 infill residential lots available for future growth. In addition, there are four major areas identified by the City that are planned for growth. These areas are shown on Figure 2-2. The combination of these lots allow for a population growth of approximately 900 to 1,000 people. Therefore, build-out population would be approximately 11,500 based on this potential remaining development.

For this model, the Morro Bay Planning Department estimates of the City build-out population will be used, to be conservative. The build-out population is approximately 12,500, including an 80% occupancy factor. A peak summertime population is estimated at approximately 15,650. On average, the city has been growing by approximately 60 housing units per year, or approximately 125 people per year. At this rate, build-out would be reached in approximately 16 years.

POTENTIAL FUTURE DEVELOPMENT

Residential. As stated earlier in this chapter, there are approximately 200 infill residential lots available for future growth. In addition to the infill, there are several areas of development that could potentially develop denser than currently zoned, shown on Figure 2-1. In north Morro Bay, there is high density potential, of up to 180 units. Another 64 potential units near the power plant on the east side of Highway 101 are planned. This area is fairly steep, with power lines over it. A PG&E facility will be included in the development. Another area is near Lift Station 3 (near the senior retirement center). There are about 10 single family homes currently planned for this area. South of Lift Station 3, near the trailer park, there are another 17 single family dwelling units already under way. There is also currently a 16-unit development under construction on Main Street at Quintana Road near the Lemos Pet Supply store, a 19-unit development at Main Street and Driftwood Avenue, as well as various other small developments in the City.

There is also the potential for a number of second units to be added to existing SFR units, and on new SFR lots currently not developed. The quantity of future second units is not known, however such development could be significant.

CITY OF
MORRO BAY

Sewer System Master Plan Update

Figure 2-2

Legend

- City Boundary
- Areas of Future Development

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Commercial. There is not a significant amount of future commercial development planned for the city. In the downtown core, there is a potential for mixed use increasing including housing units on top of businesses. It is becoming more prevalent, but to what degree that will occur is not known. In the Embarcadero area, there is not likely much to happen because the area is built-out. Near Atascadero Road and Highway 1, there is the potential for another motel similar to the Motel 6, identified as a 31-room Comfort Inn.

The Tri "W" site was rezoned by Measure B in 1986 to mixed commercial uses. In 1990, Measure H repealed the 1986 initiative and changed the zoning and limited the use of the Tri "W" property. No develop has occurred on this property and it is very unlikely that any development will occur within the planning horizon of this study.

AFFORDABLE HOUSING ELEMENT

State law requires cities to provide a Housing Element that encourages affordable housing in the community. For a new development with more than 10 units, 10% of those units are required to be affordable housing units. As an incentive to offer more affordable housing units to developments of at least 5 new units, a developer can get a maximum density bonus of 35% beyond the current land use designation and zoning. Variable Density would promote affordable units by allowing a greater number of residential units than otherwise allowed under the existing land use and/or zoning, provided 65% of the units are studio or 1-bedroom units rather than the typical 2 or more bedroom units. Because of the limited number of potential larger developments and limited land availability, the affordable housing element will not greatly impact future developments and therefore will not affect the growth of the city substantially.

CHAPTER 3

WASTEWATER FLOWS

This Chapter presents the flow projections for the City, to be used in the analysis of the Collection System.

INTRODUCTION

When discussing sewage flows it is important to define some of the terminology commonly used to describe and analyze wastewater flows.

Average Daily Flow (ADF) is the average daily wastewater flow over the course of a year and is generally obtained by averaging the mean monthly flows conveyed to the WWTP through the course of a year. In the case of this report the ADF is based on flow records at the WWTP for a 27 month period between January 2002 to March 2005. The ADF for the City of Morro Bay is 0.83 mgd.

Maximum Day Dry Weather Flow (MDDWF) reflects the maximum day flow rate during the peak month of summer. This condition reflects the seasonal variation in dry weather flow. For the purposes of this study, the recent historical MDDWF is 1.5 mgd at the WWTP, and this occurred on the 4th of July, 2004.

Peak Hour Wet Weather Flow (PHWWF) is the maximum flow rate that occurs in a single hour during wet weather (a significant rain storm event). This factor is derived from standard engineering methodology and judgment combined with actual flow monitoring data. This flow condition will govern the design of the sewage collection system and represents the maximum flow rate that the system must convey. As described in Chapter 5 of this report, PHWWF is derived by multiplying ADF times the diurnal peaking factor, then adding the wet weather flow component. The existing PHWWF for the Morro Bay flow component is approximately 2,600 gpm, or 3.8 mgd.

Peak Month Flow is the average daily flow received at the WWTP over the course of the peak month. This flow is used to report WWTP flows to the Regional Water Quality Control Board. This peak month flow occurred in January 2005 and the value was 1.533 mgd for combined Morro Bay and Cayucos Sanitary District flows.

COLLECTION SYSTEM OVERVIEW

This section presents an overview of the various components of the collection system, including the City's 11 tributary basins and three lift stations.

Tributary Basins

The sewage collection system has eleven drainage basins: A01, A02, A03, A04, A05, A06, A07, B01, B02, B03, and TP, as shown on Figure 3-1. Basins A06, A05, A07, and A01c flow into basin A01a before entering the treatment plant. Basins B01, B02, and B03 flow into basin TP before entering the treatment plant. Basins A03 and A04 flow into A02 before entering the TP basin. Basin A01b flows into the TP basin without any upstream tributary flow. The flow chart depicting the relationship between basins is shown on Figure 3-2.

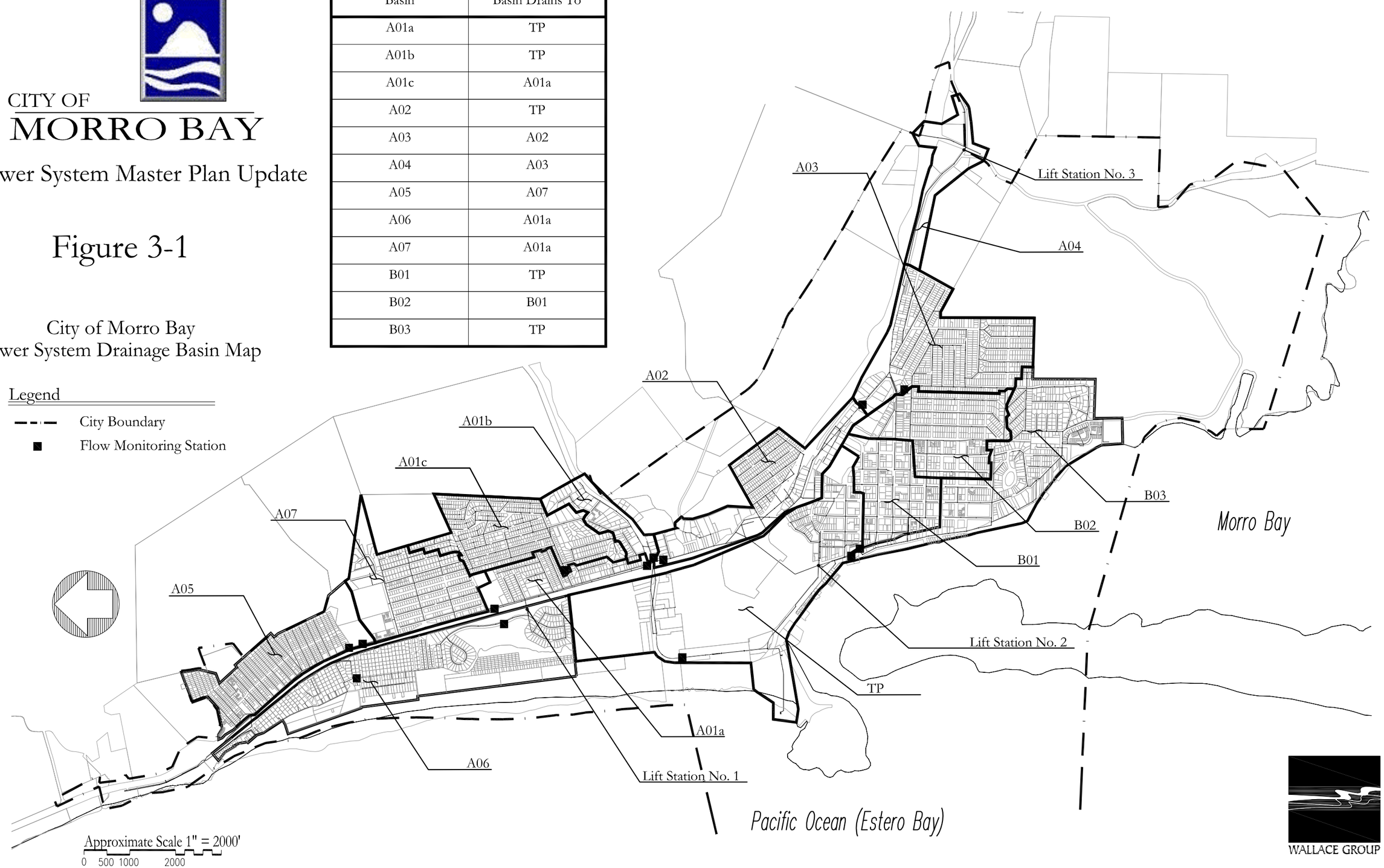
CITY OF
MORRO BAY

Sewer System Master Plan Update

Figure 3-1

City of Morro Bay
Sewer System Drainage Basin Map

Basin	Basin Drains To
A01a	TP
A01b	TP
A01c	A01a
A02	TP
A03	A02
A04	A03
A05	A07
A06	A01a
A07	A01a
B01	TP
B02	B01
B03	TP



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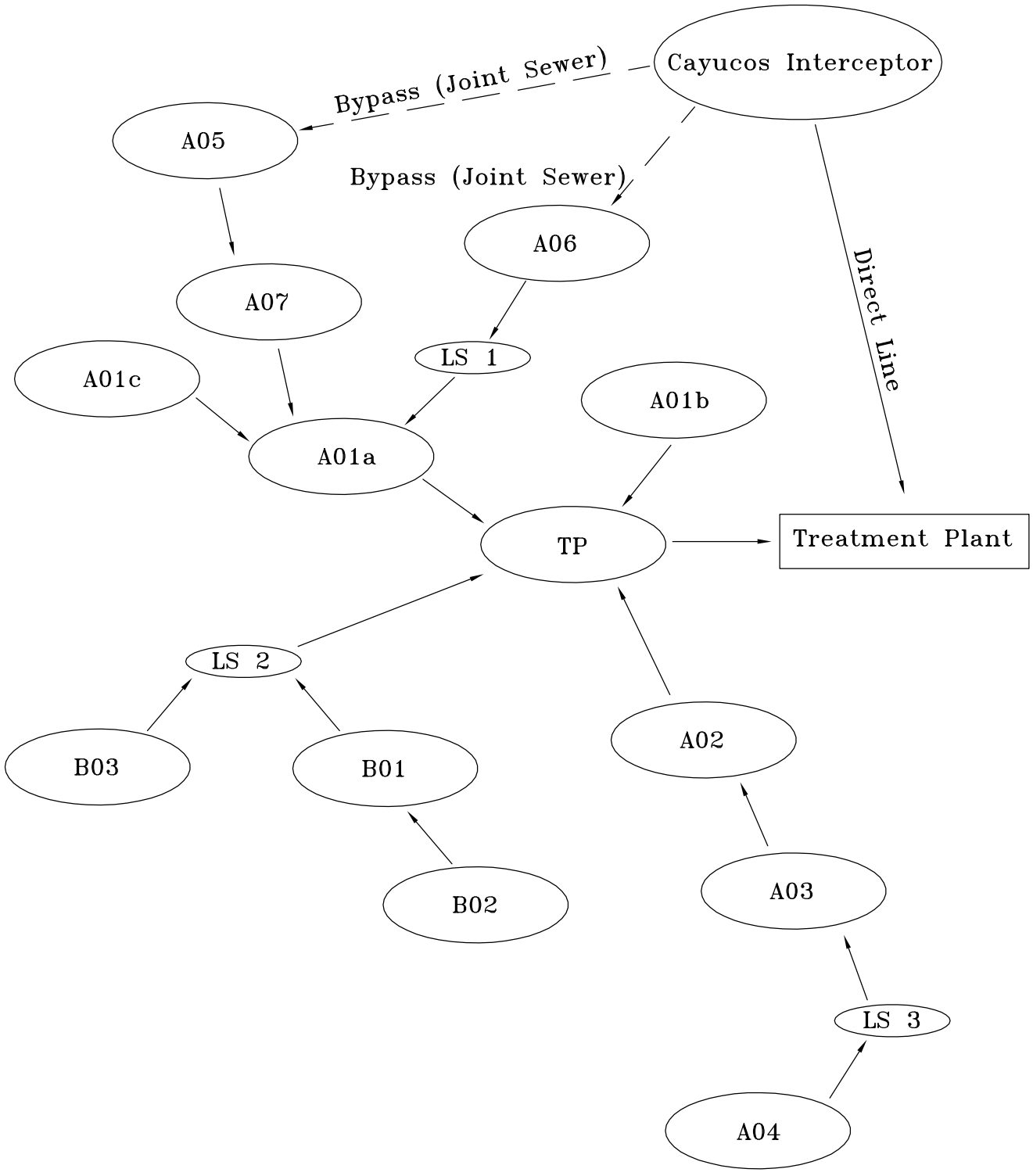

CITY OF
MORRO BAY
Sewer System Master Plan Update

Figure 3-2

Basin Flow Chart

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Lift Stations

The City of Morro Bay owns and operates three sewage lift stations. These lift stations and corresponding service areas are briefly summarized in this chapter, and details of the hydraulic capacity, equipment and other details of the lift stations is provided in Chapter 6.

- Lift Station 1. Lift Station 1 is located in "the Cloisters" development, at 2601 Coral Street. The lift station services all of Basin A06, which then pumps the wastewater flow to Basin A01a. The lift station discharges through a 6-inch diameter ductile and cast iron (cast iron under Highway 1) force main to a manhole at the intersection of Main Street and Las Vegas Street.
- Lift Station 2. Lift Station 2 is located on the "back side" (east) of a parking lot on the Embarcadero, near one of the City's maintenance yards. It is across the street from the Great American Fish Company and South Pier. The lift station services a portion of the TP drainage basin, and Basins B01, B02 and B03. The lift station discharges through a 12-inch diameter asbestos cement force main to a manhole on Embarcadero Street (west of the Duke Power Plant) and 18-inch diameter gravity sewer. There is a second "backup" force main, that is an 8-inch diameter cast iron force main, that discharges to a 12-inch diameter gravity sewer in Basin A-02, on Main Street/Frontage Road.
- Lift Station 3. Lift Station 3 is located on Quintana Road just west of the intersection with South Bay Boulevard. The lift station services drainage basin A04, which collects sewage from two mobile home parks, a convalescent hospital, mortuary and Christian church. The lift station discharges through a 6-inch diameter asbestos cement force main up Quintana Road, where it discharges to Basin A03.

Cayucos Sanitary District

The City of Morro Bay and the Cayucos Sanitary District own and operate the treatment plant under a joint powers agreement. The Cayucos SD portion of flow to the treatment plant is delivered directly through a dedicated trunk line from Cayucos. A lift station immediately north of Morro Bay City limits pumps Cayucos wastewater to this interceptor, as shown on Figure 3-3.

There are also valves in the system that connect the Cayucos CSD force main (downstream of their last lift station prior to entering Morro Bay) to the joint Morro Bay/Cayucos gravity sewer (12" and 15" on Main Street), that allows Cayucos SD flow to enter the joint main to the treatment plant. The Cayucos SD and Morro Bay have joint ownership in the trunk line running down Main Street to the treatment plant (60% Morro Bay and 40% Cayucos). The Agreement does not stipulate specifics as to actual flow deliveries, but simply states capacities based on gallons per day. Another connection to basin A06 exists, providing further flexibility for Cayucos SD to divert wastewater flows to this portion of the City's collection system in the event of disruption to the Cayucos interceptor. However, Cayucos does not own any capacity rights to basin A06.

CITY OF
MORRO BAY

Sewer System Master Plan Update

Figure 3-3

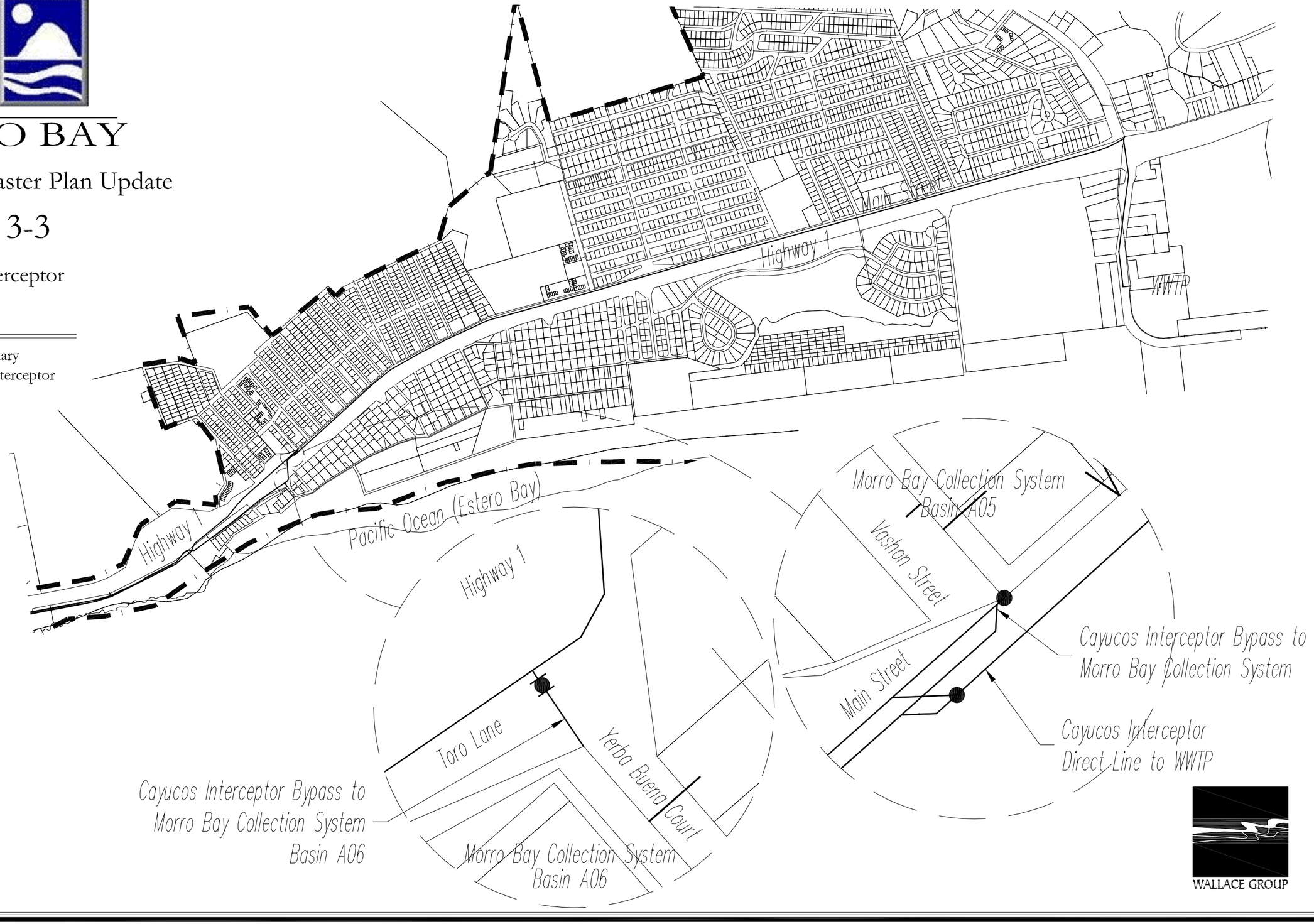
Cayucos Interceptor

Legend

- City Boundary
- Cayucos Interceptor

Approximate Scale 1" = 1000'

0 250 500 1000



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EXISTING WW FLOWS

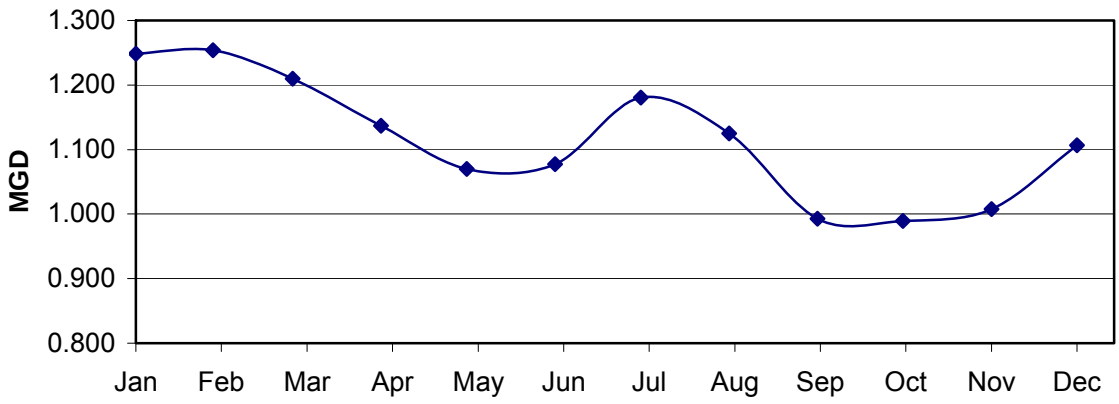
Flow to the treatment plant is monitored at the entrance to the treatment plant. Cayucos flows account for approximately 26% of the total flow to the treatment plant. Actual flow to the treatment plant for calendar year 2004 are summarized in Table 3-1, and portrayed graphically on Figure 3-4.

Table 3-1. WWTP Flow Summary[1]

	Average Daily Flow (MGD)	Peak Day Wet Weather Flow (MGD)
Total WWTP Flow	1.125	3.496
Morro Bay Flow	0.833	2.587
Cayucos Flow	0.292	0.909
Treatment Plant Design Capacity	2.06[2]	6.64[3]

[1]Data collected from January 1, 2002 to March 31, 2005
[2]Treatment capacity based on organic loading to plant.
[3]Peak hydraulic Capacity to plant.

Figure 3-4. Summary of Wastewater Flows to WWTP (2004)



During the winter months, the joint wastewater treatment plant has experienced a substantial increase in wastewater flows to the wastewater treatment plant as shown in Table 3-1 and on Figure 3-4. The flow peaks in the summer months due to summer tourist demands, and also peaks during the winter due to inflow/infiltration. In 2003, the City retained Wallace Group to conduct flow monitoring of each of the City's basins during the winter of 2004 and 2005 to establish the magnitude of I/I coming from each basin. The results of the flow monitoring are reported in Chapter 4.

Both the City of Morro Bay and Cayucos SD collection systems show significant signs of wet weather inflow. This is evidenced by the flow monitoring conducted in 2004 and 2005 in the City of Morro Bay, by the plant flow records, and Cayucos SD reporting of flows. The increase in wet weather flows to the WWTP are substantial. It is noted that the winter flows

to the treatment plant increase over 3-fold during wet weather. In contrast, the summer maximum day is approximately 1.5 times average flows.

