CITY OF SAN MARCOS
WASTEWATER MASTER PLAN
HAYS COUNTY, TEXAS

Document No. 050103
PBS&J Job No. 441201

CITY OF SAN MARCOS
WASTEWATER MASTER PLAN
HAYS COUNTY, TEXAS

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October 2005

Printed on recycled paper

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Acronyms and Abbreviations

ADF	average daily flow
ADWF	average dry weather flow
CAMPO	Capital Area Metropolitan Planning Organization
CIP	Capital Improvement Plan
CS	Collection Systems
GIS	Geographic Information System
gpd	gallons per day
gpm	gallons per minute
GWI	groundwater infiltration
HDPE	high-density polyethylene
HGL	hydraulic grade line
I/I	infiltration/inflow
IH 35	Interstate Highway 35
LS	lift station
MGD	million gallons per day
PWWF	peak wet weather flow
SSES	Sanitary Sewer Evaluation Survey
SSOs	sanitary sewer overflows
TCEQ	Texas Commission on Environmental Quality
TSZs	traffic serial zones
USGS	U.S. Geological Survey
WWG	Wastewater Generator
WWTP	Wastewater Treatment Plant

Executive Summary

The City of San Marcos (City) employed PBS&J to update the City's 25-year wastewater master plan through a detailed study that included:

- a flow monitoring program to record and analyze dry and wet weather flows at key locations in the collection system,
- development of a dynamic computer model of the collection system with current and projected future flows,
- use of the computer model for development of improvements projects required to correct capacity deficiencies and serve future growth areas,
- analysis of improvements projects required to intercept and convey a portion of the City's future flows to a proposed second treatment plant, to be located in the Cottonwood Creek watershed, versus improvements required to transport all future flows to the existing plant,
- preparation of construction cost estimates for the various alternatives, and
- assessment of applicable projects for impact fee calculation.

The initial phase of the study focused on collecting data on the existing wastewater system needed for generation of the computer model. A flow monitoring study was completed using 12 temporary flow meters and 4 rain gauges to quantify existing dry and wet weather flows at strategic manholes in the collection system. Influent flows at seven key lift stations were determined using SCADA data supplied by the City and pump testing results. Data on the piping network was furnished by the City from a number of sources: the City's geographic information system (GIS) files with the location of pipes, manholes, lift stations, and force mains; the previous computer model developed by Black & Veatch in 1994; digital and hardcopy construction plans of recent and upcoming projects that were not in the GIS; flowline elevations surveyed by the City for manholes with no other source of pipe elevation data; and