From the model, the number of connections estimated for Morro Bay is approximately 6,125. Using the actual existing ADF of 0.833 MGD and a 2004 base population of 10,510 persons, the City experiences a base flow of 80 gallons per capita per day (gpcd). These figures include all land uses including residential and commercial. Population figures include a 20% vacancy rate. The existing number of persons per household is approximately 2.02 per the City planning department.

As shown in Table 3-2, flow per capita varies throughout the year. In the summer, the per capita flow is greater than in the spring and fall, due to the transient tourist population. During the winter (January, February, and March), the per capita flow is on the high extreme due to wet weather inflow and infiltration.

Table 3-2. Morro Bay Monthly Flows

Month	Monthly Flow (MGD)	Flow per Capita (gpcd)
January	0.949	90.29
February	0.953	90.68
March	0.920	87.49
April	0.864	82.23
May	0.813	77.37
June	0.819	77.91
July	0.897	85.37
August	0.855	81.35
September	0.754	71.78
October	0.752	71.53
November	0.766	72.85
December	0.841	80.03

Flow Comparison to 1986 Sewer Master Plan

It is interesting to note and compare sewer flow conditions from the 1986 master plan report to current flow trends. In 1985, the average dry weather flow for the City of Morro Bay (excluding Cayucos SD) was reported at 1.31 mgd. The 1986 report described a "planning" population of approximately 10,660 at that time; it is believed this referenced population included summer tourist influence. The actual permanent residential population in 1986 was likely around 9,000 to 9,500 people. This would translate into a per capita wastewater flow of approximately 140 gpcd at that time. The 1986 report broke down per capita flow into components of residential, commercial, and motel/hotel. At that time, the residential wastewater flow component was 75 gpcd. In 2005, the City's population has grown to approximately 10,500 people, or a 10 percent increase from 1986. Current flow data shows a dry weather flow of 0.835 mgd, or approximately 80 gpcd overall (including all components of flow). Thus, the City's overall population increased 10 percent in the last 2 decades, while decreasing wastewater flows by 40 percent or more. This reduction in flows is likely due in part, to the aggressive water conservation measures that have taken place over the recent years. Another contributing factor to flow reduction at the plant was the replacement of the WWTP influent flow meter in August 2001, which reduced measured plant flows by

25%. It is likely that a number of the pipeline upgrade recommendations in the 1986 report will not be necessary due to this reduction in wastewater flows.

Peaking Factor Analysis

In order to appropriately design wastewater collection facilities, peak flow conditions must be quantified. Peak flow was determined based on the flow monitoring conducted for each basin in 2004 and 2005. The peaking factor for the diurnal peak does not include inflow/infiltration flow contributions to the collection system. The inflow/infiltration amounts were determined as a fixed amount based on the flow monitoring results of each basin. This value is assumed constant and will not increase for future conditions. In fact, it is hopeful that the I/I component will be reduced in the coming years. Table 3-3 shows the Inflow/Infiltration component of the flow and the dry weather peaking factor. Dry weather peaking factors were based on actual flow monitoring data, and increased somewhat to be conservative. The "A" basins that were predominantly residential in nature were all recommended at a 3.0 peaking factor to be conservative, based on the typical morning peak when people are preparing to leave for work. The flow conditions are summarized below:

ADF is obtained by averaging the mean daily flows conveyed by the Treatment Plant over a 27 month period.

PHWWF. The peak hour wet weather flow will include the peaking factor for dry weather plus the measured Inflow/Infiltration component. The peaking factor differs in basin B03 most likely because of the high commercial component of basin B03. Basin B03 feeds into and effects Basin TP.

MDDWF factor for the purposes of this study was derived from the historical MDDWF of 1.211 MGD (July 2003) and dividing by the average daily flow.

Peak Month factor based on historical data is 1.45.

Table 3-3. Dry Weather Peaking Factor and Inflow/Infiltration Component

Basin	Dry Weather Peaking Factor	Inflow/Infiltration Component (gpm) (measured)
A01a	2.0	659
A01b	3.0	31
A01c	3.0	320
A02	3.0	190
A03	3.0	145
A04	3.0	145
A05	3.0	245
A06	3.0	375
A07	3.0	950
B01	2.5	50
B02	2.5	50
B03	3.5	55
TP	2.0	0

FUTURE WASTEWATER FLOWS

Projection of wastewater flows is tied closely to population projections, and anticipated development, see Chapter 2 – Demographics. There are several methods for projecting future flows, based on population/per capita, and based on land use. For this master plan update, flow projections were based on a per capita calculation to future build-out population. This is a common method, and believed to be reasonably accurate since the majority of remaining development in the City will be residential in nature.

The build-out population of Morro Bay is 12,500. For the purpose of this report the future flow per capita is anticipated to remain at approximately 80 gpcd. Although future conditions of drought or wet years can influence this number, it is expected the City will continue to conserve water and new construction will include low flow fixtures thus maintaining current water usage demands. Using a projected base population of 12,500 persons and a base flow of 80 gpcd, the estimated future flow is approximately 1.0 MGD. As stated earlier, it is anticipated that build-out will be reached in approximately 16 years; however, accurate projections of growth rates are always difficult to predict.

For comparison, the U.S. EPA has established a historical average of 120 gpcd for wastewater flows. The breakdown of the historical average flow as established by the EPA is as follows: 70 gpcd for domestic flow, 10 gpcd for light commercial/industrial, and 40 gpcd for non-excessive infiltration (p. 200 M&E[7]). The City's overall per capita flow of 80 gpcd includes all flow components, and thus is substantially below the national average due mainly to the arid California region and water conservation efforts.

WATER EQUIVALENCY PROGRAM

The Water Equivalency Program was adopted in 1981 to conserve water. The program effectively limited the amount of new construction permitted in the City each year. As a condition of development, any new growth was required to develop an equivalent amount of water units by implementing plumbing retrofits to other homes and businesses. This program was discontinued in 2001 due to the addition of imported state water, approval of non-emergency operation of the desalination plant, and the success of the program resulting in essentially no more retrofit candidates remaining. This program was a significant contributor to reducing the overall wastewater flows to the City's WWTP. It is noted that the existing wastewater flows this year (0.83 mgd), are lower than in 1986 with less population (1.4 mgd) due to conservation efforts. This is a significant finding, and likely has enhanced the City's capacity to serve residents throughout the collection system.

STATE WATER

State water became available in 1997, however, no rate increase occurred at that time. Rates were increased in 1994 and 1995 to address the debt service for the State Water project. According to the City, there were no significant reductions in water consumption/sewer generation, as a result of the State Water Project.

RETROFIT REBATE PROGRAM

The City offers incentives ($100.00 rebate) to residents who replace old toilets with 1.6-gallon flush toilets, or purchase energy star washing machines that use low amounts of water.

CHAPTER 4

INFLOW AND INFILTRATION

During the winter months, the wastewater treatment plant experiences a substantial increase in wastewater flows, both from the City's collection system and the Cayucos SD system. In 2003, the City retained Wallace Group to conduct flow monitoring of each of the City's basins during the winter of 2004 to establish the magnitude of I/I coming from each basin. Unfortunately, the 2003-2004 rain season was very mild, which resulted in only one significant rain event during the two-month flow monitoring period. Wallace Group recommended to complete additional flow monitoring on specific basins during the following 2004-2005 rain season. This section provides an overview of the results from the I/I monitoring from both rain seasons.

INFLOW AND INFILTRATION DEFINITION

The inflow and infiltration (I/I) of storm water into a sewer system can result in peak flows that far exceed dry weather conditions. For the purposes of this report, these terms are defined as follows:

> *Infiltration* is the water entering a sewer system and service connections from groundwater, through such means as defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include inflow and is relatively constant over a period of days, weeks, or even months as high groundwater conditions persist.
>
> *Inflow* is the water discharged into a sewer system and service connections from such sources as roof drains, cellar, yard and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross connections from storm sewers, catch basins, storm water, surface runoff, or drainage. Inflow does not include infiltration. Inflow varies rapidly with rainfall conditions, with flows rising and falling within minutes or hours of a severe storm event.

2004 AND 2005 FLOW MONITORING RESULTS

Wallace Group conducted flow monitoring on eleven manholes from February 11, 2004 through April 1, 2004 to determine the extent of I/I problems in each of the City's wastewater basins. During the initial 60 days of flow monitoring there was only one significant rain event, 1.85" on Wednesday, February 25, 2004 followed by 0.24" of rain on Thursday, February 27, 2004. Based on the rain data, it appeared that the rain event on Wednesday produced heavy downpour in the mid-afternoon. Based on observations, the City had continuous rainfall from approximately 6:00 am through 5:00 pm. This one rain event did not provide enough information to determine the extent of the I/I problem. Wallace Group and the City agreed that another round of flow monitoring in the rainy season of 2005 on seven of the eleven manholes would be prudent to better assess I/I in the City's collection system.

Wallace Group conducted additional flow monitoring from January 19, 2005 through February 18, 2005 on the recommended manholes. In addition, Wallace Group extended the flow monitoring in two locations (Basins A05 and A07) for two more weeks. During the 2005 flow monitoring season, Wallace Group monitored Basins A05, A07,

A06 in two separate locations, A01c, A01a, A03 in two separate locations, and B01 & B02. Not all of the locations listed above were monitored the entire time. The flow monitoring for the 2005 season occurred from January 19, 2005 through February 18, 2005, during which the rainfall was moderate. There were two significant rainfall days that both occurred on a Friday, early in the morning when domestic flow was low. In addition, the rainfall was intense for a short duration. The first rain day was January 28, 2005 and produced 0.9 inches of rain and the second was February 18, 2005 and produced 1.0 inches of rain. Other rain days did occur throughout the monitoring period, however these rain events were characterized by longer duration, less intense or intermittent rains occurring throughout the entire day.

A description of each of the flow monitoring locations, a summary of the 2004 monitoring results and recommendations, and the results of the 2005 monitoring period and recommendations are provided below. Figure 3-1 illustrates the locations of the basins as well as the flow monitoring locations.

Morro Bay Rainfall Information

Rain measurements were obtained from three locations throughout the City, at the following locations:

- City Fire Station, 715 Harbor Street
- Elena Tank Site, Elena Street at Juniper Avenue
- Residence, 331 Luzon Street

The rainfall data was collected for the 2004 and 2005 monitoring periods, and significant rainfall events are summarized in Table 4-1. A comparison of average monthly rainfall to the Year 2004 and 2005 monthly rainfall totals (during which time the flow monitoring was conducted) is presented in Table 4-2.

Summary of Flow Measurement Data by Basin

Basin A01a

Basin A01a receives flow from Basins A01c, A05, A06 and A07. In 2004, the average flow was 240 gpm and the peak flow was 440 gpm. During the 2004 rain event, there were high levels of inflow, however, it was suspected that the inflow came from the other basins and that Basin A01a did not have a significant I/I problem, based on review of upstream inflow monitoring data. It was recommended to re-monitor this basin in 2005 to help determine the total flow from all of the basins, as well as determine if basin A01a does have a significant I/I problem. In 2005, the flow meter was placed in the same manhole as the previous year and was monitored from January 19, 2005 through February 18, 2005. The flow patterns in Basin A01a were very good. The first rain event showed approximately 475 gpm of inflow with approximately 50 gpm of lingering infiltration for approximately 24 hours. The second rain event showed approximately 750 gpm of inflow (before the meter malfunctioned). Flows for the 1/28/05 and 2/18/05 rain days, and a typical dry day (2/11/05), are shown on Figure 4-1. The meter was pulled after this last rain day, and thus prolonged inflow or infiltration could not be assessed. Table 4-3 provides a summary of the flow going to Basin A01a. This will help determine if the high inflow is from the upstream basins or if it is from Basin A01a.

Table 4-1. Summary of Significant Rainfall Events During Monitoring Period

Date	Rainfall, inches			Comments[1]
	Fire Station	Elena Tank	331 Luzon St	
2/4/04 to 2/16/04	0	0	0	Year 2004 monitoring period begins
2/17/04	1.08	1.20	0.86	Most rain fell 2 am to 4 am
2/18/04	0.01	0.30	0.32	6:15 am intense rain
2/25/04	1.57	1.40	1.85	Heavy rains throughout day from 7 am to 4 pm, with intermittent downpours
3/1/04	0.20	0.30	0.30	Intermittent light rain
3/2/04 to 3/31/04	0.30	0.30	0.10	Only rain day was March 25, 2005 during this period
1/19/05	0	0	0	Year 2005 monitoring period begins
1/28/05	0.01	1.1	0.9	(rain gage malfunction at City Fire Station)
1/29/05 to 2/10/05	0	0	0	Dry
2/11/05	0.31	0.25	0.2	
2/15/05	0.47	0.20	0.4	
2/17/05	1.23	0.50	0.5	
2/18/05	2.25	3.75	1.0	
2/19/05	0.21		0.5	1:30 pm intense downpour
2/20/05	0.54		0.25	
2/21/05	0.92		0.75	1:45 pm to 2:30 pm intense downpour
2/22/05	0.08		0.25	
2/27/05	0.55	0.70	0.5	
3/4/05	0	0	0	Metering interval ends

[1]All comments based on observations by Steve Tanaka, 331 Luzon Street

Table 4-2. Summary of Monthly Rainfall in Inches

	January	February	March
Average[1, 2]	3.0	3.5	3.0
Year 2004[1]	1.32	4.11	0.50
Year 2005[1]	5.88	6.73	2.89

[1]Based on City of Morro Bay Fire Station data.
[2]Average is based on historical data from 1959 to 2004.

Figure 4-1
Basin A01a

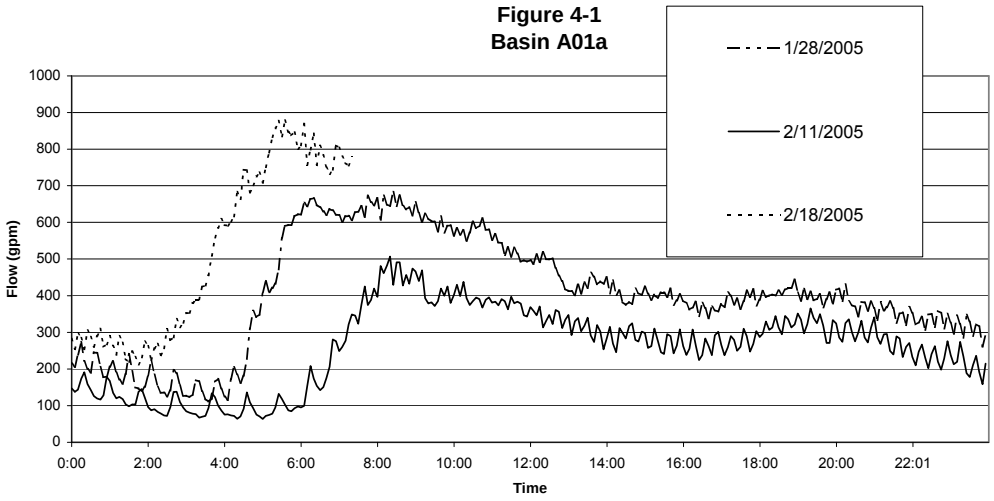


Table 4-3. Basin A01a Summary

Basin	January 28, 2005 Rain Event		February 18, 2005 Rain Event	
	Inflow	Infiltration	Inflow	Infiltration
A05	125	25	125	25
A07	375	0	475	0
A06	125	0	225	0
A01c	150	0	150	0
Total	**775**	**25**	**975**	**25**
A01a	475	50	750	50

According to Table 4-3, the total of the flows from the four basins was greater than the peak flow from Basin A01a, but the infiltration was higher in basin A01a than the other four basins. This could mean that Basin A01a does not have any significant inflow problems. The total is higher probably due to time delays in the readings and accuracy of flow measurement. The inflow amounts shown above are from the peaks of the storm, which occurred at the same time, however the peak would not be as significant once it got to Basin A01a because of the time for the peaks from the upstream basins to reach Basin A01a. The peak period for Basin A01a was significantly wider than the other four basins and this is due to the time delay. Basin A01a probably does not have a significant inflow problem as indicated in Table 4-3. Most likely every basin has some infiltration problem, but is not apparent in the actual basin readings, except Basin A05. It is recommended the City not focus I/I efforts on Basin A01a.

Basin A01b

Basin A01b is a stand-alone basin, which means it does not contribute flow to any other basin. It is comprised of all residential zoning. In 2004, the average flow in Basin A01b was 5 gpm. The peak dry weather flow was approximately 20 gpm. During the rain

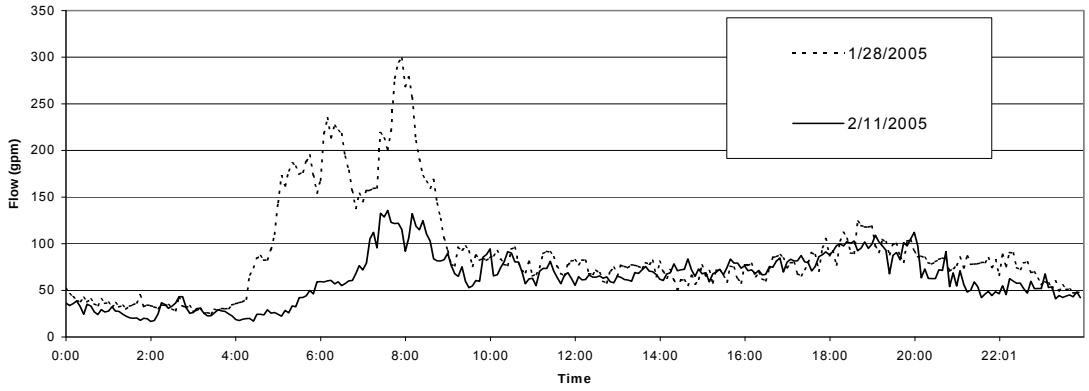
event inflow was noticed, however, the total inflow was minimal, approximately 30 gpm. This was six times average day flow, however, the flows were so minimal that it was recommended in 2004 to not be re-monitored in 2005.

Basin A01c

Basin A01c is a small basin that flows into Basin A01a. Basin A01c is comprised of residential zoning. In 2004, the average flow was 75 gpm and the peak dry weather flow was 150 gpm. During the rain event there was approximately 320 gpm of inflow from this basin. In addition, approximately 35 gpm of infiltration lingered. It was recommended to re-monitor this basin in 2005. In 2005, the flow meter was placed in the same manhole as the previous year and was monitored from January 19, 2005 through February 18, 2005. Normal flows appeared to be the same as the previous year. During the two big rain events, approximately 150 gpm of inflow was measured and flows returned back to normal fairly quickly. Basin A01c does have a problem with inflow and it is recommended the City continue efforts to determine the location(s) of the inflow problem, and to minimize wet weather flows.

Figure 4-2 shows the diurnal peaks for a typical day (Friday, 01/21/05) and a significant rain event (Friday, 01/28/05).

Figure 4-2
Basin A01c



Basin A02

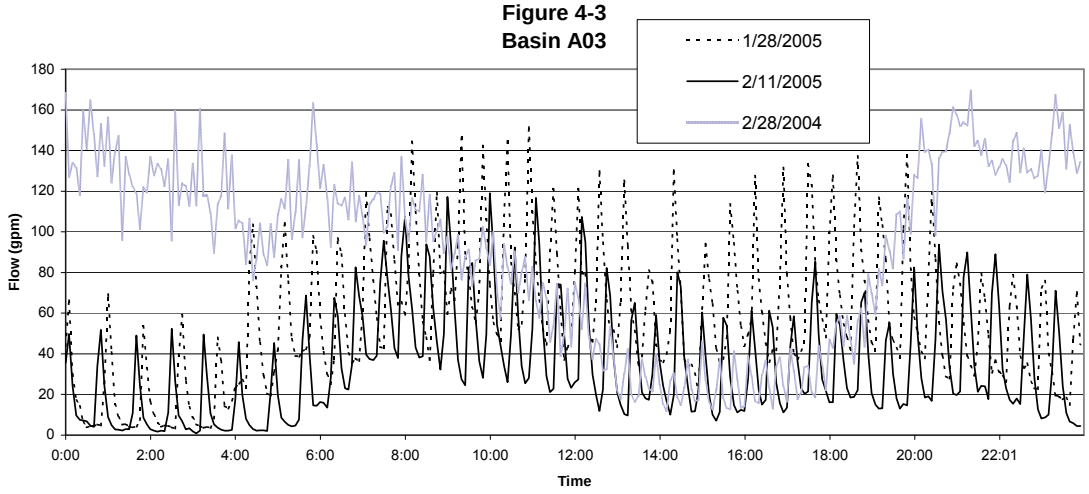
Basin A02 receives flow from Basins A03 and A04. Basin A02 is comprised of residential and commercial zoning. In 2004, the average flow to Basin A02 was 100 gpm and the peak dry weather flow was 225 gpm. The wet weather flow from Basin A02 was 315 gpm. This was 190 gpm more than typical flows seen at the time of the inflow, however, the majority of the inflow was suspected to have come from Basins A03 and A04. The infiltration noticed from Basins A03 and A04 also seemed to diminish in the monitoring of Basin A02 and therefore, it was determined that Basin A02 did not have a significant issue with infiltration. Therefore, it was recommended in 2004 that Basin A02 not be monitored in 2005.

Basins A03 and A04

Basin A03 receives flow from Basin A04 and then flows to Basin A02. These basins are comprised of predominantly residential and commercial land uses. The flow monitor

was located at the end of Basin A03 and therefore received flow from Basin A03 and A04. In 2004, the average day flow was approximately 75 gpm and peak dry weather flow was approximately 150 gpm. During the rain event in 2004, there was approximately 145 gpm, or approximately 2.8 times the flow during the rain event than a normal dry day and approximately 30 gpm of infiltration seemed to linger for approximately 12 hours. It was recommended to re-monitor this location in 2005. In 2005, the flow meter was placed in the same location from January 19, 2005 through February 14, 2005. The meter captured the first rain event and it did not show any signs of inflow or infiltration. This may be due to the fact that the 2004 rain event was a substantial rain (1.85 inches) throughout the day, causing more flooding, whereas the first rain event in 2005 was approximately 1 inch of rain. Figure 4-3 shows the flow patterns for 2/28/04 and 1/28/05, compared to a typical dry day (2/11/05). Flows remained at approximately 80 gpm average, similar to the year prior. From February 15, 2005 through February 18, 2005 the flow meter was moved to MH #14 on Kern, which was able to capture only gravity flow from a portion of Basin A03 in the Morro Heights area. This move captured the second rain event and did not show any signs of inflow or infiltration either. Basins A03 and A04 do not appear to have issues with inflow or infiltration, based on the flow monitoring data. However, in Chapter 6, Lift Station Analysis, A04 is shown to have a problem with infiltration. It is Wallace Group's recommendation to study this basin for further infiltration problems concerning Lift Station No. 3.

Figure 4-3
Basin A03

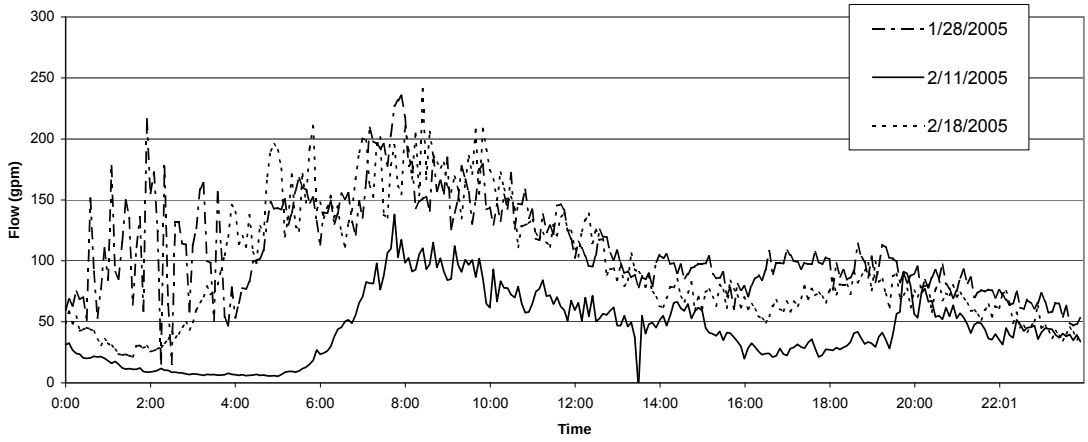


Basin A05

Basin A05 flows into Basin A07 and is comprised of mainly residential zoning with some commercial. In 2004 the average flow in basin A05 was 55 gpm and the peak flow was 120 gpm. The inflow into Basin A05 during the rain event was 295 gpm, which was 245 gpm more than the typical 50 gpm flow seen during the time of the rain event. No signs of infiltration were present as the elevated flows receded in less than an hour. It was recommended to re-monitor this basin in 2005 due to the significant inflow measured in 2004. From January 19, 2005 through February 10, 2005, the flow meter was placed in the same manhole as 2004. The manhole had very low velocities and high levels of flows. Material hung up in the manhole, which caused the meter to spike continuously throughout this time period. The first rain event on January 28, 2005 was

apparent and showed about 125 gpm of inflow. In addition, flows did not appear to go back to normal for approximately 12 hours. Flows stayed approximately 25 gpm higher than normal during this 12 hour period. Figure 4-4 shows the flow patterns for 1/28/05 (rainfall event was evident in early morning hours), 2/18/05, compared to a typical dry day (2/11/05). The meter was moved upstream from February 10, 2005 through March 2, 2005. The move was an attempt to find a manhole that may not cause sporadic readings. In addition, it was determined in analyzing the flow patterns from Basin A05 and Basin A07, downstream of Basin A05, the flow readings in Basin A05 were approximately the same as or higher than Basin A07 flows in some instances. This is a problem since the readings in Basin A07 should read approximately double that of Basin A05. Moving the flow meter did help the readings from being as sporadic, however the patterns and volumes stayed the same. The meter in Basin A05 was also replaced several times and re-calibrated to ensure the readings were accurate. The second rain event was captured in the new location. It showed the same intensity of inflow and infiltration as the first rain event. In addition, Basin A05 showed notable signs of inflow during almost every rain event, regardless of the intensity of the rain event. Basin A05 does have a significant inflow problem and should be further investigated by the City to determine the extent of, and reduce the I/I problem.

Figure 4-4
Basin A05

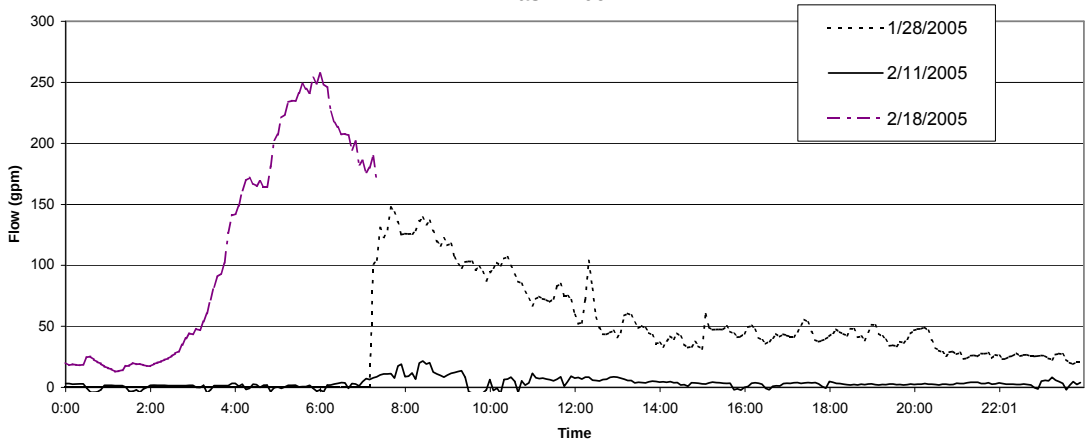


Basin A06

Basin A06 flows to Basin A01a through Lift Station 1. Basin A06 is comprised of only residential zoning. The flow meter location was chosen to focus on the older beach tract, as the Cloisters area is expected to have very little contribution to I/I (being a new development). In 2004, the average flow in Basin A06 was 30 gpm and the peak flow was 95 gpm. During the rain event in 2004, Basin A06 had a peak flow of 420 gpm, or 380 gpm over the typical flow at the time of the rain event of 40 gpm. Due to the significant amount of inflow, it was recommended to re-monitor this basin in 2005. In 2005, the flow meter was placed in the same manhole as the previous year. The meter read sporadically at the beginning of the monitoring period due to large objects such as rags hanging up on the flow meter. Basin A06 was monitored from January 19, 2005 through February 18, 2005. The first rain event showed approximately 125 gpm of inflow occurring during the tail end of the peak portion of the storm. The meter was not working during the true peak of the storm. During the second rain event, the meter

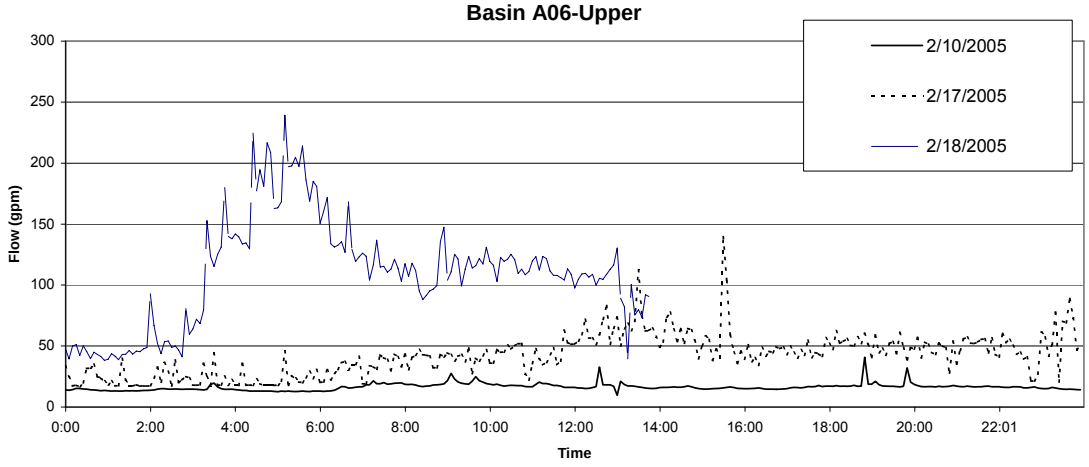
showed approximately 225 gpm of inflow. Other rain events did show inflow, but not as significant as these. Infiltration or lingering inflow was also apparent in Basin A06. Flows stayed above normal for at least 24 hours. Refer to Figure 4-5 which depicts the flows and peaks observed on 1/28/05, 2/18/05, and a typical dry day (2/11/05). Although for both rain days the flow monitoring data was only partial during the peak, the magnitude of the inflow was apparent.

Figure 4-5
Basin A06



The City requested that Wallace Group move one meter upstream in Basin A06 half way through the monitoring period to see if the inflow problem was occurring in the northern or southern portion of Basin A06. The flow meter was located in the upper portion of Basin A06 from February 9, 2005 through February 18, 2005. The normal level readings were not as accurate in this basin due to the very low flows and low velocities, however, during rain events, levels and velocities picked up and the meter was reading accurately. During the second rain event, the flows read approximately 175 gpm over normal. This means that 75% of the total basin inflow came from the northern portion (approximately 1/3 of the flow) of Basin A06. Basin A06 does have a significant inflow and infiltration problem. It is recommended the City prioritize their efforts on the northern portion of the basin to determine the location of the problems, and to reduce I/I from this region. Figure 4-6 shows the flow patterns for a typical dry day (2/10/05) and two significant rain events (2/17/05 and 2/18/05).

Figure 4-6
Basin A06-Upper

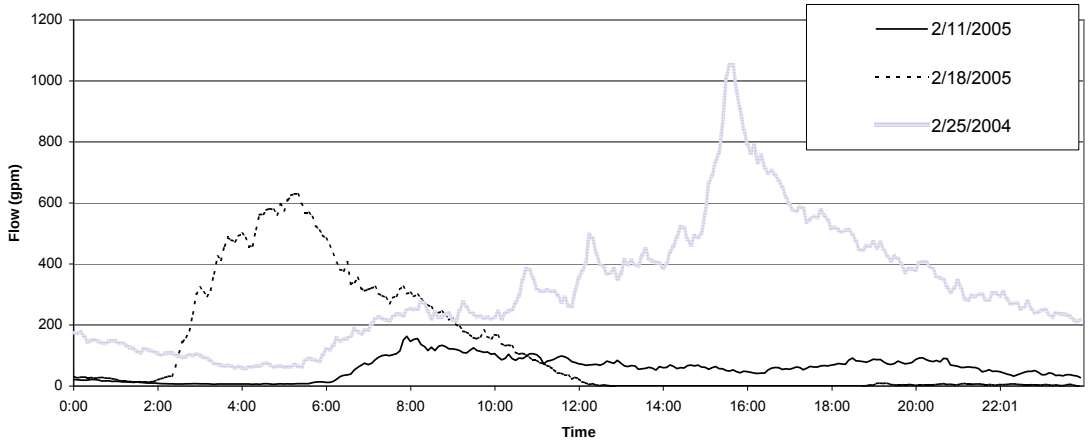


Basin A07

Basin A07 receives flow from Basin A05 and flows to Basin A01a. Similar to Basin A05, Basin A07 is comprised of mainly residential with some commercial. The flow readings for this Basin were reading low since the flows were nearly identical to that of Basin A05, which is upstream of Basin A07. It was determined however, that the flow patterns were accurate. During the rain event in 2004, a surcharge in the sewer main most likely occurred and flows read 1,000 gpm over normal. It was recommended to re-monitor this manhole to determine if this was truly happening during every rain event. In 2005 the flow meter was placed in the same manhole as the previous year and was monitored from January 19, 2005 through March 2, 2005. Again, the flow meter appeared to read the flow accurately in terms of the pattern of the diurnal curve, however, it seemed like the flows were low for the number of houses and the readings that came from Basin A05. Meters were changed out during the course of the flow period. However, as requested by the City, the meter remained in the same manhole. The readings did not change between meter replacements. During most of the rain events, increases in flow levels were very apparent, reaching 400 to 500 gpm of inflow. Other rain events produced flows on the order of 150 gpm. A portion of the inflow comes from Basin A05, approximately 125 gpm, therefore, 375 gpm to 475 gpm was coming from Basin A07. Lingering infiltration did not seem as apparent in Basin A07 as it did in Basin A05. It appears that Basin A07 has a more serious inflow problem than Basin A05. It is recommended the City further investigate this basin to determine the location(s) of the I/I problem, and to reduce I/I from this basin.

Figure 4-7 shows the flow patterns for a typical dry day (2/11/05) and two significant rain days (2/18/05 and 2/25/04).

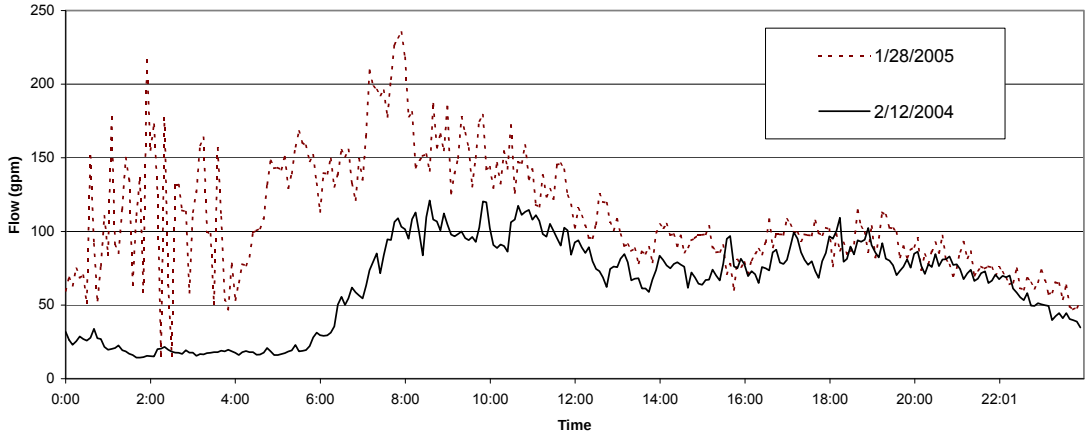
Figure 4-7
Basin A07



Basins B01 and B02

Basins B01 and B02 were monitored together at the downstream end of Basin B01. Both basins are comprised of mainly residential zoning. Basins B01 and B02 were monitored in 2004, however, the meter was not functioning at the time of the rain event. During another rain event in 2004, there did not appear to be any inflow or infiltration. Since the major rain event in 2004 was not captured, it was recommended to re-monitor the Basins in 2005. Basin B01 and B02 were monitored in the same manhole as 2004 from January 19, 2005 through February 7, 2005. The average flow for these two basins was 70 gpm and the peak dry weather flow was 150 gpm. The meter captured the first rain event. It appeared to have approximately 50 to 75 gpm of inflow (with short peaks of 100 gpm) during the downpour early in the morning, but quickly went back to normal flows immediately following. Since the inflow appeared to be relatively minor and infiltration was not noticed, it was recommended to move the meter to another basin experiencing high inflow problems. Therefore, on February 7, 2005 the meter was moved and the second rain event was not captured for Basin B01 and B02. Basins B01 and B02 do have a slight inflow problem, but infiltration does not appear to be an issue. This is consistent with the lift station pump run times discussed in Chapter 6. Figure 4-8 depicts a typical dry day compared to the 1/28/05 rain day.

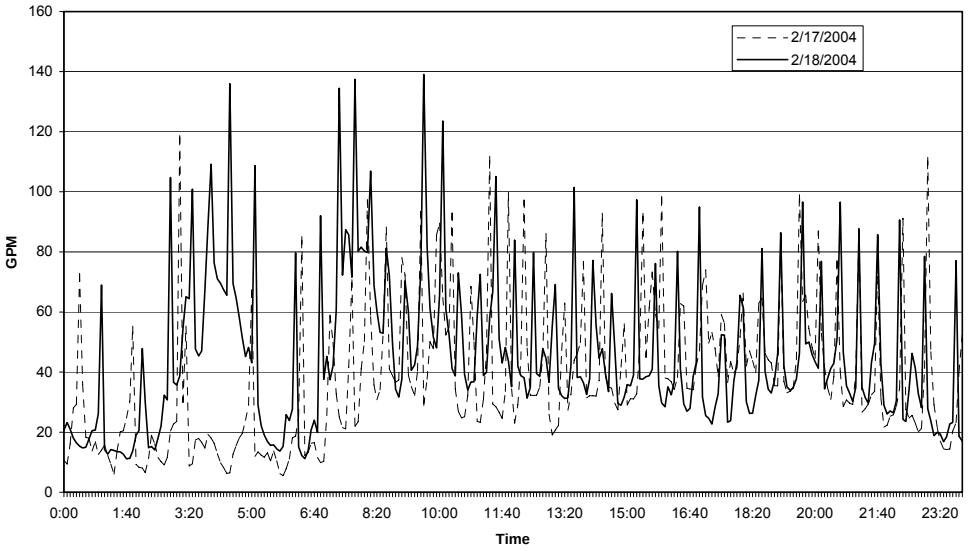
Figure 4-8
B01/02



Basin B03

Basin B03 flows to Lift Station 2, which is pumped to the Treatment Plant. Basin B03 is comprised of mainly residential zoning. In 2004 the average flow for Basin B03 was 30 gpm and the peak dry weather flow was 115 gpm. Basin B03 saw approximately 55 gpm of inflow and did not experience any lingering infiltration. The inflow appeared to come from the gravity portion of the basin and not from the upstream lift station that served a hotel, restaurant and park. This observation was based on seeing no signs of increased pumping cycles during the rainfall period. The 2004 data for this Basin was deemed accurate and sufficient, and it was recommended in 2004 to not monitor this basin in 2005. Figure 4-9 depicts a 02/17/04 rain day (rain intermittent throughout the day) which showed little to no inflow, and the 02/18/04 rain (short intense rainfall early morning hours) which showed "short-lived" inflow before flow patterns returned back to normal.

Figure 4-9
Basin B03



Basin TP

Basin TP has a small gravity fed zone, as well as receiving all of the flow from Basins B01, B02, and B03 through Lift Station 2. Average flow in the TP Basin is 120 gpm and the peak flow is 550 gpm. The gravity zone did not show any significant I/I in the monitoring for 2004 and therefore it was recommended to not be monitored in 2005.

SUMMARY OF RECOMMENDATIONS

After two years of flow monitoring, the basins that provide the City with the most inflow and infiltration problems have been determined. The ranking is based on the magnitude of I/I problems. The severity of the inflow and infiltration has been ranked as follows (1 being the most severe I/I problem):

1. Basin A07
2. Basin A06 Upper
3. Basin A01c
4. Basin A05
5. Basin A06 Lower
6. Basin A04
7. Basin A03
8. Basins B01 and B02
9. Basin B03
10. Basin A01b
11. Basin A02
12. Basin A01a
13. Basin TP

As mentioned in Chapter 3, it is also recommended that the overall I/I problem be addressed, and thus the Cayucos SD should also actively pursue means of reducing and controlling I/I to the joint wastewater treatment plant.

Goals for Infiltration/Inflow Reduction Program

All wastewater collection systems will exhibit some degree of inflow and infiltration, and the degree and extent of these sources of flow depend on a number of factors, including age of the collection system, materials of construction, groundwater and drainage conditions, and others. The City's I/I reduction efforts will benefit not only the collection system, but the operations at the treatment plant as well. Identification and mitigation of all sources of I/I is not an easy task, and takes time and effort to conduct necessary detailed investigations. However, focusing on the priority areas where significant I/I issues were identified is a good start to addressing the impacts associated with infiltration/inflow. It is difficult to determine a realistic goal for I/I reduction, and the benefits relative to costs for I/I reduction must be weighed. However, with focused efforts on reducing I/I in these identified areas, the City should expect to achieve an overall 40 to 50% reduction in I/I, at a reasonable cost. This degree of I/I reduction is significant, and certainly would benefit the City by preserving collection system capacity for needed sanitary service, and averting added treatment costs at the wastewater plant.

CHAPTER 5

COLLECTION SYSTEM ANALYSIS

This Chapter presents the analysis of the gravity sewage collection system for the City of Morro Bay. For evaluation of the City's three lift stations, refer to Chapter 6.

INTRODUCTION

The Morro Bay collection system consists of 6-inch to 27-inch gravity sewer lines, and three lift stations. All lines 8-inches and larger were modeled. Figure 5-1 depicts the gravity sewer lines that were modeled.

The analysis of the sewage collection system is based on a pipeline layout derived from the CH2MHILL atlas and database dated 1984, the current City atlas sheets dated March 2003, and record drawings.

A model of the existing collection system was created in spreadsheet format, as discussed below. The existing and projected flows are assigned to the model to create two scenarios for each drainage area. The scenarios are as follows:

- *Existing* – The Existing Scenario represents the flows based on the recent historical recorded ADF, PHWWF, and maximum day flow, and represents the conditions encountered in 2004/2005.

- *Build-out* – The Build-out Scenario represents the anticipated flow (ADF, PHWWF, maximum day) when the City reaches the full build-out as described in Chapter 2.

In addition, the joint Morro Bay/Cayucos sewer was evaluated based on "40 percent" capacity in the line being used by Cayucos Sanitary District. The 40 percent flow was defined based on the most restrictive reach of sewer, which was determined to be a flat 12" gravity sewer (Main Street, between Mindoro and Luzon Streets) with a corresponding d/D of 0.75 at peak hour dry weather flow. This flow value was determined as 242 gallons per minute. For comparison purposes, this flow rate would be equal to 0.35 mgd peak flow, or 0.17 mgd ADWF with a diurnal peaking factor of 2.0. Currently, Cayucos SD generates approximately 0.29 mgd wastewater flow on an ADWF basis.

CITY OF
MORRO BAY

Sewer System Master Plan Update

Figure 5-1

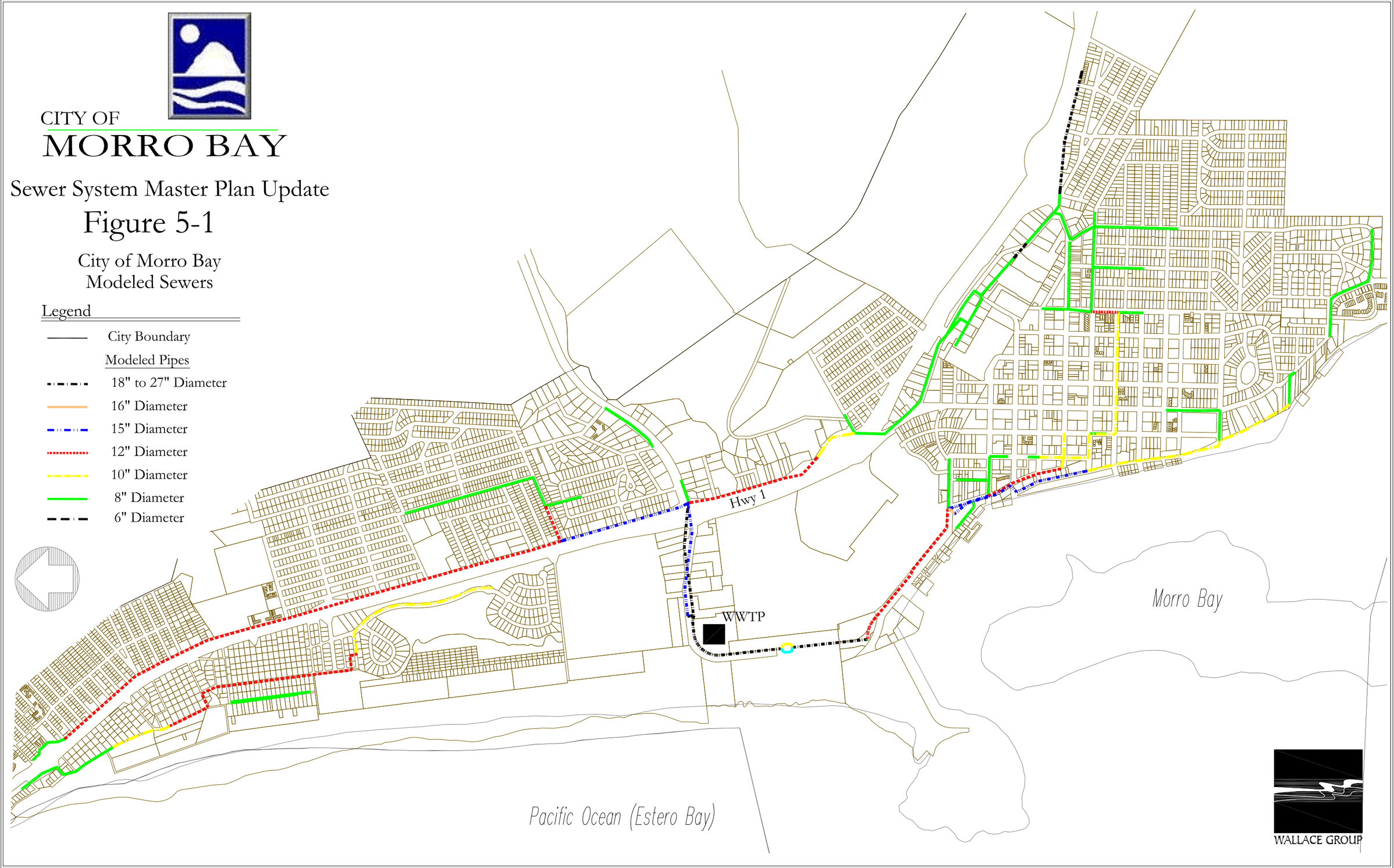
City of Morro Bay
Modeled Sewers

Legend

- City Boundary

Modeled Pipes

- 18" to 27" Diameter
- 16" Diameter
- 15" Diameter
- 12" Diameter
- 10" Diameter
- 8" Diameter
- 6" Diameter



WALLACE GROUP

COLLECTION SYSTEM PARAMETERS

The following design criteria were applied to the collection system:

- Maximum Capacity –

Flow Condition	Allowable Flow Depth
Average Dry Weather Flow	0.5
Peak Dry Weather Flow	0.75
Peak Wet Weather Flow	0.9

- Minimum Velocity – 2.0 feet per second (fps) during Average Daily Flow
- Maximum Velocity – 10.0 fps should not be exceeded in the pipeline;
- Manning coefficient n = 0.010 for PVC, n = 0.014 for VCP.
- All new sewers are modeled as PVC pipe (n=0.010), unless otherwise indicated.

These design flow parameters are accepted industry-wide standard criteria for design flow conditions. These design criteria provide capacity buffer from surcharge conditions, for fluctuations in flows from diurnal variations and reasonably anticipated peak wet weather flows.

System deficiencies and specific requirements for the respective drainage areas are discussed in the Results section of this Chapter.

COLLECTION SYSTEM MODELING

In order to evaluate the performance of the existing and build-out collection system, a spreadsheet was developed by Wallace Group to calculate pipeline capacity, flow velocity, and d/D ratio. This spreadsheet program utilized Manning's Equation for Circular channel flow in conjunction with information from the Circular Channel Ratios graph for d/D vs. Q/Q_{full}. This graph relates the ratio of depth of flow to the diameter of pipe (d/D) and the ratio of the actual flow rate to the full capacity flow rate (Q/Q_{full}).

The collection system model cross-references the City's old Atlas Map manhole numbering system, the City's new Atlas Map (2003) manhole numbering system (GIS), and a third Wallace Group numbering system. The Wallace Group numbering system was developed in the model for ease of reference to the direction and sequence of flow from manhole to manhole. A cross-reference sheet to all three numbering systems is included in Appendix B.

Flow Breakdown

Flows are assigned to the sewer system based on estimated flows from Chapter 3, existing and build-out population, described previously. Tables 5-1 and 5-2 show the estimated flows for each drainage area, as assigned in the collection system model for existing and future flows, respectively. The collection system was analyzed based on average dry weather flow, peak dry weather flow (at maximum day) and peak hour wet weather flow conditions for existing and build-out flow conditions. Flows were dispersed throughout the collection system based on number of connections and connection types

(residential, commercial, etc.). Details of how wastewater flows were distributed in the model are summarized as follows:

- Existing and future ADWF is distributed in the model based on connection type (SFR, MFR, commercial, etc.), and are added to manhole locations corresponding to their physical location in the collection system.
- As discussed earlier in this report, I/I is assumed to remain constant in the future build-out condition, and hopefully will decrease as a result of I/I mitigation measures. Wet weather flow peaks are added to the model, based on actual measured inflow peaks during the 2004 and 2005 flow monitoring. Where wet weather peaks flow from one basin to another, inflow peaks are added together. This creates a conservative peak inflow value, assuming that the additive peaks are timed to coincide with one another. This will be important to keep in mind as decisions are made whether a pipeline upgrade in the future is necessary or not.
- Wet weather flows in each individual basin are distributed to the collection system by taking the total inflow amount, and distributing the flow equally amongst the manholes of the modeled sewers.
- Peak dry weather flows are calculated based on the dry weather flow value, multiplied by the diurnal peaking factor.

Table 5-1. Collection System Model Flows (gpm) by Drainage Basin – Existing

Basin	Average Flow	Basin Dry Weather Peaking Factor	Peak Hour Dry Weather Flow	Max Day Average Flow	Max Day Peak Hour Dry Weather Flow	Inflow Peak Value	Wet Weather Peak Flow
A01a	26	2.0	52	32	64	25	77
A01b	13	3.0	39	19	57	31	70
A01c	45	3.0	135	55	165	150	285
A02	66	3.0	198	76	228	190	388
A03	74	3.0	222	88	264	145	367
A04	24	3.0	72	28	84	145	217
A05	32	3.0	96	40	120	125	221
A06	47	3.0	141	56	162	225	366
A07	53	3.0	159	65	195	420	579
B01	46	2.5	115	79	198	50	165
B02	49	2.5	123	66	165	50	173
B03	56	3.5	196	79	277	55	251
TP	48	2.0	72	70	105	0	72
TOTAL	**580 (0.84 mgd)**	---	---	**753 (1.08 mgd)**	---	**---**	**---**

Table 5-2. Collection System Model Flows (gpm) by Drainage Basin – Future

Basin	Average Flow	Basin Dry Weather Peaking Factor	Peak Hour Dry Weather Flow	Max Day Average Flow	Max Day Peak Hour Dry Weather Flow	Inflow Peak Value	Wet Weather Peak Flow
A01a	26	2.0	52	32	64	25	77
A01b	14	3.0	42	20	60	31	73
A01c	46	3.0	138	56	168	150	288
A02	82	3.0	246	123	369	190	436
A03	79	3.0	237	101	303	145	382
A04	28	3.0	84	33	99	145	229
A05	92	3.0	276	112	336	125	401
A06	48	3.0	144	57	171	225	369
A07	54	3.0	162	66	198	420	582
B01	54	2.5	135	92	230	50	185
B02	52	2.5	130	70	175	50	180
B03	62	3.5	217	90	315	55	272
TP	51	2.0	77	74	111	0	77
TOTAL	**690 (0.99 mgd)**	---	---	**925 (1.33 mgd)**	---	**---**	**---**

- Peak wet weather flows are calculated by adding the wet weather flow component for that basin to the peak dry weather flow value. The peak dry weather flow for average flow conditions was used, not the maximum day flow condition (which occurs in the summer).
- A special model run was conducted to address the Morro Bay/Cayucos joint sewer. The 12" and 15" joint sewer was evaluated under conditions where the Cayucos SD contribution of 40% flow is included, and excluded from the flow analysis. The 40% flow component was calculated based on the pipe reach of lowest flow capacity (12" sewer) at a d/D of 0.75, as described earlier in this report.

RESULTS

For both existing and build-out flows, the inflow/infiltration wet weather flow (added on top of dry weather diurnal peak flow) component contributes to many of the pipeline hydraulic deficiencies. This is important to note as consideration is given to whether a particular gravity sewer needs replacement in the future or not. The majority of the City's collection system is adequate for existing and future build-out conditions under peak dry weather flow conditions. This is due, in part, to the fact that the City's current wastewater flows have decreased by nearly 40 percent from 2 decades ago.

Further discussion is provided later in this chapter relative to the need for pipe upgrades should I/I be reduced or not. The discussion of model results will first focus on the Morro Bay/Cayucos SD joint sewer, and other pipeline deficiencies.

Morro Bay/Cayucos Joint Sewer

The collection system model and hydraulic evaluation focused specifically on the unique situation with this Morro Bay/Cayucos joint sewer. This sewer begins on Main Street at Vashon Street (north Morro Bay, east of Highway 1) as a 12" gravity sewer, extending south on Main Street to Highway 41 (12" to 15" sewer), then heading west on Atascadero Road (Hwy 41) to the WWTP (18" to 27" sewer).

As stated earlier in this report, the Cayucos SD owns 40% capacity in this sewer. The definition of 40% capacity is not clearly defined in the joint powers agreement; however, for the purposes of this evaluation, it was assumed that the Cayucos SD could discharge a 40% share of the available hydraulic capacity in this joint sewer (242 gpm), at any given time during the day. For modeling purposes, it must be assumed that the corresponding 40% capacity flow of 242 gpm is the peak or maximum flow allowable to meet this capacity criterion. Thus, it was assumed that the Cayucos SD would not discharge more than this flow value, although it certainly would be possible given the capacity of their upstream lift station. The Cayucos SD lift station pumps discharge around 2,000 gpm, and thus would surcharge this joint sewer during each pumping cycle.

Table 5-3 summarizes the Morro Bay/Cayucos joint sewer model flow runs for existing and future flow conditions, for the three hydraulic flow scenarios:

- Average day dry weather flow times hydraulic peaking factor
- Average day dry weather flow times hydraulic peaking factor, plus I/I component
- Maximum day dry weather flow times hydraulic peaking factor

The model hydraulic evaluation was also run for two conditions:

- Assuming that Cayucos SD is discharging 40% capacity, or 242 gpm
- Assuming that Cayucos SD is not discharging their capacity share

The scenario of Cayucos discharging to this sewer, or not, was important to evaluate, to determine what influence the Cayucos capacity component bears on capacity constraints, and the timing of and need for future upgrade/upsizing of this joint sewer. Figures 5-2 and 5-3 depict deficiencies in this joint sewer under the two flow conditions of discharge/no discharge from Cayucos SD, respectively.

General observations are summarized as follows:

- It is apparent from Table 5-3 that during dry weather flow conditions, existing and future, and within Basins A05, A07 and A01a, this joint sewer is of sufficient capacity when Cayucos SD <u>is not</u> discharging to this sewer.

Table 5-3 Summary of Hydraulic Deficiencies in Morro Bay/Cayucos Joint Sewer

Basin	Pipeline Segment					Existing Conditions						Future Conditions					
						Excluding 40% Cayucos Flow Component			Including 40% Cayucos Flow Component			Excluding 40% Cayucos Flow Component			Including 40% Cayucos Flow Component		
						[ADWFxPF] (d/D>0.75)	[ADWFxPF]+I/I (d/D>0.9)	[Max Day x PF] (d/D>0.75)	[ADWFxPF] (d/D>0.75)	[ADWFxPF]+I/I (d/D>0.9)	[Max Day x PF] (d/D>0.75)	[ADWFxPF] (d/D>0.75)	[ADWFxPF]+I/I (d/D>0.9)	[Max Day x PF] (d/D>0.75)	[ADWFxPF] (d/D>0.75)	[ADWFxPF]+I/I (d/D>0.9)	[Max Day x PF] (d/D>0.75)
	Upstream	Downstream	GIS Numbering Upstrm	GIS Numbering Dwnstrm	Diameter	d/D	d/D	d/D	d/D	d/D	d/D	d/D	d/D	d/D	d/D	d/D	d/D
A05	A05-AA01	A07-AA09	G6-003	G6-005	12										0.778	0.910	0.856
	A05-AA02	A05-AA01	G5-012	G6-003	12												0.750
	A05-AA03	A05-AA02	G5-009	G5-012	12												0.788
	A05-AA04	A05-AA03	G5-002	G5-009	12												
	A05-AA05	A05-AA04	F5-025	G5-002	12												
	A05-AA06	A05-AA05	F5-022	F5-025	12												
	A05-AA07	A05-AA06	F5-017	F5-022	12												
	A05-AA08	A05-AA07	F5-014	F5-017	12										0.750		0.772
	A05-AA09	A05-AA08	F5-009	F5-014	12												
	A05-AA10	A05-AA09	F5-005	F5-009	12												
	A05-AA11	A05-AA10	F5-001	F5-005	12												
	A05-AA12	A05-AA11	E5-026	F5-001	12												
	A05-AA17	A05-AA16	E5-010	E5-013	12												
AO7	A07-AA01	A01a-AA13	J6-006	J6-008	12		0.910 (s)			0.910 (s)	0.819		0.910 (s)	0.778	0.910	0.910 (s)	0.910 (s)
	A07-AA02	A07-AA01	I6-020	J6-006	12		0.910			0.910 (s)	0.772		0.910 (s)	0.756	0.910	0.910 (s)	0.910
	A07-AA03	A07-AA02	I6-014	I6-020	12		0.910			0.910 (s)	0.772		0.910 (s)	0.756	0.910	0.910 (s)	0.910
	A07-AA04	A07-AA03	I6-006	I6-014	12					0.910			0.910			0.910 (s)	0.809
	A07-AA05	A07-AA04	H6-020	I6-006	12					0.910 (s)			0.910		0.835	0.910 (s)	0.910
	A07-AA06	A07-AA05	H6-013	H6-020	12											0.910 (s)	0.778
	A07-AA07	A07-AA06	H6-007	H6-013	12												
	A07-AA08	A07-AA07	H6-005	H6-007	12												
	A07-AA09	A07-AA08	G6-005	H6-005	12										0.761	0.910	0.835
A01a	A01a-AA02	TP-ATP05	M7-018	M7-022	15		0.910 (s)			0.910 (s)			0.910 (s)			0.910 (s)	
	A01a-AA03	A01a-AA02	M7-011	M7-018	15												
	A01a-AA04	A01a-AA03	M7-005	M7-011	15		0.910 (s)			0.910 (s)			0.910 (s)			0.910 (s)	0.767
	A01a-AA05	A01a-AA04	L6-006	M7-005	15		0.910 (s)			0.910 (s)			0.910 (s)			0.910 (s)	
	A01a-AA06	A01a-AA05	L6-004	L6-006	15		0.910			0.910 (s)			0.910			0.910 (s)	
	A01a-AA07	A01a-AA06	L6-002	L6-004	15		0.910			0.910 (s)			0.910			0.910 (s)	
	A01a-AA08	A01a-AA07	K6-015	L6-002	15												
	A01a-AA09	A01a-AA08	K6-012	K6-015	15		0.910			0.910 (s)			0.910			0.910 (s)	
	A01a-AA10	A01a-AA09	K6-008	K6-012	15		0.900			0.910			0.910			0.910 (s)	
	A01a-AA11	A01a-AA10	K6-003	K6-008	15											0.910	
	A01a-AA12	A01a-AA11	J6-012	K6-003	12					0.910			0.910			0.910 (s)	
	A01a-AA13	A01a-AA12	J6-008	J6-012	12		0.910			0.910 (s)			0.910			0.910 (s)	
TP	TP-ATP02	TP-ATP01	M6-001	M6-006	18	0.880	0.910 (s)	0.910	0.910	0.910 (s)	0.910	0.910	0.910 (s)	0.910 (s)	0.910 (s)	0.910 (s)	0.910 (s)
	TP-ATP03	TP-ATP02	M6-003	M6-001	18												
	TP-ATP04	TP-ATP03	M7-020	M6-003	18		0.910 (s)	0.860	0.761	0.910 (s)	0.860	0.767	0.910 (s)	0.910	0.880	0.910 (s)	0.910 (s)

Notes:

1. No deficiencies

2. d/D conditions of 0.910 signify a value of 0.91 or greater. **(s)** Signifies the model shows sewer is in surcharge condition under the given flow condition

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Sewer System Master Plan Update

Figure 5-2

Morro Bay/Cayucos SD
Joint Sewer Deficiencies
(Zero Discharge from Cayucos SD)

Legend

- Deficiencies - Existing PHWWF
- Deficiencies - Future PHWWF

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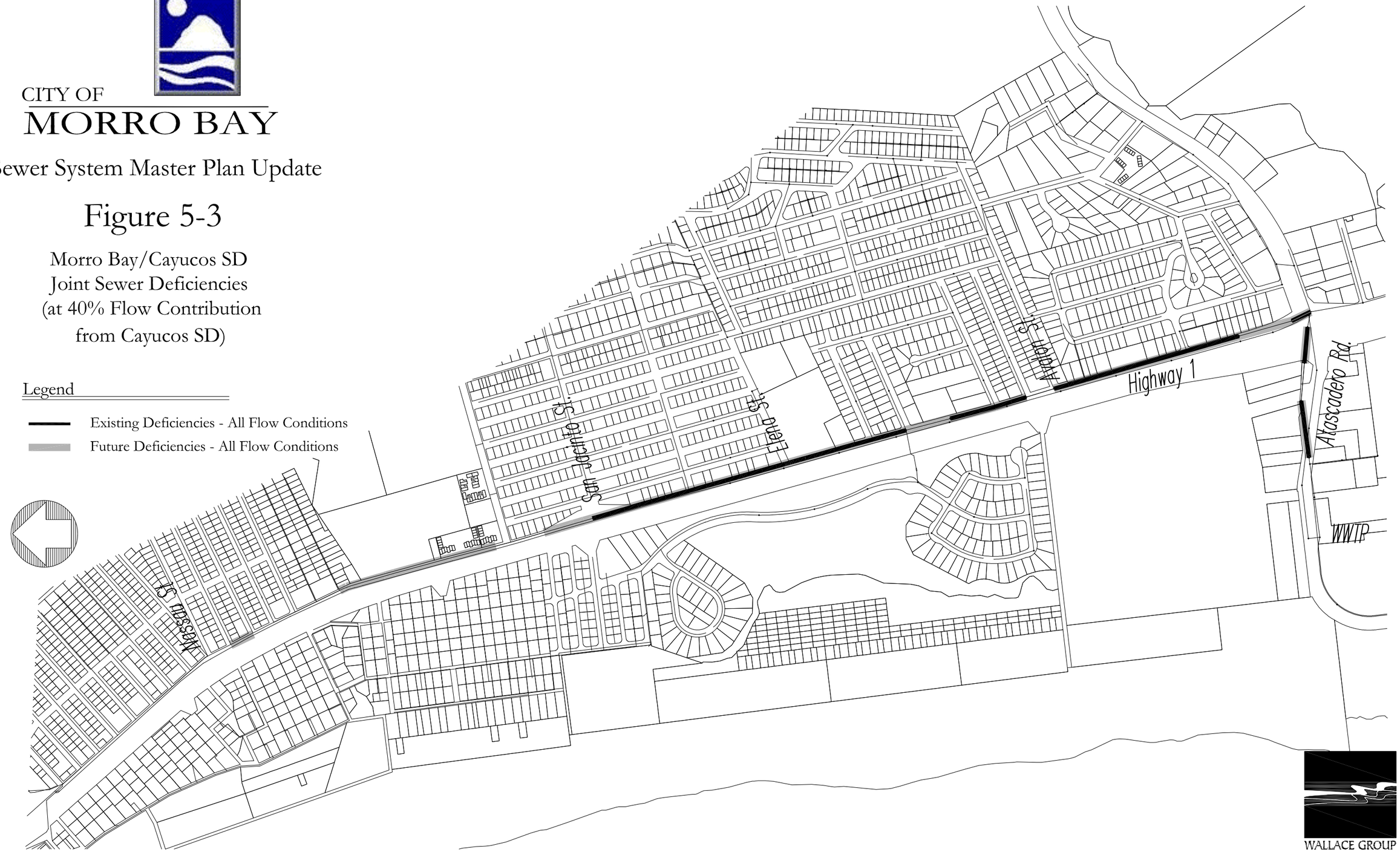
Sewer System Master Plan Update

Figure 5-3

Morro Bay/Cayucos SD
Joint Sewer Deficiencies
(at 40% Flow Contribution
from Cayucos SD)

Legend

- Existing Deficiencies - All Flow Conditions
- Future Deficiencies - All Flow Conditions



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- During existing dry weather flow conditions, and within Basins A05, A07 and A01a, this joint sewer is still of sufficient capacity when Cayucos SD is discharging to this sewer, with the exception of the reach from Elena Street (near Taco Temple) to San Joaquin Street. This marginal deficiency occurs during dry weather, maximum day flow conditions. Again, this determination is based on a Cayucos SD flow component of 242 gpm occurring at any time during the day.

- During future dry weather conditions, and excluding Cayucos SD flow, the same reach of sewer from Elena Street to San Joaquin Street, is marginally over capacity (exceeding d/D of 0.75) during average day flow conditions, but the remaining reach through Basins A05, A07 and A01a are of sufficient capacity.

- During future dry weather conditions, and including the Cayucos SD flow component, a significant portion of the 12" sewer in Basins A05 and A07 will exceed capacity criteria of d/D=0.75, and could possibly reach surcharge conditions in a portion of A07 during maximum day conditions.

- During wet weather flow conditions, the 12" and 15" joint sewer exceeds a d/D of 0.9, and in fact may be surcharging during peak periods of inflow. From the 2004/2005 flow monitoring, it is suspected that there are reaches that do in fact surcharge in Basin A07. The collective peak inflow through Basin A01a (which includes inflow from Basins A05, A06 and A07), is 800 gpm. Reduction in this peak inflow can diminish this peak wet weather flow capacity condition, possibly averting the need for a pipeline upgrade.

- In the TP Basin, the 18" gravity sewer should be replaced in the near future, as it is already over capacity during dry weather flow conditions. The sewer should be replaced to 27" diameter to match the recently replaced sewer in this same area. This should relieve some conditions of surcharge noted by City staff on immediate reaches upstream of this sewer.

With the scenario of no flows entering the joint sewer from Cayucos SD, no recommendations to upgrade this joint sewer in Basins A05, A07 and A01a would be made, so long as the I/I component can be reduced by 40 to 50 percent in Basins A05, A06, and A07. Close monitoring of the reach in the area of Elena Street to San Joaquin Street would be warranted, to assess the accuracy of model predictions of reaching d/D=0.8 at build-out.

With the scenario of including Cayucos SD flows to this joint sewer, with no other options to eliminate or reduce flow from Cayucos SD, it would be recommended to upsize all of the 12" sewer (from Vashon Street to Sequoia Street) to 15" diameter. However, the Cayucos SD need for joint capacity in this sewer should be resolved prior to any consideration of new sewer capacity.

Regardless of existing or future flow condition, or if Cayucos SD discharges to this joint trunk or not, it is recommended to replace the remaining portion of 18" sewer, from Hwy 41/Hwy 1 to the WWTP, as the model results indicate this 18" sewer approaches full pipe flow during dry weather peak flow conditions. This upgrade should be accomplished as soon as practicable, and definitely within the next 5 years.

Other Collection System Areas

As stated earlier, the majority of the City's collection system is adequate to serve existing and future wastewater flows. There are no noted deficiencies for existing or future conditions for Basins A01b, A01c, A06, B01, B02 and B03. Hydraulic deficiencies are summarized in Table 5-4, and shown on Figure 5-4.

The only areas noted with deficiencies are in Basins A02 and A03. These deficiencies are described as follows:

Basin A02. The gravity sewer main that shows deficiencies is generally described as the sewer beginning at Quintana Road/Main Street, and heading north on Main Street to Highway 41. At existing flow conditions, this entire gravity sewer is adequate at average dry weather flow conditions, but may be marginal during maximum day peak flows (summertime). During wet weather flow conditions, this sewer reach may be reaching full pipe flow in certain reaches, particularly on Main Street near Errol Street. A first attempt should be made to reduce I/I in this Basin, thus deferring a pipeline upgrade until later years. Based on the model, a 50% reduction in the I/I component in Basins A02 and A03 could defer this upgrade possibly through build-out. It is noted that this reach of sewer was also identified in the 1986 master plan to be upgraded (parallel sewer construction).

At build-out flows, this pipeline reach will exceed design criteria at average dry weather flow/peak hour conditions, and will also likely exceed flow criteria during maximum day flow conditions. It does not appear that this reach will surcharge; however, if not replaced, it would need to be monitored closely in the future. Given that in future years this sewer will approach d/D of 0.9 during maximum day/peak hour, it is recommended that the sewers in this reach be upgraded when the future average dry weather flow capacity reaches 0.75 d/D. This would amount to an increase in daily flow of 20,000 gpd, or approximately 130 equivalent units of development. Basins A02, A03 and A04 collectively are projected to increase in flow by 36,000 gpd at build-out. From Quintana Road to Manhole O7-005 (approximately 1,100 LF), this gravity sewer should be upsized to 12" diameter (or a parallel 8" sewer installed), and the 12" sewer from Manhole O7-003 to Highway 41 (approximately 1,600 LF) should be upsized to 15" sewer (or a parallel 8" sewer should be constructed).

Basin A03. There are several pipe reaches that show potential hydraulic deficiencies based on the hydraulic model. The first reach begins near Morro Bay Boulevard/Quintana Road (near the ARCO Mini-Mart and Mobil Station), on City Atlas S9, Manhole S9-001, extending to Manhole R9-009 (near McDonalds/Burger King), approximately 600 LF.

Table 5-4 Summary of Hydraulic Deficiencies [a]

Basin	Pipeline Segment					Existing Conditions			Future Conditions		
						[ADWFxPF] (d/D>0.75)	[ADWFxPF]+I/I (d/D>0.9)	[Max Day x PF] (d/D>0.75)	[ADWFxPF] (d/D>0.75)	[ADWFxPF]+I/I (d/D>0.9)	[Max Day x PF] (d/D>0.75)
	Upstream	Downstream	GIS Numbering Upstrm	GIS Numbering Dwnstrm	Diameter	d/D	d/D[b]	d/D[b]	d/D	d/D[b]	d/D[b]
A01b											
A01c											
A02	A02-AA02	TP-ATP05	N7-002	M7-022	12			0.746	0.742	0.910	0.910
	A02-AA03	A02-AA02	N7-004	N7-002	12		0.910	0.829	0.788	0.910 (s)	0.910
	A02-AA04	A02-AA03	N7-007	N7-004	12						0.761
	A02-AA05	A02-AA04	O7-002	N7-007	12						0.788
	A02-AA06	A02-AA05	O7-003	O7-002	12						0.798
	A02-AA08	A02-AA07	O7-007	O7-005	10			0.756	0.746	0.910	0.910
	A02-AA09	A02-AA08	P7-003	O7-007	10			0.756	0.746	0.910	0.910
	A02-AA10	A02-AA09	P7-007	P7-003	8		0.910	0.841	0.798	0.910	0.910
	A02-AA12	A02-AA11	P7-009	P7-008	8						0.788
	A02-AA13	A02-AA12	Q8-002	P7-009	8						0.835
	A02-AA14	A02-AA13	Q8-007	Q8-002	8			0.746	0.721	0.910	0.860
	A02-AA22	A02-AA21	R9-003	R9-002	8		0.910	0.819	0.761	0.910 (s)	0.910
A03[c]	A03-AA01	A03-AA02	S11-002	S11-001	6	0.910	0.910 (s)	0.910 (s)	0.910	0.910 (s)	0.910 (s)
	A03-AA02	A03-AA03	S11-001	S10-006	6	0.910	0.910 (s)	0.910 (s)	0.910	0.910 (s)	0.910 (s)
	A03-AA03	A03-AA04	S10-006	S10-004	6	0.910	0.910 (s)	0.910	0.910	0.910 (s)	0.910
	A03-AA04	A03-AA05	S10-004	S10-003	6	0.848	0.910		0.848	0.910	
	A03-AA05	A03-AA06	S10-003	S10-002	6	0.860	0.910		0.860	0.910	0.900
	A03-AA06	A03-AA07	S10-002	S10-001	6	0.848	0.910		0.848	0.910	
	A03-AA07	A03-AA08	S10-001	S9-003	8			0.809	0.761		0.910
	A03-AA09	A03-AA10	S9-001	R9-010	8						0.778
	A03-AA10	A03-AA11	R9-010	R9-009	8		0.910	0.809	0.746	0.910	0.900
	A03-AA11	A03-AA12	R9-009	R9-008	8	0.756	0.910	0.880	0.809	0.910 (s)	0.910
A06											
B01											
B02											
B03											

Notes:

No deficiencies

[a] For Morro Bay/Cayucos joint sewer results, refer to Table 5-3

[b] d/D conditions of 0.910 signify a value of 0.91 or greater. (s) Signifies the model shows sewer is in surcharge condition under the given flow condition

[c] The stretch from S11-002 to S10-001 which is a 6" ACP & VCP included the Lift Station 3 flows of 220 gpm

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Sewer System Master Plan Update

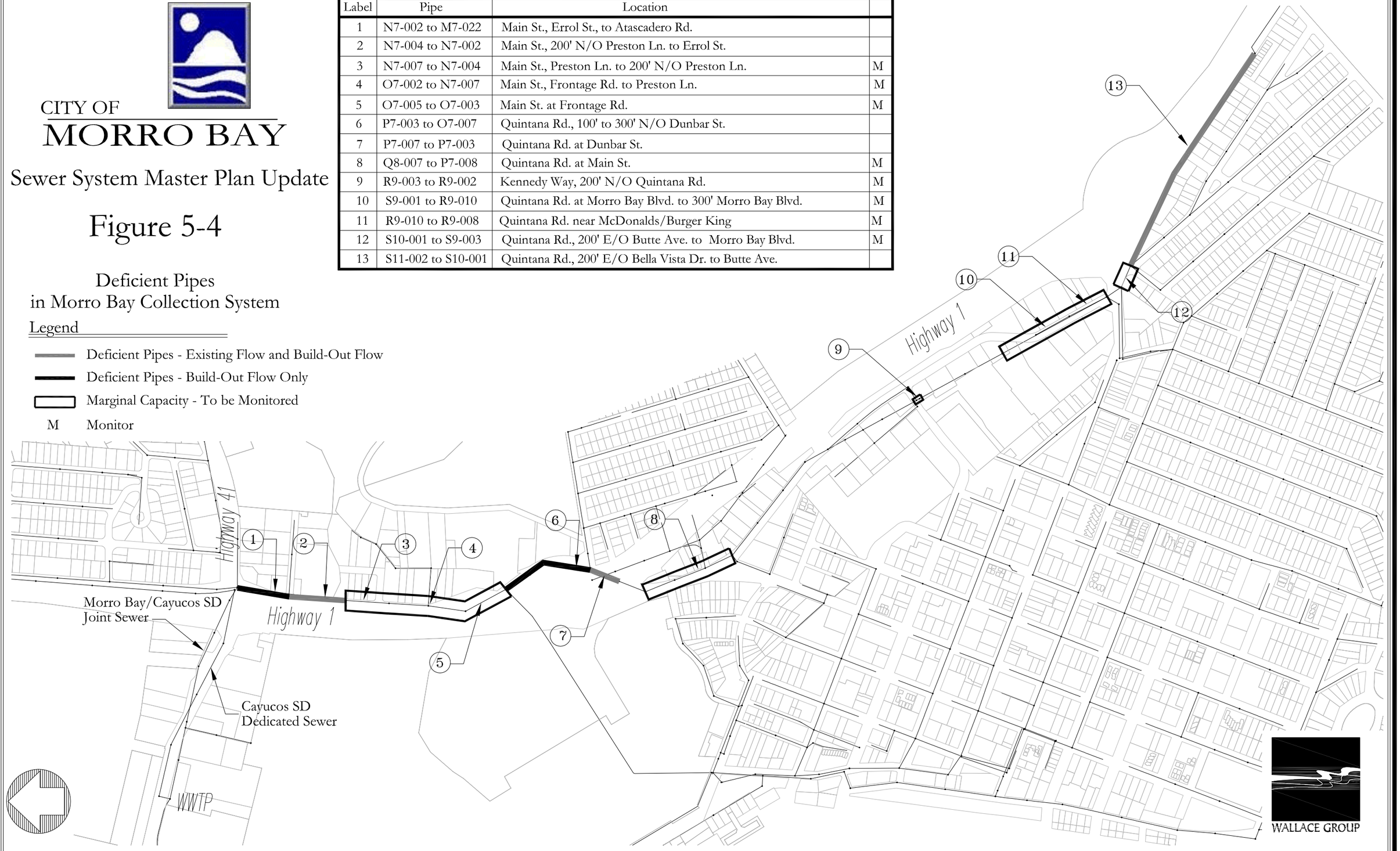
Figure 5-4

Label	Pipe	Location	
1	N7-002 to M7-022	Main St., Errol St., to Atascadero Rd.	
2	N7-004 to N7-002	Main St., 200' N/O Preston Ln. to Errol St.	
3	N7-007 to N7-004	Main St., Preston Ln. to 200' N/O Preston Ln.	M
4	O7-002 to N7-007	Main St., Frontage Rd. to Preston Ln.	M
5	O7-005 to O7-003	Main St. at Frontage Rd.	M
6	P7-003 to O7-007	Quintana Rd., 100' to 300' N/O Dunbar St.	
7	P7-007 to P7-003	Quintana Rd. at Dunbar St.	
8	Q8-007 to P7-008	Quintana Rd. at Main St.	M
9	R9-003 to R9-002	Kennedy Way, 200' N/O Quintana Rd.	M
10	S9-001 to R9-010	Quintana Rd. at Morro Bay Blvd. to 300' Morro Bay Blvd.	M
11	R9-010 to R9-008	Quintana Rd. near McDonalds/Burger King	M
12	S10-001 to S9-003	Quintana Rd., 200' E/O Butte Ave. to Morro Bay Blvd.	M
13	S11-002 to S10-001	Quintana Rd., 200' E/O Bella Vista Dr. to Butte Ave.	

Deficient Pipes in Morro Bay Collection System

Legend

- Deficient Pipes - Existing Flow and Build-Out Flow
- Deficient Pipes - Build-Out Flow Only
- Marginal Capacity - To be Monitored
- M Monitor



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This is an 8" diameter sewer, with no apparent deficiencies immediately upstream or downstream of this reach. However, it is noted that this sewer receives flow from Lift Station 3, and flow surges of 70 to 90 gpm were measured during the flow monitoring conducted in 2004/2005. The City should first attempt to reduce overall I/I in this Basin to reduce the potential for upgrade of this 8" sewer in the future. During existing maximum day conditions, the 8" sewer may reach d/D of 0.8 to 0.9, which is marginal; however, the City should continue monitoring this condition, as modeled flows are slightly on the conservative side. Aside from wet weather flow conditions, this reach of sewer appears adequate for future ADWF conditions (0.8 d/D at ultimate build-out would likely be deemed acceptable, upgrades not necessary), but may require upgrades to meet future maximum daily flow conditions. This should be further assessed in the coming years to verify if replacement of this sewer is warranted or not depending on future growth in the area. Also, according to the City Atlas maps, the sewer between Manhole R9-008 and R9-009 (280 LF) is indicated to be a 6" sewer "sandwiched" between the 8" diameter sewer on either side. If this sewer is confirmed to be a 6" sewer, it should be replaced to an 8" sewer.

The entire gravity sewer reach, from S11-002 to S10-001, encompassing the 6" sewer described below, is subject to Lift Station 3 cyclic flows matching the peak pumping rate of the lift station. This 220 gpm lift station flow was input into the model, and based on this, the entire reach from S11-002 to S10-001 (approximately 1,750 LF) should be upsized to 8" diameter.

A short 8" diameter pipe segment, on Quintana Road at Butte Avenue (S10-001 to S9-003), shows flows exceeding d/D of 0.75 at build-out maximum day flows (based on modeling A04 drainage as gravity flow). When considering the peak flow from Basin A03, I/I, and the peak pumping rate from Lift Station 3, this reach of sewer may reach d/D conditions greater than 0.9. The flow pulse from the lift station begins over 1,600 feet upstream of this reach of sewer, and to what degree this flow wave dissipates by the time it reaches Manhole S10-001 is unclear. It is known that the measured peak from the lift station, at Manhole R9-008 (flow monitoring station) downstream of this sewer reach by approximately 900 feet, was approximately 70 to 90 gpm (in comparison to the 220 gpm pump capacity). Based on this, the lift station peak flow influence likely is approximately half the pumping rate (100 gpm) at Manhole S10-001, and increases to 220 gpm at Manhole S11-002. This sewer reach on Quintana Road at Butte Avenue, to Morro Bay Boulevard, approaches a d/D of 0.9 at peak hydraulic flow in the future; thus, it is recommended to monitor and observe this pipeline in the future, and only replace or upsize this gravity main should monitoring data warrant such replacement in later years.

PREVIOUS COLLECTION SYSTEM MODEL

In the 1986 collection system model, insufficiently sized pipes were identified and correction measures proposed in the 1986 capital improvement program. For comparison purposes, these pipes are listed in Table 5-6 along with an indication of whether or not they were completed. Many of the projects were not completed. With actual wastewater flows decreasing since 1986, some of the recommended upgrades from the prior model may not be required. Also indicated in this table is whether the recommended upgrade is still required based on current modeling efforts.

CONCLUSIONS/RECOMMENDATIONS

Based on the model results, the majority of the City's collection system is adequate to serve build-out needs. However, one major consideration surrounds the Morro Bay/Cayucos joint sewer relative to its ability to meet the City's existing and future flow conditions.

Table 5-5 lists all existing and future deficient sewer pipes noted from the sewer modeling analysis, with no reduction of the infiltration/inflow component. This table shows the deficiencies relative to design capacity during average dry weather flow and peak hour wet weather flow, remaining capacity based on flows, and remaining capacity expressed in equivalent dwelling units (EDUs). In Chapter 3, an EDU was defined as the approximate wastewater flow generated from a single family dwelling unit. As can be seen by this summary table, there are specific reaches of sewer with no existing capacity remaining based on peak hour wet weather flow. The 18" trunk sewer on Atascadero Road also has no remaining capacity based on current dry weather flow conditions. As stated earlier, it is recommended that this 18" trunk sewer be upgraded as soon as practicable. In all other areas where there are no existing deficiencies based on ADWF, but there are deficiencies based on PHWWF conditions, it is recommended that any further development of units be considered under the following conditions:

- Each proposed development provides a minimum of 3 EDU equivalent flow reductions in I/I (e.g., one SFR provides I/I reduction of 900 gpm ADWF, or approximately 2,700 gpm PHWWF per household), or equivalent monetary contribution to the City to fund such reductions.
- With the exception of the existing 18" sewer in Atascadero (recommended for near-term upgrade), the sewer reach tributary to the proposed development should not receive ADWF beyond the ADWF sewer capacity at d/D=0.75.

Based on the above criteria, development "upstream" of the following sewer reaches should be carefully considered in one area:

- Basin A07, Main Street, north of Elena Street.

Recommendation for Morro Bay/Cayucos Sewer

Considerable detail was provided in this chapter relative to flow conditions to this joint sewer, with the various flow conditions, and with the two scenarios of discharge or no discharge by Cayucos SD.

Table 5-5 Summary of Capacity Remaining in Identified Deficient Pipes

Sewer Reach				Capacity at 0.75 d/D (mgd)	Capacity at 0.90d/D (mgd)	Existing Flows (mgd)			Remaining Capacity (mgd)		Remaining Capacity in EDUs[2]	
Upstream	GIS Numbering	Location	Diameter (in)			ADWF[1]	% ADWF Capacity Utilized[6]	PHWWF[1]	ADWF	PHWWF	ADWF only[3]	PHWWF[3]
A01a-AA02	M7-018	Main St/ Hwy 41	15	1.270	1.570	0.641	50%	2.179	0.629	0.000	2096	0
A01a-AA04	M7-005	Main St/ Hill St	15	1.125	1.380	0.640	57%	2.170	0.485	0.000	1616	0
A01a-AA05	L6-006	Main St/ Hill St	15	1.170	1.450	0.637	54%	2.162	0.533	0.000	1775	0
A01a-AA06	L6-004	Main St/ Hill St	15	1.430	1.760	0.636	44%	2.155	0.794	0.000	2648	0
A01a-AA07	L6-002	Main St/Avalon St	15	1.420	1.750	0.629	44%	2.140	0.791	0.000	2636	0
A01a-AA09	K6-012	Main St/ Bonita St	15	1.170	1.450	0.558	48%	1.775	0.612	0.000	2041	0
A01a-AA10	K6-008	Main St/ LasVegas/Bonita	15	1.230	1.520	0.548	45%	1.752	0.682	0.000	2273	0
A01a-AA11	K6-003	Main St/Las Vegas St	15	1.450	1.800	0.547	38%	1.747	0.903	0.053	3010	176
A01a-AA12	J6-012	Main St/Pico St	12	0.960	1.180	0.476	50%	1.278	0.484	0.000	1613	0
A01a-AA13	J6-008	Main St/Elena St/Pico	12	0.820	1.010	0.472	58%	1.268	0.348	0.000	1159	0
A02-AA02	N7-002	Main St/Errol St	12	0.840	1.001	0.236	28%	0.981	0.604	0.020	2014	67
A02-AA03	N7-004	Main St/Errol St/ Preston St	12	0.690	0.983	0.212	31%	0.907	0.478	0.076	1593	253
A02-AA08	O7-007	Frontage Rd/Dunbar	10	0.730	0.891	0.203	28%	0.869	0.527	0.023	1757	75
A02-AA09	P7-003	Frontage Rd/Dunbar	10	0.730	0.889	0.203	28%	0.867	0.527	0.023	1757	75
A02-AA10	P7-007	Quintana Rd/Dunbar	8	0.590	0.880	0.187	32%	0.812	0.403	0.068	1345	227
A02-AA14	Q8-007	Surf St/Front St	8	0.660	0.761	0.178	27%	0.777	0.482	0.000	1607	0
A02-AA22	R9-003	Quintana Rd/Kennedy Wy	8	0.455	0.560	0.141	31%	0.635	0.314	0.000	1048	0
A03-AA10	R9-010	Quintana Rd east of Kennedy Wy	8	0.460	0.565	0.141	31%	0.608	0.319	0.000	1064	0
A03-AA11	R9-009		8	0.420	0.515	0.141	34%	0.619	0.279	0.000	931	0
A05-AA01	G6-003	Main St/south Island St	12	0.700	0.870	0.395	56%	0.667	0.305	0.203	1018	677
A05-AA03	G5-009	Main St/ Island St	12	0.690	0.850	0.394	57%	0.648	0.296	0.202	988	674
A05-AA08	F5-014	Main St/ Mindford St	12	0.615	0.760	0.381	62%	0.569	0.234	0.191	779	637
A07-AA01	J6-006	Main St/ Elena St	12	0.720	0.895	0.471	65%	1.501	0.249	0.000	829	0
A07-AA02	I6-020	Main St/ San Joaquin	12	0.750	0.930	0.469	63%	1.429	0.281	0.000	936	0
A07-AA03	I6-014	Main St/ San Joaquin	12	0.750	0.925	0.468	62%	1.358	0.282	0.000	939	0
A07-AA04	I6-006	Main St/ San Joaquin	12	0.800	0.990	0.407	51%	1.107	0.393	0.000	1310	0
A07-AA05	H6-020	Main St/ San Jacinto	12	0.680	0.840	0.405	60%	1.036	0.275	0.000	915	0
A07-AA06	H6-013	Main St/ Sequoia	12	0.810	1.010	0.404	50%	0.964	0.406	0.046	1353	153
A07-AA09	G6-005	Main St/ south of Island St	12	0.730	0.900	0.395	54%	0.734	0.335	0.166	1118	553
TP-ATP02	M6-001	Atascadero St[5]	18	0.920	1.140	0.905	98%	3.024	0.015	0.000	51	0
TP-ATP03[4]	M6-003	Atascadero St	18	1.300	1.620	0.900	69%	3.015	0.400	0.000	1333	0
TP-ATP04	M7-020	Atascadero St	18	1.300	1.620	0.897	69%	3.009	0.403	0.000	1342	0

1. ADWF includes flow allocation from Cayucos CSD; PHWWF includes inflow and inflitration, and flow allocation from Cayucos CSD

2. EDU defined at 300 gpd per single family residence

3. Remaining capacity at given condition

4. Capacities are to match the downstream pipe; this stretch of pipe has a significantly steeper slope

5. Existing ADWF's are within 90% of recommended ADWF capacity at d/D=0.75

6. Based on existing ADWF and d/D=0.75; values over 100% denote over capacity.

It is first recommended that the City aggressively pursue sources of I/I in Basins A05, A06 and A07. This is discussed in Chapter 4 of this report. Reduction in I/I throughout these Basins may be able to eliminate the peak hour flow deficiencies noted in the model, for both existing and future conditions, under the scenario that the Cayucos SD does not use their allotted capacity in this joint sewer. If the source of I/I is significantly reduced, this joint sewer could serve the City's needs alone with no need for major capital expenditures in the future. A realistic expectation of I/I reduction may be around 20% to 50%, as stated in Chapter 4.

Should the Cayucos SD retain their right to capacity in this joint sewer, the 12" sewer reach from approximately Nevis/Nassau or Mindoro Street, to Las Vegas Street (approximately 1.1 miles) is deficient in future dry weather flow conditions, even if I/I in these basins is significantly reduced in the future. Based on this, it would then be recommended to replace/upsize this 12" sewer to 15" diameter. As noted earlier, the reach at Main Street near Elena Street is already flowing at or near design capacity under maximum day dry weather flow conditions.

The 180-unit development which will occur in the near future, will contribute flow south of Island Street at Main Street. The potential 75-150 unit development, if it occurs, will occur later in time than the 180-unit development south of Island Street. However, timing of when reaches of this 12" should be upgraded will depend not only on these two developments, but also timing of infill development in this area. Assuming that the I/I component is significantly reduced, timing of replacement of the 12" gravity sewer, between Mindoro Street and Las Vegas Street, should be considered as follows:

- For the segment between Island Street and Las Vegas Street, the 12" sewer upgrade to 15" diameter should occur prior to a total increase of 250 units tributary to the 12" sewer from Las Vegas Street north throughout Basin A07 and A05. This could include a combination of flows from the 180-unit development, 150-unit development and/or infill.
- For the segment between Mindoro Street and Island Street, upgrade of the sewer to 15" diameter would be triggered by 200 units of development from Island Street north throughout Basin A05.
- If no significant reduction in I/I is anticipated in the future, this entire sewer reach should be upsized in the near-term.

It is recommended that the City and Cayucos SD clarify their contractual understanding of specific hydraulic capacities before considering future expansion of this sewer.

Regardless of existing or future flow conditions, or if Cayucos SD discharges to this joint trunk or not, it is recommended to replace the remaining portion of 18" sewer, from Hwy 41/Hwy 1 to the WWTP. This upgrade should be accomplished as soon as practicable, and definitely within the next 5 years. Consideration may be given to combining this upgrade with the future anticipated upgrade of the wastewater treatment plant. Should this upgrade of the plant be extended beyond 5 years, the City should continue closely monitoring the capacity of this 18" trunk sewer to confirm remaining capacity.

Further Considerations. To better understand the extent of the future capacity short-fall in the jointly owned sewer, Table 5-6 was prepared to summarize the

Table 5-6 Hydraulic Capacity Required for Future Needs

Sewer Reach				Capacity at 0.75 d/D (mgd)	Capacity at 0.90 d/D (mgd)	Future Flows (mgd)				Future Hydraulic Capacity Required		
Upstream	GIS Numbering	Location (along Main St and nearest cross street)	Diameter (in)			ADWF[1]	% ADWF Capacity Utilized	Max Day[1] + PF	PHWWF[1]	ADWF (gpm)	Max Day+PF (gpm)	PHWWF (gpm)
TP-ATP01	M6-006	Atascadero St	27	6.700	8.300	1.032	15%	1.990	3.279	0	0	0
TP-ATP02	M6-001	Atascadero St	18	0.920	1.140	1.032	**112%**	1.990	3.279	78	743	1,485
TP-ATP03	M6-003	Atascadero St	18	1.300	1.620	1.028	79%	1.972	3.270	0	467	1,146
TP-ATP04	M7-020	Atascadero St	18	1.300	1.620	1.025	79%	1.961	3.264	0	459	1,142
A01a-AA02	M7-018	Main St/ Hwy 41	15	1.270	1.570	0.732	58%	1.161	2.359	0	0	548
A01a-AA03	M7-011	Hill St	15	2.000	2.480	0.732	37%	1.161	2.356	0	0	0
A01a-AA04	M7-005	Hill St	15	1.125	1.380	0.730	65%	1.158	2.351	0	23	674
A01a-AA05	L6-006	Hill St	15	1.170	1.450	0.728	62%	1.150	2.342	0	0	620
A01a-AA06	L6-004	Hill St	15	1.430	1.760	0.726	51%	1.145	2.336	0	0	400
A01a-AA07	L6-002	Avalon St	15	1.420	1.750	0.720	51%	1.130	2.320	0	0	396
A01a-AA08	K6-015	La Jolla St	15	1.720	2.118	0.652	38%	0.969	1.967	0	0	0
A01a-AA09	K6-012	Bonita St	15	1.170	1.450	0.647	55%	0.955	1.952	0	0	349
A01a-AA10	K6-008	between LasVegas St & Bonita St	15	1.230	1.520	0.637	52%	0.931	1.930	0	0	284
A01a-AA11	K6-003	Las Vegas St	15	1.450	1.800	0.636	44%	0.929	1.925	0	0	87
A01a-AA12	J6-012	Pico St	12	0.960	1.180	0.563	59%	0.756	1.453	0	0	190
A01a-AA13	J6-008	between Elena St & Pico St	12	0.820	1.010	0.560	68%	0.748	1.443	0	0	301
A07-AA01	J6-006	Elena St	12	0.720	0.895	0.559	78%	1.117	1.764	0	276	603
A07-AA02	I6-020	San Joaquin	12	0.750	0.930	0.557	74%	1.110	1.691	0	250	528
A07-AA03	I6-014	San Joaquin	12	0.750	0.925	0.556	74%	1.105	1.620	0	247	483
A07-AA04	I6-006	San Joaquin	12	0.800	0.990	0.493	62%	0.879	1.365	0	55	261
A07-AA05	H6-020	San Jacinto	12	0.680	0.840	0.492	72%	0.873	1.294	0	134	315
A07-AA06	H6-013	Sequoia St	12	0.810	1.010	0.490	61%	0.867	1.223	0	40	148
A07-AA07	H6-007	Sequoia St	12	1.000	1.240	0.488	49%	0.859	1.150	0	0	0
A07-AA08	H6-005	Sequoia St	12	1.400	1.730	0.481	34%	0.830	1.060	0	0	0
A07-AA09	G6-005	south of Island St	12	0.730	0.900	0.481	66%	0.830	0.993	0	70	64
A05-AA01	G6-003	south Island St	12	0.700	0.870	0.481	69%	0.830	0.925	0	91	38
A05-AA02	G5-012	Island St	12	0.830	1.020	0.481	58%	0.830	0.917	0	0	0
A05-AA03	G5-009	Island St	12	0.690	0.850	0.455	66%	0.739	0.833	0	34	M
A05-AA04	G5-002	Jamaica St	12	0.830	1.020	0.452	54%	0.721	0.813	0	0	0
A05-AA05	F5-025	Java St	12	0.830	1.020	0.449	54%	0.706	0.797	0	0	0
A05-AA06	F5-022	Kodiak St	12	0.810	1.000	0.445	55%	0.685	0.776	0	0	0
A05-AA07	F5-017	Luzon St	12	0.740	0.910	0.442	60%	0.665	0.759	0	0	0
A05-AA08	F5-014	Mindoro St	12	0.615	0.760	0.438	71%	0.645	0.739	0	20	M[2]
A05-AA09	F5-009	Nassau St	12	0.690	0.860	0.433	63%	0.622	0.718	0	0	0
A05-AA10	F5-005	Oahu St	12	0.940	1.160	0.428	46%	0.593	0.693	0	0	0
A05-AA11	F5-001	Orcas St	12	1.150	1.430	0.423	37%	0.571	0.670	0	0	0
A05-AA12	E5-026	Panay St	12	0.850	1.050	0.419	49%	0.549	0.651	0	0	0
A05-AA13	E5-023	Rennell St	12	1.900	2.340	0.415	22%	0.529	0.631	0	0	0
A05-AA14	E5-021	Sicily St	12	2.000	2.480	0.412	21%	0.506	0.613	0	0	0
A05-AA15	E5-016	Trinidad St	12	2.050	2.550	0.409	20%	0.489	0.595	0	0	0
A05-AA16	E5-013	Tahiti St	12	2.040	2.510	0.404	20%	0.466	0.571	0	0	0
A05-AA17	E5-010	Vashon St - point of connection	12	0.960	1.190	0.399	42%	0.449	0.549	0	0	0

1. ADWFand Max Day includes flow allocation from Cayucos CSD; PHWWF includes inflow and inflitration, and flow allocation from Cayucos CSD

M = Marginal; pipe very near capacity

2. Identified overflow area during wet weather events.

hydraulic carrying capacity required by the City (taking into account the Cayucos SD 40% capacity) to fulfill future demands. Included in this table is a summary of hydraulic carrying capacity of the joint sewer, future flows anticipated in the joint sewer, and the additional hydraulic capacity needed to fulfill this future need. This table also assumes no reduction in the I/I flow component.

Other Collection System Recommendations

Table 5-7 provides a summary of the deficient pipes identified in the 1986 master plan, and an indication if the upgrade occurred, and if not, if it is still recommended as part of this plan. Table 5-8 presents a summary of the deficient and marginally deficient pipes in the system that are recommended for upgrade or close monitoring. These recommendations are based on the infiltration/inflow component remaining the same as currently measured during the 2004/2005 rainy season. As stated in Chapter 4, it is recommended that the City pursue the sources of I/I in the collection system, and reduce the overall wet weather flow components for overall benefit to the collection system and the wastewater treatment plant. An overall reduction in the I/I component of 40% to 50% would be a reasonable goal to achieve.

Table 5-7. Pipes Identified in the 1986 Master Plan for Upgrade

Basin	Location	Upstream/ Downstream Manhole		1986 Plan Previous Diameter (in)	1986 Recommended Construction (in)	Current Existing Diameter (in) (Atlas Map)	Upgrades Completed	Future Upgrades Still Required
		Old System	GIS/Atlas					
A01a	Atascadero Road, Hwy 41 to w/o High School	107-2-09/107-1-01	M7-022/M6-006	18	30 Replace	27	Sliplined 12/00	Completed
A01b	Atascadero Road, East of Main Street	206-3-16/107-2-04	M7-016/M7-019	6	8 Replace	6	No	No. Limited tributary area
A01a	Main Street, Pico St. to Las Vegas St.	105-4-06/105-4-08	J6-012/K6-003	12	15 Parallel	12 (no parallel line)	No	See Note 1
A01a	Main Street, Las Vegas St. to Hwy 41	105-4-08/107-2-09	K6-003/M7-022	15	15 Parallel	15 (no parallel line)	No	See Note 1
A07	Main Street, Pico St. to Sequoia St.	104-4-30/105-4-06	H6-007J6-012	12	8" to 15" Parallel	12 (no parallel line)	No	See Note 1
TP	Atascadero Road/WWTP	107-1-01/107-1-03	M6-008/M6-007	18	30 Replace	27	27" PVC 12/98	Completed
A01	Main Street, Errol St. to Hwy 41	107-2-12/107-2-09	N7-002/M7-022	12	10 Parallel	12 (no parallel line)	No	Yes[2]
A02	Main Street n/o Dunbar St.	208-1-35/208-1-21	P7-003/P7-007	8	10 Replace	8	No	Yes[2]
A02	Main Street s/o Errol St.	208-1-21/107-4-07	P7-007/O7-005	10	8 Parallel	10 (no parallel line)	No	Yes[2]
A02	Main Street, Errol St.	107-4-07/107-2-12	O7-005/N7-002	12	8 Parallel	12 (no parallel line)	No	Yes[2]
A03	Quintana Road, Kings St. to Butte St.	309-2-03/309-1-08	S10-006/S10-001	6	8 Replace	6	No	Yes[3]
A03	Quintana Road at Morro Bay Blvd.	309-1-08/309-1-05	S10-001/S9-003	8	10 Replace	8	No	No[4]
B01	Market Avenue, Pacific St. to Morro Bay Blvd.	209-3-23/209-1-30	S7-009/S7-001	10	8 Parallel	10 (no parallel line)	Replaced	Completed

Basin	Location	Upstream/ Downstream Manhole		1986 Plan Previous Diameter (in)	1986 Recommended Construction (in)	Current Existing Diameter (in) (Atlas Map)	Upgrades Completed	Future Upgrades Still Required
		Old System	GIS/Atlas					
B01	Front St., Morro Bay Blvd to Beach St.	209-1-30/209-1-26	S7-001/R7-001	12	8 Parallel	12 no parallel line	No	No
B01	Embarcadero, South St. to Pacific St.	210-1-08/209-3-03	T7-003/S7-006	10	15 Replace	10	No	No
B01	Embarcadero, Morro St. to BeachSt.	209-3-03/209-1-03	S7-006/R7-007	15	10 Parallel	15 no parallel line	No	No
B01	Embarcadero, Beach St. to LS 2	209-1-03/108-4-04	R7-007/LS 2	15	12 Parallel	15 no parallel line	No	No
B03	Embarcadero, w/o Acacia (Marina Pkg. Lot) to Embarcadero w/o Olive	210-3-10/210-1-17	V8-007/U7-002	10	8 Parallel	10 no parallel line	No	No
B03	Embarcadero, W/o Olive to South St.	210-1-17/210-1-08	U7-002/T7-003	10	12 Parallel	10 no parallel line	No	No
TP	Embarcadero, Coleman Dr. to WWTP	108-1-01/107-1-03	P5-001/N6-001	18	21 Replace	18	No	No

[1]See discussion on Morro Bay/Cayucos Joint Sewer in this chapter.
[2]Upgrade with parallel sewer, or upsize sewer main one pipe size.
[3]Upsize recommended due to Lift Station 3 influence, from S11-002 to S10-001.
[4]Monitor this sewer, and replace in future only if warranted by future growth and confirmation of peaking factors.

Table 5-8. Summary of Gravity Sewer Recommendations

Basin	Upstream Manhole	Downstream Manhole	Current Diameter	Recommended Upgrade	Deficiency Timing	
					Exist.	Future
A02	N7-002	M7-022	12	15[a]		X
	N7-004	N7-002	12	15[a]	X	
	N7-007	N7-004	12			Monitor[b]
	O7-002	N7-007	12			Monitor[b]
	O7-005	O7-003	12			Monitor[b]
	P7-003	O7-007	10	12[a]		X
	P7-007	P7-003	8	12[a]	X	
	P7-009	P7-008	8			Monitor[b]
	Q8-002	P7-009	8			Monitor[b]
	Q8-007	Q8-002	8			Monitor[b]
	R9-003	R9-002	8			Monitor[b]
A03	S11-002	S10-001	6	8	X	
	S10-001	S9-003	8		Monitor[b]	
	S9-001	R9-010	8			Monitor[b]
	R9-010	R9-009	8		Monitor[b]	
	R9-009	R9-008	8		Monitor[b]	

[a]upsize to indicated diameter, or construct parallel 8" sewer if utility spacing allows.
[b]Monitor closely, and upgrade only if warranted. Reduction in I/I may defer need for pipeline upgrade.

CHAPTER 6

EVALUATION OF SEWAGE LIFT STATIONS

The City of Morro Bay owns and operates three sewage lift stations as part of the City's overall sewage collection system. These lift stations and corresponding service areas are depicted on Figure 6-1, and are briefly summarized in Table 6-1. Details of the hydraulic capacity, equipment and other details of the lift stations will be provided later in this Chapter.

Table 6-1. Summary of Lift Station Locations and Areas Served

Lift Station	Location	Tributary Served
1	Cloisters (2601 Coral)	Basin A06
2	Embarcadero	Portion of TP, B01, B02, B03
3	Quintana Rd./South Bay Blvd	A04

LIFT STATION EVALUATION

This section provides a detailed evaluation of each of the three lift stations. The lift stations are evaluated from a general operational standpoint, and then from a hydraulic/operations standpoint.

Lift Station General Evaluation (non-hydraulic)

The three lift stations were evaluated based on non-hydraulic parameters. This evaluation included review of existing information, as-built drawings, and a site visit to each lift station with City staff on June 15, 2005. A summary of the pertinent non-hydraulic parameters of the lift stations is presented in Table 6-2.

Lift Station 1. Lift Station 1 is located in "the Cloisters" development, at 2601 Coral Street. The lift station services all of Basin A06, which then pumps the wastewater flow to Basin A01a. The lift station discharges through a 6-inch diameter ductile and cast iron (cast iron under Highway 1) force main to a manhole at the intersection of Main Street and Las Vegas Street.

- Lift Station/Pumps. The lift station has a wetwell/submersible pump setup, with pumps on slide rails, and a drypit (valve vault). The lift station is relatively new, and was installed in 1997. The submersible pumps have been in operation since the wetwell was constructed, and no major refurbishments have been required. The City has one spare pump on the shelf for backup.
- Wetwell. The wetwell is a circular unlined concrete wetwell. The wetwell is in very good condition, with no visible signs of corrosion.
- Site Conditions. The lift station site area is paved, with a driveway/access off of Coral Street. The lift station is open (not fenced). Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. The site does not have any lighting for night-time emergency maintenance. There is potable water available for sanitation and washdown purposes.
- Valve Vault. The valve vault is equipped with an auxiliary connection to accommodate emergency pump connection.

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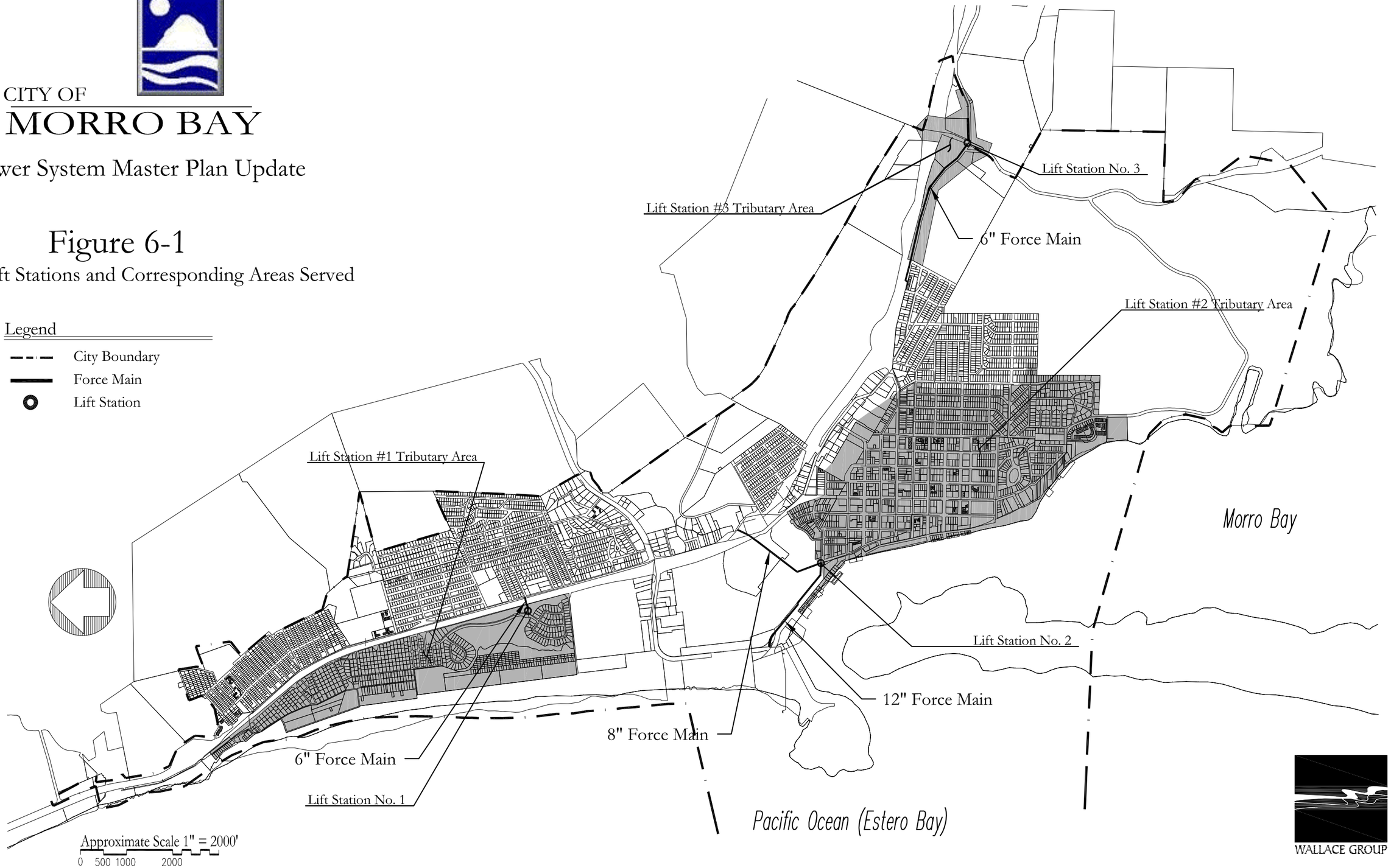
Sewer System Master Plan Update

Figure 6-1
Lift Stations and Corresponding Areas Served



WALLACE GROUP

Table 6-2. Summary of Lift Station Conditions (non-hydraulic)

Item	Lift Station 1	Lift Station 2	Lift Station 3
Year Built	1997	1954	1966
Lift Station Type	Wetwell w/Submersible Pumps	Wetwell w/Drypit and Vertical Non-Clog Pumps	Wetwell w/Drypit and Vertical Non-Clog Pumps
Standby Power	Receptacle for Portable	Receptacle for Portable	Receptacle for Portable
Electrical Service	New as of 1997	Replaced in 2004	Old and in need of replacement
Alarms	Telephone Alarm to Dispatch Who Calls Staff	Telephone Alarm to Dispatch Who Calls Staff	Telephone Alarm to Dispatch Who Calls Staff
Wetwell Material	Concrete	Concrete	Concrete
Wetwell Coating	No	No	No
Wetwell Condition	Very Good	Good	Fair to Poor (H_2S Pitting/corrosion)
Chemical Feed (Ferrous Chloride) Tanks/Piping	---	Conc. Containment re-coated, tank replaced in 2003	Replaced in 2003 (in vault)
Site Drainage	Good	Good	Poor (inundates during extreme weather)
Potable Water at Site	Yes	Yes	No
Site Lighting	None	None	None
Site Security/Fencing	None	None	None

- Site Power. The electrical service to the lift station is relatively new, and thus in good condition. The lift station is equipped with a receptacle to receive a portable generator, in the event of power outages. The electrical control panel is above-ground in an outdoor NEMA-rated enclosure.
- Telemetry/Alarms. Pump failures, high and low water conditions trigger alarm, via telephone alarm to Dispatch, who notifies City staff. Staff has indicated a desire to convert to an auto-dialer system that will dial multiple emergency numbers directly to staff for response to an alarm condition.
- Other. This is the only lift station of the three that does not have a ferrous chloride feed station located at the station.

Lift Station 2. Lift Station 2 is located on the "back side" (east) of a parking lot on the Embarcadero, near one of the City's maintenance yard. It is across the street from the Great American Fish Company and South Pier. The lift station services a portion of the TP drainage basin, and Basins B01, B02 and B03. The lift station discharges through a 12-inch diameter asbestos cement force main to a manhole on Embarcadero Road (west of the Duke Power Plant) and 18-inch diameter gravity sewer. There is a second "backup" force main, that is an 8-inch diameter cast iron force main, that discharges to a 12-inch diameter gravity sewer in Basin A-02, on Main Street/Frontage Road.

- Lift Station/Pumps. The lift station has a wet pit/dry pit set up, and was installed in 1954. Pumps have been rehabilitated and replaced (one in the dry pit, one on the shelf) in 2004. The City maintains one re-built pump on the shelf for backup. Removing and replacing pumps is difficult due to the depth of the dry pit, small opening, location of opening (not centered over pumps), confined space entry procedures. Piping provisions have been made to hook up a temporary emergency pump in the event of pump failure.
- Wetwell/Drypit. The wetwell is a circular unlined concrete wetwell. The wetwell is in good condition, with no visible signs of corrosion. One significant operational issue identified with this lift station, is that staff operates the wetwell level such that the influent gravity sewer is allowed to back up, and grease tends to build up in the gravity sewer, and/or clog the pumps occasionally. This lift station collects sewage from the majority of restaurants in the City. The City placed a screening device upstream to catch restaurant grease, and therefore also must clean it routinely to prevent grease buildup. If the City replaces the existing wetwell/lift station in the future, consideration should be given to installation of a self-cleaning wetwell system. According to City staff, the drypit lid is in poor condition and should be replaced. In addition, the access ladder to the drypit also is in need of replacement.
- Site Conditions. The lift station is in a paved parking lot area, and the lift station is open (not fenced). Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. However, runoff in this area goes directly to the harbor. The site does not have any lighting for night-time emergency maintenance. There is potable water available for sanitation and washdown purposes.
- Site Power. The electrical service to the lift station was replaced in 2004, and therefore is in good shape. The service was replaced from the transformer to the drywell. The lift station is equipped with a receptacle to receive a portable generator, in the event of power outages. Electrical controls/panels are in the drypit, underground.
- Telemetry/Alarms. Pump failures, high and low water conditions trigger alarm, via telephone alarm to Dispatch, who notifies City staff. Staff has indicated a desire to convert to an auto-dialer system that will dial multiple emergency numbers directly to staff for response to an alarm condition.
- Other. The chemical feed tank and piping were replaced, and the concrete containment area re-coated, within the last couple of years, approximately Year 2003. The chemical feed facilities are located in a separate secured fenced area, adjacent to the lift station and the City Yard.

Lift Station 3. Lift Station 3 is located on Quintana Road just west of the intersection with South Bay Boulevard. The lift station services drainage basin A04, which collects sewage from two mobile home parks, a convalescent hospital, mortuary, residentially zoned properties, and a Christian church. The lift station discharges through a 6-inch diameter asbestos cement force main up Quintana Road, where it discharges to Basin A03.

- Lift Station/Dry Pit. The lift station and dry pit is a Smith & Loveless package lift station. The station was installed in 1966. Pumps have been rehabilitated and replaced (one in the dry pit, one on the shelf) in 2005. Removing and replacing pumps is difficult due to the depth of the dry pit, small opening, location of opening (not centered over pumps), and required confined space entry procedures.
- Wetwell. The wetwell is a circular unlined concrete wetwell. The wetwell is in fair condition, but has some pitting and corrosion due to sulfuric acid attack (resulting from liberation of hydrogen sulfide gas).

- Site Conditions. The lift station is on the north shoulder of Quintana Road, and is open (not fenced). Hatches are padlocked for security. The site is within the defined 100-year flood plain. There is a drainage catch basin at the corner of Quintana Road and South Bay Boulevard, adjacent to the lift station, that conveys water directly to the nearby creek. Historically, the lift station has been inundated during floods. Of recent, the 1995 winter storms caused severe flooding in this area and inundated the lift station. However, the openings and hatches are sealed fairly well, so the lift station pumps and equipment were not damaged as a result of this flooding. The site does not have any lighting for night-time emergency maintenance.
- Site Power. The electrical service to the lift station is old, and should be scheduled for replacement. The lift station is equipped with a receptacle to receive a portable generator, in the event of power outages. Electrical controls/panels are in the drypit, underground.
- Telemetry/Alarms. Pump failures, high and low water conditions trigger alarm, via telephone alarm to Dispatch, who notifies City staff. Staff has indicated a desire to convert to an auto-dialer system that will dial multiple emergency numbers directly to staff for response to an alarm condition.
- Other. The chemical feed tank and piping were replaced in the last couple of years, approximately Year 2003. The chemical feed facilities are located in a concrete vault adjacent to the lift station.

Lift Station Hydraulic Performance Evaluation

The hydraulic characteristics of each lift station were analyzed and deficiencies were noted. Design criteria that apply to the lift stations and force mains are summarized below. Table 6-3 summarizes the hydraulic parameters of each lift station.

- Force main velocities should be greater that 2.0 feet per second to maintain self cleaning properties but less than 5.0 feet per second to minimize head loss and water hammer.

- Lift stations should be able to convey peak flows with the largest pump out of service. Station “capacity” is therefore calculated with the largest pump out of service.

- Lift station wet wells should be sized to limit the number of pump starts per hour to acceptable limits as defined by the pump manufacturer.

- Lift stations should have a means of conveying peak flows during a power outage. Lift stations serving a small number of customers could use wet well storage to meet this requirement.

Lift Station Flows. This subsection describes details of the existing lift stations and tributary flows (existing and future) relative to the pumping capacities of the existing lift stations. Flow parameters for each lift station are summarized in Table 6-4.

The peak hour wet weather flow is calculated as follows:

The average wastewater flow is multiplied by the diurnal peaking factor measured during the flow monitoring of 2004 and 2005, to obtain peak hour flow (dry weather). The actual diurnal peak measured for Lift Station 1 (Cloisters) is approximately 2.2, and therefore a peaking factor of 2.5 was chosen for the purposes of this study. Lift Station 2

receives flows from Basins B01, B02 and B03 primarily. The B01/B02 flow monitoring showed a peaking factor generally around 1.8 fairly consistently. These combined flows are approximately 2/3 of the total flow to Lift Station 2. Basin B03 had a notable peaking factor (including influence from a small private lift station) of around 3.5, but contributes only 1/3 of the total flow to Lift Station 2. Thus, a weighted average peaking factor was calculated at 2.75 for Lift Station 2. For Lift Station 3, which receives flow from Basin A04 only, a peaking factor was not measured. Thus, it was assumed that a reasonable peaking factor would be around 2.5 similar to Lift Station 1, since this basin receives only domestic, non-commercial flows. The peak hour wet weather flow is not calculated using the maximum day demand, as it is assumed that the PHWWF occurs during the winter, when wastewater flows are subject to normal (non-tourist season) flows.

Table 6-3. Summary of Hydraulic Characteristics

Item	Lift Station 1	Lift Station 2	Lift Station 3
Pump Type	Submersible	Vertical Non-Clog	Vertical Non-Clog
Pump Manufacturer/Model	Fairbanks Morse Model 5433MV	PACO Model 401221/22	Smith & Loveless, Model 4B3-4C3-4D3
No. of Pumps	2	2	2
Pump Motor HP	7.5	15	20
Motor Speed, rpm	1,200	1,150	1,760
Date of Last Pump Upgrade/Overhaul	(new since 1997)	2004	2005
Design Flow/Head (gpm@TDH)	380 at 42’	750 at 31.05’	220 at 150’
Pump Design Flow Condition[1]	Simplex	Simplex	Simplex
Approximate Pump Operating Efficiency at Design Point, %	65	63	50
Wetwell Operating Volume, Gallons	1,030[5]	1,128	329
Force Main Diameter, Inches	6	12/8[2]	6
Force Main Velocity, ft/s, Simplex (Duplex)	3.1 (5.5)[4]	2.0 (3.4)[3, 4] 4.7 (7.7)[4, 6]	2.5 (4.4)[4]

[1]Lift stations are designed to operate in the simplex mode, but operate in the duplex mode on an occasional basis when necessary during wet weather flow conditions.
[2]12” Diameter is the main force main. The 8” diameter force main is used as backup.
[3]Based on 12” diameter force main.
[4]Estimate based on assumed duplex flow conditions.
[5]Actual volume calculated, based on pump settings on as-built drawings, is 1,493 gallons. It is assumed that the City operates the pump on and off levels at a much smaller range, possibly smaller than even assumed to derive the 1,030 gallon capacity.
[6]Based on 8” backup force main.

Force Main Velocities. For all three lift stations, the force main velocities under simplex pump mode are within generally accepted criteria for self-cleansing and for

minimizing headloss. In the duplex mode, which occurs very infrequently, force main velocities are still generally acceptable. For Lift Station 1, the velocity of 5.5 fps under duplex operation is slightly high; however, as discussed, this mode of operation is infrequent. For Lift Station 2, for the backup 8" force main, in the duplex mode, velocities of 7.7 fps are expected and would be considered on the high range in regards to energy consumption and headloss.

Table 6-4. Summary of Lift Station Flows

Parameter	Lift Station 1	Lift Station 2	Lift Station 3
Average Annual Flow, gpd (gpm)(existing)[1]	67,592 (47)	250,359 (174)	33,914 (24)
Maximum Day Flow, gpd (existing)[1]	80,675 (56)	372,076 (258)	40,892 (28)
Peaking Factor (diurnal)[2]	2.5	2.75	2.5
Dry Weather Peak Flow, gpm (existing)	140	710	70
Wet Weather Infiltration/Inflow Component (gpm)[3]	375	105	145
Peak Hour Wet Weather Flow, gpm (existing)[5]	515	815	215
Average Annual Flow, gpd (gpm) (future)	68,952 (48)	280,882 (195)	39,931 (28)
Maximum Day Flow, gpd (gpm) (future)	82,307 (57)	417,734 (290)	48,112 (33)
Dry Weather Peak Flow, gpm (future)	143	798	83
Peak Hour Wet Weather Flow, gpd (gpm) (future)[5]	518	903	228
Lift Station Capacity, gpm, Simplex (Duplex)[4]	277 (550)	694 (1,000)	220 (400)

[1]AAF and MDF based on calculated flows from the hydraulic model.
[2]Diurnal peaking factor based on measured flows during 2004/2005 flow monitoring (excluding LS 3). See Chapter 4 for further discussion on peaking factors.
[3]Measured infiltration/inflow quantity based on 2004/2005 flow monitoring. This value is assumed to remain constant from existing to future flow conditions.
[4]Simplex capacity based on City records. Estimated duplex pumping capacity based on pump curves and head conditions.
[5]Based on ADF, peaking factor, plus wet weather flow. Assumes that wet weather flow condition will occur during winter time, and not during maximum day demand (summer tourist season).

- The actual measured wet weather inflow, from the 2004/2005 flow monitoring, is added to the dry weather peak hour flow, to obtain the peak hour wet weather flow. This wet weather inflow value is assumed to remain constant for existing and future flows. Through continued efforts by the City to minimize Infiltration/inflow, the wet weather flow component should not increase, and hopefully will decrease in future years.

- Lift Station 1. Lift Station 1 has a design capacity of 380 gpm@42'. City records indicate a hydraulic capacity of 277 gpm under simplex operation. In July 2005, the City conducted field pump tests to ascertain approximate pump flow rates. Based on this information, the pumps are currently pumping much higher than the design rating, in excess of 500 gpm each (in simplex) and over 1,000 gpm (in duplex operation). Based on the field results, the lift station hydraulic capacity is sufficient to pump existing and future peak hydraulic rates in the simplex mode. However, if the pumps were to pump at the design point and flow, the lift station would be marginal or deficient to meet existing and future peak hour wet weather flow conditions. Although it appears that the peak hour wet weather flow can be met by a single pump at this time, the peak hour wet weather flow component (inflow/infiltration) should be reduced in the coming years. When compared to the other lift stations on a flow/percentage basis, the I/I problem in Basin A06 and Lift Station 1 is severe and thus should be mitigated.

- Lift Station 2. Lift Station 2 has a design capacity of 750 gpm@31'. City records indicate a hydraulic capacity of 694 gpm under simplex operation. July 2005 field pump tests conducted by the City indicated a pump flow of only 350 to 400 gpm in simplex mode. It is suspected that this flow data is not representative of actual pump rates, as based on the City's records, this lift station has been able to handle all peaks, summer demand and winter wet weather peaks, in the simplex mode of operation. Although the lift station pumps appear to have sufficient capacity at this time, the lift station hydraulic capacity is likely marginal to deficient under simplex operation to meet existing and future peak hour wet weather flow conditions. Based on measured (2004/2005 flow monitoring) diurnal peaks, the peak dry weather hydraulic flows under existing conditions can be handled under simplex mode. In the future, the ability to handle peak dry weather flow in the simplex mode may be marginal. Should any future deficiencies be confirmed, the City should consider upgrades to pumping capacity to safeguard against overflow during wet weather events, and future peak day flows. It is noted that any sewage spills in this area would immediately discharge to the harbor. Given the nature of this lift station, if one pump fails during peak flow events, it will take too long for the City to install the standby pump. However, it is recognized that the City has the capability to provide a submersible pump on an emergency basis, and tie into the existing force main at this site. Increasing the pump motor and/or impeller will likely increase pumping capacity by 300 gpm or so, but still may not be enough to overcome the future peak flows and peak wet weather flow conditions. When compared to the other lift stations on a flow/percentage basis, the overall I/I problem in Basin B01, B02 and B03 is relatively minor; however, it is noted that there is definitely an inflow component, but very little to no infiltration.

- Lift Station 3. Lift Station 3 has a design capacity of 220 gpm@150' head. Based on existing and future maximum day flows, the lift station will be capable of pumping the peak hydraulic flow in Simplex mode of operation, and likely including the I/I component. The July 2005 field pump data suggests the lift station pumps are pumping at or near the design pumping rate. Although this lift station appears to be hydraulically sufficient for existing and future conditions, this should not preclude the City, however, from being aggressive at investigating and minimizing the infiltration component of flow detected (see next section on review of pump run times) in Basin A04.

Lift Station Wetwell Capacity. The lift station volumes were calculated, and pump cycle times were computed for each station, based on average day and maximum day flows (running in simplex mode). Table 6-5 summarizes the wetwell cycle time calculations.

Table 6-5. Summary of Lift Station Cycle Times

Item	Lift Station 1	Lift Station 2	Lift Station 3
Wetwell Operating Volume, gallons	1,030[1]	1,128	329
Cycles per Hour at Existing AAF	2.2	8.0	3.8
Cycles per Hour at Existing MDF	3.0	9.2	5.3
Cycles per Hour at Future AAF	2.2	8.4	3.9
Cycles per Hour at Future MDF	3.0	9.2	5.5

[1]See footnote 5 of Table 6-3.

Lift station pumps should typically cycle not more than 5 to 6 times per hour, to limit pump starts. This recommendation, however, should be based on the actual pump manufacturer's information. Pump motors and starters have improved significantly over the years, and thus can withstand more frequent starts than in years past. Lift Station 1 is the only modern station, built in 1997. Lift Station 2 and 3 are older stations, although pumps at both stations have been recently upgraded.

- Lift Station 1. As mentioned earlier, the operating volume of Lift Station 1 is relatively large, and based on the actual design operating levels specified on the as-built drawings, would be oversized for existing and future conditions. However, the pump levels can be easily field adjusted, and this provides operator flexibility with the lift station. The lift station cycles relatively infrequently; typically one would see a properly sized lift station cycle approximately 5 to 6 times an hour at average flow conditions. This lower number of cycles tends to allow raw sewage to sit longer in the wetwell, thus increasing the chances of off-gassing and sulfuric acid attack to the wetwell. Setting the operating levels to increase the number of cycles would enhance throughput of raw sewage through the wetwell and lift station.

- Lift Station 2. The wetwell operating volume for Lift Station 2 is somewhat undersized, based on the range (8 to 9.2) of cycles per hour. According to City staff, the wetwell level is operated to surcharge the influent gravity sewer as part of normal operations. This tends to complicate an already existing grease problem that occurs from the large number of area restaurants on the Embarcadero. If wetwell operating height cannot be increased, the only means of increasing wetwell capacity is to construct a larger wetwell adjacent to the existing one. Increasing the wetwell operating volume to approximately 1,800 gallons (increasing operating level by 2 feet, or increasing wetwell diameter) would decrease cycle times to approximately 5 to 6 cycles per hour. However,

cycle times must also be balanced with sewage idle time in the wetwell and potential for H_2S formation.

- Lift Station 3. The cycle times for Lift Station 3 are adequate for existing and future flows. If any adjustments to cycle times are desired, they can be accomplished by modifying the operating levels in the existing wetwell. According to the City, this lift station is also operated to surcharge the influent gravity sewer to minimize cycle times. As with Lift Station 2, cycle times must also be balanced with sewage idle time in the wetwell and potential for H_2S formation.

Review of Pump Run Times. Wallace Group reviewed the pump run times/log sheets provided by the City, from June 2004 to May 2005. The pump run times were reviewed to assess flow conditions and patterns throughout the year and at specific critical times, such as wet weather flow conditions, typical summer versus winter flows, and holiday trends. This review of pump run times revealed some interesting results and flow trends that were not evident in the gravity system flow monitoring conducted.

Select days were evaluated to determine flow characteristics, as follows:

- Dry Weather Flow Weekday (February 15, 2005)
- February 18, 2005 (heavy rain day)
- "Typical Summer Weekend" (June 12/13/14[1], 2005)
- July 4, 2004 Holiday Weekend
- Labor Day 2004 Holiday Weekend
- Harbor Festival 2004 Weekend

Table 6-6 summarizes the pump run times relative to these dates. Figure 6-2 shows a summary of pump run times throughout the year.

Observations. The following observations were made regarding wastewater flows relative to the pump run time data:

- Lift Station 1. In general, Lift Station 1 is not significantly impacted by holiday weekends. The determination of actual run times over the critical holiday weekends was difficult due to lack of readings on a daily basis. Lift Station 1 receives all residential flow, and thus is not influenced greatly by holidays. However, it does appear that the lift station receives larger flows based on the July 4th and Labor Day comparisons. It could be concluded that residents leave town during the Harbor Festival thus decreasing flows, or conversely, a number of residents are at the Harbor Festival, where wastewater flows are predominantly collected in portable toilets. Lift Station 1 has a significant wet weather component corresponding to large wet weather events, consistent with the flow monitoring results. The monthly run times, as shown by Figure 6-2, show a sharp increase in pump run times due to wet weather inflow in January/February 2005, with a slight extension of elevated flows through March 2005 suggesting some infiltration.

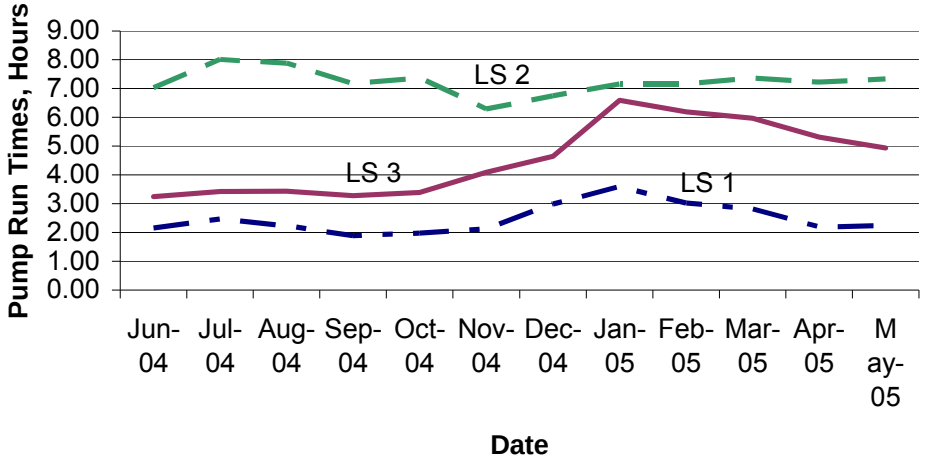
[1] Typical summer weekend was evaluated through Monday, June 14, 2005, due to the available times that staff reads the pump run times.

Table 6-6. Summary of Pump Run Times for Specific Dates

Date	Pump Run Times, Hours		
	Lift Station 1	Lift Station 2	Lift Station 3
Dry Weather Day (2/15/05)	2.1	8.5	6.8
Wet Weather Day (2/18/05)	6.9	19	11.5
"Typical Summer Weekend" (June 12 through June 14, 2004)	7.6	23	11
July 4, 2004 3-day Weekend[a]	10	29	7.2
Labor Day 2004 3-day Weekend[a]	8.5	27	11
Harbor Festival Weekend 2004	5.6	24	8.8

[a]Determined by taking the entire 4-day interval reading (readings not available for 4 days), and subtracting one "typical" dry weekday).

Figure 6-2. Lift Station Pump Run Times



- Lift Station 2. Lift Station 2 receives the majority of commercial flows, and thus corresponds heavily to the tourist season, as can be seen by the results. The

station sees up to a 25% increase in run time during holiday peaks, as compared to a "typical" summer weekend. As noted above, the Harbor Festival weekend flows are not as high as other holidays, since the majority of wastewater generated during the Festival is collected in portable toilets. It is evident that LS 2 had a significant increase in flows on the wet weather day, a sign that there is localized inflow. However, the overall pump run times show that this condition subsides quickly. Figure 6-2 shows that Basins B01, B02 and B03 are relatively "tight" with respect to infiltration. As noted earlier, there is some obvious inflow based on specific rain events and corresponding increased pump run times; however, it is noted that there is no apparent long-term increases in pump run times during the winter months, suggesting there is little to no infiltration occurring. It is also noted that the months of November/December see a noticeable drop, evidently due to a significant drop in the tourist season at this time.

- Lift Station 3. Lift Station 3 receives flows from the trailer parks, nursing home, Christian church, mortuary, and several proposed multi-family developments. This lift station appears to be relatively unaffected by holiday weekends and major events. The data is not consistent, and there does not seem to be a clear explanation for the pump run times for typical summer versus holidays. It is suspected there are anomalies in the data. The most significant finding is related to the monthly pump run times/trends over the year. From Figure 6-2, it is evident that the pump run times begin increasing around November/December 2004, in part due to some inflow from early rains, but also in part due to increasing infiltration. Beyond the peak wet weather events in January and February 2005, the increased pump run times sustain and gradually decrease, suggesting a fairly significant infiltration component reaching Lift Station 3. The May 2005 run times still appear to be higher than the June 2004 flows from a year prior, suggesting that the lift station may still be seeing infiltration effects from the 2005 rainy season. We are not aware of any significant developments from June 2004 to May 2005 that would explain an increase in lift station base/dry weather flows.

LIFT STATION CAPITAL IMPROVEMENTS

Recommended capital improvements with corresponding capital costs are presented in Chapter 7. A summary of the recommended capital improvements and their justification is included in this section.

General Recommendations

- It is recommended that the City strive to reduce infiltration/inflow to the extent practicable at all three lift stations, with Basin A06 being the priority, followed by Basin A04.

Lift Station 1 Recommendations

Priority 1 Capital Improvements:

- Provide fresh water washdown system for wetwell interior. If any deterioration is noted in the future, line wetwell for protection against sulfuric acid (from H2S) attack.
- Provide improved security features to minimize tampering potential.

Priority 2 Capital Improvements:

- Provide auto-dialer system to telephone/page designated City staff to respond to alarm conditions.
- Continue assessing I/I and pump capacity relative to peak hour flows.

Lift Station 2 Recommendations

Priority 1 Capital Improvements:

- Provide fresh water washdown system for wetwell interior. If any deterioration is noted in the future, line wetwell for protection against sulfuric acid (from H2S) attack.
- Provide improved security features to minimize tampering potential.

Priority 2 Capital Improvements:

- Upgrade lift station to submersible pump station with self-cleaning wetwell, with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs), and with sufficient hydraulic capacity/redundancy to meet future peak flows in the simplex mode of operation.
- Provide auto-dialer system to telephone/page designated City staff to respond to alarm conditions.

Lift Station 3 Recommendations

Priority 1 Improvements:

- Resurface wetwell interior, and line wetwell for protection against sulfuric acid (from H2S) attack. Provide fresh water washdown system.
- Upgrade/Replace electrical service.
- Provide improved security features to minimize tampering potential.

Priority 2 Capital Improvements:

- Upgrade lift station to submersible pump station with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs).
- In conjunction with lift station upgrade to submersible station, provide flood protection (flood wall or berm) to safeguard vaults from inundation.
- Provide auto-dialer system to telephone/page designated City staff to respond to alarm conditions.
- Provide potable water for sanitary and washdown purposes.

CHAPTER 7

CAPITAL IMPROVEMENT PROGRAM

This Chapter presents the proposed Capital Improvements Program, with a brief description of the proposed projects and a preliminary cost estimate for each proposed improvement. Also included in the CIP recommendations are general timelines and scheduling for the needed improvements, and general guidelines for cost allocations relative to existing and future developments.

BASIS OF CAPITAL IMPROVEMENT PROGRAM COSTS

The capital improvement program (CIP) costs were developed based on engineering judgment, confirmed bid prices for similar work in the Central Coast area, consultation with vendors and contractors, established budgetary unit prices for the work, and other reliable sources. Hard construction costs are escalated by a factor of 1.4, to allow budget for "soft costs" that include preliminary engineering, engineering, administration, construction management and inspection costs. All CIP costs are expressed in Year 2005 dollars, using an ENR Construction Cost Index of **7647**, and will need to be escalated to the year or years scheduled for the work. The unit cost for new gravity sewers includes the proposed pipelines, manholes, lateral re-connections, sewer bypassing, traffic control, etc., and all other aspects of sewer system construction.

TIMING OF RECOMMENDED IMPROVEMENTS

The timing of recommended improvements that are triggered by future development and growth is always difficult to ascertain. For the purposes of this report, the timing of future recommendations will be expressed in terms of equivalent dwelling units (EDUs) coming on line to the City's collection system and thus inducing the need for the specified improvements. For example, a project may be recommended to be in place when an additional 100 equivalent units are developed. Based on a household density of 2.02 and a per capita wastewater demand projected at 80 gpcd, an equivalent flow would be calculated at 16,160 gpd (11.2 gpm). Conversely, if historic trends for development indicate new housing development at 60 units per year, one might expect this development to occur with the next 18 to 24 months. Given this timing, the City would then need to anticipate this timing, and plan for the CIP accordingly, well enough in advance to anticipate the anticipated growth needs.

In addition to the timing of future needs, this CIP also addresses existing and "immediate" recommended CIPs. Recommendations for immediate improvements are made in this report, and are recommended to be completed within the next 3 to 5 years.

ALLOCATION OF CAPITAL COSTS TO EXISTING CUSTOMER BASE AND FUTURE DEVELOPMENT

It is recommended that the City conduct a rate study by a qualified firm to determine recommended updates to the City's rate structure, for sewer hook-up fees, development impact fees, and operation and maintenance costs. This section describes a general recommendation for how the City should allocate capital costs of improvements to both the existing customer and future development (impact fees).

Impact fees, or future increased connection fees, for future development are typically calculated based on the development's percentage increase in flow or impact to the infrastructure which will support the development. This potential impact could be based on a basin by basin review, or more globally based on City-wide services. It is recommended that any allocation of impact fees be based on the overall service area as a whole. It is felt this approach would be most equitable to all parties concerned. As an example, a developer could build a substantial development in one area of the City, within a collection system area that will require no future improvements to support this development. If impact fees are assessed based on basin-specific needs, this Developer would need only pay the ordinary sewer connection fee per unit. Conversely, one could build the same number of homes or fewer, in another area of the City, and could trigger collection system improvements that would be more costly, thus paying a larger incremental share of the improvement.

The existing population is approximately 10,510, leaving an additional 2,000 persons to reach the projected build-out population. This represents an increase of approximately 20 percent additional overall demand on the City sewer facilities. An outline of cost allocation would be as follows:

- New development pays the standard connection fee, plus impact fees for:
    - All costs for new improvements that are necessary due to development capacity increase only.
    - 20% of new improvements that are required to correct deficiencies not resulting from the Development, i.e., I/I mitigation, capacity issues with Cayucos SD, etc.

CAPITAL IMPROVEMENT PROGRAM

This section presents the capital improvement program recommendations for the City of Morro Bay sewer collection system. Immediate or near-term CIPs are presented first, followed by future CIPs and estimated timelines for implementation. As indicated earlier, all costs are expressed in current Year 2005 dollars and must be escalated appropriately, to estimated midpoint of construction in future years, for fiscal budgeting purposes.

Near-Term CIP Recommendations

The following are "immediate" or near-term recommendations for improvements to the City's sewer collection system:

<u>General Recommendations</u>

- Implement a program to identify sources of inflow and infiltration throughout the collection system, and implement measures to mitigate the I/I sources once identified. The priority for targeting I/I identification and source removal programs within the City's collection system are summarized as follows (1 being the most severe I/I problem):

1. Basin A07
2. Basin A06 Upper
3. Basin A01c
4. Basin A05
5. Basin A06 Lower
6. Basin A04
7. Basin B03
8. Basins B01 and B02
9. Basin A03
10. Basin A01b
11. Basin A02
12. Basin A01a
13. Basin TP

As mentioned in Chapter 3 and 4, it is also recommended that the overall I/I problem be addressed, and thus the Cayucos SD should consider also actively pursuing means of reducing and controlling I/I to the joint wastewater treatment plant.

Budgeting for I/I investigation and source reduction can be difficult. Until the source(s) of infiltration and inflow are identified, the magnitude of cost to reduce the sources of wet weather flow cannot be determined. However, the City should reserve an annual budget on an on-going basis to implement an I/I reduction program. With over 50 miles of gravity sewers throughout the City, video inspection alone could cost the City $130,000 or more. Recent video costs noted by Wallace Group for other local agencies were approximately $0.50/LF.

The City's I/I source identification program should consist of some or all of the following:

- Additional focused I/I wet weather monitoring in the "hot spots", to further pinpoint sources of I/I, Winter 2005/2006.

- Smoke test suspect areas of high inflow, including the northern portion of Basin A06, and Basin A01c, Winter/Spring 2006.

- Video inspect sewers that are of high potential for inflow and infiltration, including the northern portion of Basin A06 (particularly in the backyard sewers), and collection system components within Basin A04 (suspected infiltration), Winter 2005/2006.

- Continue maintenance program to maintain manhole gasket/water tight lids in areas prone to flooding, especially in Basins A05 and A07, and particularly on Main Street where manholes are situated in the flowline of the street. It is noted that the City has already installed such covers in over 50% of the existing manholes, predominantly in the flood-prone areas of North Morro Bay, and that any further I/I reduction from manhole water-tight covers alone is not anticipated.

Near-Term Collection System Trunk Line Upgrades

Table 7-1 presents a summary of specific gravity sewer pipe upgrades, recommended in the near-term. Refer to Figure 5-4 for locations of these specific pipe reaches. This table includes those sewers that are recommended for further observation and monitoring in the near-term. Figure 7-1 depicts the near-term collection system upgrades.

Table 7-1. Summary of Near-Term Gravity Sewer Recommendations

Basin	Upstream Manhole	Downstream Manhole	Current Diameter	Recommended Upgrade	Deficiency Timing	
					Exist.	Future
TP	M7-022	M6-006	18	27	X	
A05/ A07	F5-009	K6-003	12	15	X[c]	
A01	K6-003	M7-022	15	15[d]	X[c]	
A02	N7-004	N7-002	12	15[a]	X	
	P7-007	P7-003	8	12[a]	X	
	Q8-007	Q8-002	8		Monitor[b]	
	R9-003	R9-002	8		Monitor[b]	
A03	S11-002	S10-001	6	8	X	
	S10-001	S9-003	8		Monitor[b]	
	R9-010	R9-009	8		Monitor[b]	
	R9-009	R9-008	8		Monitor[b]	

[a]upsize to indicated diameter, or construct parallel 8" sewer if utility spacing allows.
[b]Monitor closely, and upgrade only if warranted. Reduction in I/I may defer need for pipeline upgrade.
[c]see discussion on Morro Bay/Cayucos SD Joint Sewer.
[d]Parallel relief sewer.

Morro Bay/Cayucos SD Joint Sewer. It is recommended that the City and Cayucos SD clarify their contractual understanding of specific hydraulic capacities before considering future expansion of this sewer. If the 40% capacity share of the CSD is not needed, the 12" reach of joint sewer does not likely need any future upgrades, so long as the I/I component can be reduced. Upgrade of the 12" diameter joint sewer, from Vashon Street to Las Vegas Street, however, should be anticipated in the future should the 40% capacity allocation remain intact. As indicated in Chapter 5, near-term efforts for this trunk sewer should focus on reduction of I/I as a first order of business. Regardless of this contractual capacity issue, the 15" sewer from Las Vegas Street to Atascadero Road would still require an upgrade (15" parallel relief sewer) in the future.

Aside from any capacity issues associated with the above discussion, the existing 18" gravity sewer, from Manhole M7-022 to M6-001 needs to be upgraded to a hydraulically equivalent 27" sewer (smooth-walled PVC, n=0.010), to hydraulically match the other portion of sewer recently upgraded to 27" diameter. From the model results, the short reach between M7-022 and M7-020 is not deficient; however, there are three sewers (15" joint sewer, 8" sewer from Hwy 41 east, 12" sewer from Main Street south) that all converge to M7-022. This reach is just over 1,000 LF. An option to place a parallel sewer to this 18" sewer was considered; however, the available slope is only 0.0007 ft/ft in this reach, which is very flat for even the existing 18" gravity sewer. Any parallel sewer 18" diameter or smaller would be very flat, and would not achieve good flow velocities in this area. Sewer bypassing during construction will be a major item to be addressed.

Near-Term Lift Station Improvements

Implement the following projects at all three lift stations:

- Provide improved security features to minimize tampering potential.

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Sewer System Master Plan Update

Figure 7-1

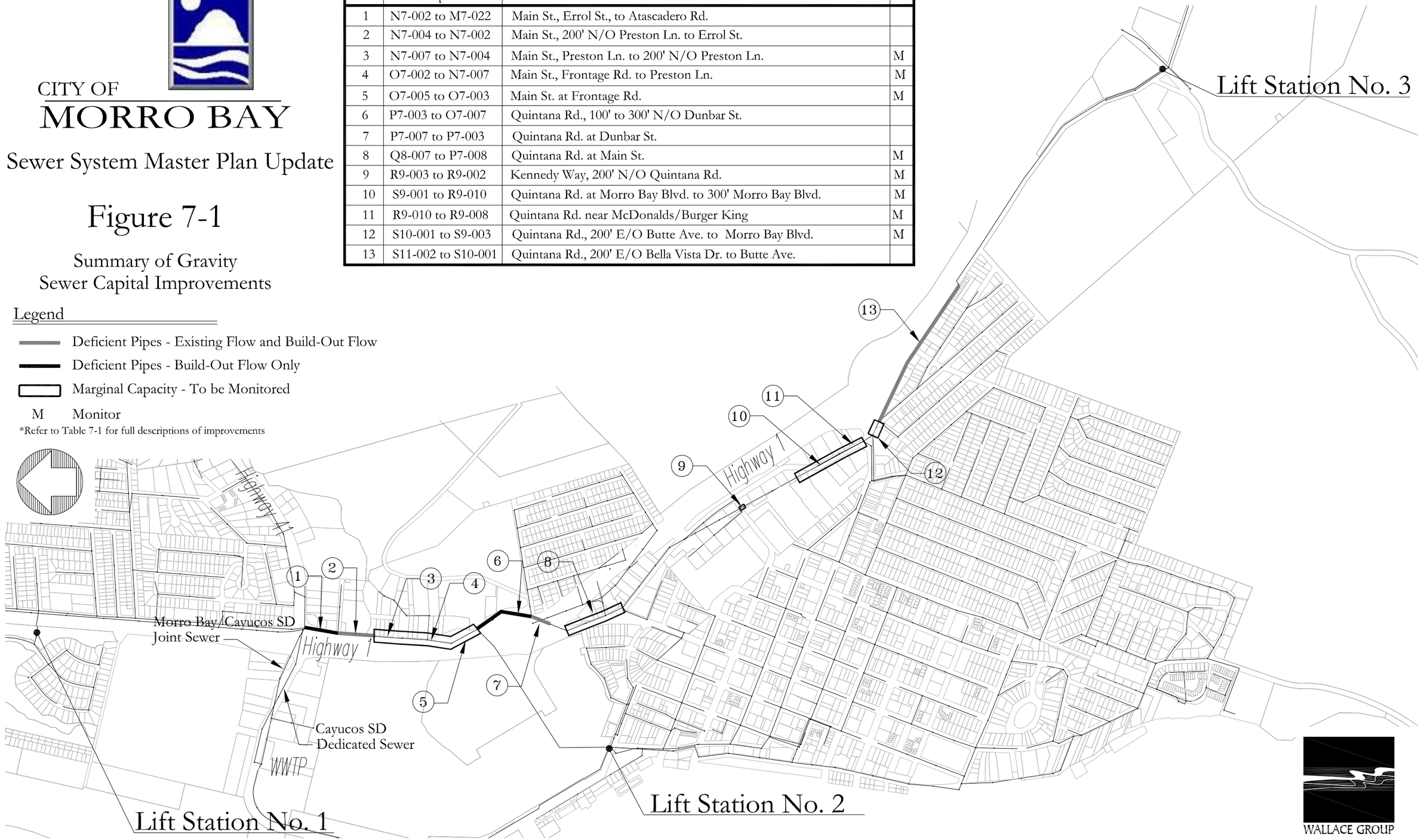
Summary of Gravity Sewer Capital Improvements

Label	Pipe	Location	
1	N7-002 to M7-022	Main St., Errol St., to Atascadero Rd.	
2	N7-004 to N7-002	Main St., 200' N/O Preston Ln. to Errol St.	
3	N7-007 to N7-004	Main St., Preston Ln. to 200' N/O Preston Ln.	M
4	O7-002 to N7-007	Main St., Frontage Rd. to Preston Ln.	M
5	O7-005 to O7-003	Main St. at Frontage Rd.	M
6	P7-003 to O7-007	Quintana Rd., 100' to 300' N/O Dunbar St.	
7	P7-007 to P7-003	Quintana Rd. at Dunbar St.	
8	Q8-007 to P7-008	Quintana Rd. at Main St.	M
9	R9-003 to R9-002	Kennedy Way, 200' N/O Quintana Rd.	M
10	S9-001 to R9-010	Quintana Rd. at Morro Bay Blvd. to 300' Morro Bay Blvd.	M
11	R9-010 to R9-008	Quintana Rd. near McDonalds/Burger King	M
12	S10-001 to S9-003	Quintana Rd., 200' E/O Butte Ave. to Morro Bay Blvd.	M
13	S11-002 to S10-001	Quintana Rd., 200' E/O Bella Vista Dr. to Butte Ave.	

Legend

- Deficient Pipes - Existing Flow and Build-Out Flow
- Deficient Pipes - Build-Out Flow Only
- Marginal Capacity - To be Monitored
- M Monitor

*Refer to Table 7-1 for full descriptions of improvements



WALLACE GROUP

Other specific lift station improvements for the near-term are summarized as follows:

Lift Station 1:

- Provide fresh water washdown system for wetwell interior. If any deterioration is noted in the future, line wetwell for protection against sulfuric acid (from H_2S) attack..

Lift Station 2:

- Replace drypit access lid and ladder.
- Provide fresh water washdown system for wetwell interior. If any deterioration is noted in the future, line wetwell for protection against sulfuric acid (from H_2S) attack.

Lift Station 3:

- Replace/upgrade existing electrical service.
- Resurface wetwell interior, and line wetwell for protection against sulfuric acid (from H_2S) attack. Provide fresh water washdown system.

Future CIP Recommendations

The following are future recommendations for improvements to the City's sewer collection system. Where appropriate, general time guidelines as to when a CIP should be implemented, are provided.

Future Collection System Improvements

Table 7-2 presents a summary of specific gravity sewer improvements, recommended in the future. Refer to Figure 5-4 for locations of these specific pipe reaches. This table includes those sewers that are recommended for further observation and monitoring in the future.

Table 7-2. Future Gravity Sewer Recommendations

Basin	Upstream Manhole	Downstream Manhole	Current Diameter	Recommended Upgrade	Deficiency Timing	
					Exist.	Future
A02	N7-002	M7-022	12	15[a]		X
	N7-007	N7-004	12			Monitor[b]
	O7-002	N7-007	12			Monitor[b]
	O7-005	O7-003	12			Monitor[b]
	P7-003	O7-007	10	12[a]		X
	P7-009	P7-008	8			Monitor[b]
	Q8-002	P7-009	8			Monitor[b]
	S9-001	R9-010	8			Monitor[b]

[a]upsize to indicated diameter, or construct parallel 8" sewer if utility spacing allows.
[b]Monitor closely, and upgrade only if warranted. Reduction in I/I may defer need for pipeline upgrade.

All sewer upgrades for Basin A02 are recommended after approximately 130 equivalent residential units of development occur. Thus, the City should begin planning/design for these upgrades approximately two years prior. Based on what is envisioned to occur in Basins A02, A03 and A04, the development of 130 equivalent residential units could take a number of years, or may not even be reached. Thus, further monitoring in future years is recommended to verify if the upgrade is necessary in the future or not.

Future Lift Station Improvements

The following are lift stations recommendations for the future. The identified future upgrades are more safety and operational oriented, and are not necessarily triggered by future growth. These recommendations for Lift Station 2 and 3, however, will include significant capital investment, and should be scheduled for in conjunction with other factors, including anticipation of when next major pump upgrades might be required, end of useful life of facilities, and funding opportunities from grants, low interest loans, or major development whose capital connection/impact fees can help fund such projects.

Implement the following projects at all three lift stations:

- Provide auto-dialer system to telephone/page designated City staff to respond to alarm conditions.

Other specific lift station improvements for the future are summarized as follows:

Lift Station 1:

- Continue assessing I/I in the future, and reduce I/I in Basin A06 to the extent practicable. Verify actual pumping capacity relative to existing and future anticipated peak flows.

Lift Station 2:

- Upgrade lift station to submersible pump station with self-cleaning wetwell, with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs), and with sufficient hydraulic capacity/redundancy to meet future peak flows in the simplex mode of operation (if confirmed that peak pumping capacity is inadequate).

Lift Station 3:

- Upgrade lift station to submersible pump station with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs).
- In conjunction with lift station upgrade to submersible station, provide flood protection (flood wall or berm) to safeguard vaults from inundation.
- Provide potable water for sanitary and washdown purposes.

STAFFING AND OPERATIONS RECOMMENDATIONS

The City currently operates and maintains the collection system with a staff of three full-time employees. This staffing level appears to be marginal for operating and maintaining the existing collection system, in order to keep up with routine sewer flushing and other

activities, and including allowances for vacation and sick leave, emergency repairs, on-call rotations and other matters. Line flushing alone, can be a full-time commitment for a staff of two. The recommended Capital Improvement Program for sewer line upgrades should not unduly impact future staffing needs, but staff and equipment budgets will need to be increased over time to account for inflation and rising costs of operations.

The recommended I/I program will take considerable time and effort, particularly in the next few years when significant work should be implemented for sewer videotaping, analysis and review, smoke testing, and other investigative activities associated with the identification of I/I sources. The City may opt to invest in equipment purchase, and perform sewer videotaping in-house. Contract firms that perform video services, however, are very cost competitive. Even if the sewer video work is contracted out, staff time will be required to review and analyze videos, conduct smoke testing, and perform certain work related to I/I reduction. The City's collection system consists of approximately 50 to 60 miles of gravity sewers; of this length of sewer, it is likely that less than half of this length needs to be videotaped on a priority basis. Based on the initial I/I studies described in Chapter 4, the City can prioritize the areas to be videotaped. For about 20 miles of sewer, for example, it is estimated the video work can be contracted out for about $60,000 to $80,000 ($0.50 to $0.75 per LF).

The City should consider at a minimum, one additional maintenance staff person (salary approximately $40,000/year plus benefits). After the initial I/I investigation work is completed over the next several years, the City can re-evaluate the need for this fourth staff person. In the CIP Table 7-3, it was recommended to budget approximately $75,000 per year for I/I work. This budget would be for actual expenditures for the initial I/I investigative work, including sewer videotaping, smoke testing, and other investigative activities. It would be recommended the City budget salary for this additional staff person, in addition to the $75,000 per year for I/I activities in the next several years.

SUMMARY OF CAPITAL IMPROVEMENT COSTS

A summary of capital costs is presented in Table 7-3. As indicated earlier, the capital costs include both "hard" (construction) and "soft" (planning, engineering, construction management, administration) costs. Construction costs are multiplied by a factor of 1.4 to budget for soft costs.

Table 7-3. Summary of Capital Improvement Projects

Basin	Project Description/Location	City GIS/Manhole Number		Quantity			Old Dia., Inches	New Diameter, Inches[f]	Capital Improvement Cost, $		
		Upstream	Downstream	Value	Units	Unit Cost, $			Construction	Engr/Admin	Total
NEAR-TERM CAPITAL PROJECTS											
	I/I Source Reduction Program[a]	---	---	1	LS	$ 75,000	---	---		$ 75,000	$ 75,000
A02	Main St., 200' n/o Preston Lane to Errol St.	N7-004	N7-002	410	LF	$ 200	12	15	$ 82,000	$ 32,800	$ 114,800
	Main St. at Hwy 1 Undercrossing	P7-007	P7-003	210	LF	$ 175	8	12	$ 36,750	$ 14,700	$ 51,450
A03	Quintana Rd., Bella Vista Dr. to Butte Ave.	S11-002	S10-001	1,750	LF	$ 150	6	8	$ 262,500	$ 105,000	$ 367,500
A06	Lift Station 1 Wetwell Washdown, Security Improvements	---	---	1	LS	$ 20,000			$ 20,000	$ 8,000	$ 28,000
TP	Lift Station 2 Wetwell Washdown, Security Improvements, Drypit Hatch and Ladder Replacement	---	---	1	LS	$ 35,000			$ 35,000	$ 14,000	$ 49,000
A04	Lift Station 3 Wetwell Washdown, Security Improvements	---	---	1	LS	$ 20,000			$ 20,000	$ 8,000	$ 28,000
A04	Lift Station 3 Electrical Service Replacement	---	---	1	LS	$ 7,500			$ 7,500	$ 3,000	$ 10,500
A05/A07	Morro Bay/Cayucos Joint Sewer Upgrade[d]	F5-009	G5-009	1,080	LF	$ 200	12	15	$ 216,000	$ 86,400	$ 302,400
	Morro Bay/Cayucos Joint Sewer Upgrade[d]	G5-009	K6-003	4,940	LF	$ 200	12	15	$ 988,000	$ 395,200	$ 1,383,200
A01	Morro Bay/Cayucos Joint Sewer Upgrade[e]	K6-003	M7-022	2,930	LF	$ 225		15	$ 659,250	$ 263,700	$ 922,950
TP	Atascadero Rd., Hwy 41 towards WWTP	M7-022	M6-006	1,010	LF	$ 375	18	27	$ 378,750	$ 151,500	$ 530,250
Subtotal: Near-term Capital Projects									$ 2,705,750	$ 1,157,300	$ 3,863,050
FUTURE CAPITAL PROJECTS											
	I/I Source Reduction Program	---	---	1	LS	$ -	---	---	$ -	$ -	TBD
A02	Main St., Errol St. to Hwy 41	N7-002	M7-022	370	LF	$ 200	12	15	$ 74,000	$ 29,600	$ 103,600
	Main St., n/o Dunbar St.	P7-003	O7-007	340	LF	$ 175	10	12	$ 59,500	$ 23,800	$ 83,300
A06	Lift Station 1 Telemetry Upgrade	---	---	1	LS	$ 5,000	---	---	$ 5,000	$ 2,000	$ 7,000
TP	Lift Station 2 Upgrade to Submersible PS[b]	---	---	1	LS	$ 300,000	---	---	$ 300,000	$ 120,000	$ 420,000
A04	Lift Station 3 Upgrade to Submersible PS[c]	---	---	1	LS	$ 250,000	---	---	$ 250,000	$ 100,000	$ 350,000
Subtotal: Future Capital Projects									$ 688,500	$ 275,400	$ 963,900

Notes:

[a]Annual budget (recurring cost) of $25,000 per year for 3 years, for I/I source reduction.

[b]Includes telemetry upgrade.

[c]Includes telemetry upgrade, water service, and site work to reduce flooding potential.

[d]Segment G5-009/K6-003, Island Street to Las Vegas Street, will need to be implemented prior to F5-009/G5-009, Mindoro Street to Island Street. Future pipe diameter is based on additional capacity needed per Table 5-6.

[e]Parallel 15" relief sewer.

[f]All sewer replacements/upgrades are modeled as PVC pipe (n=0.010), unless otherwise indicated.

TBD=To be determined. Capital cost to be determined after source of I/I identified.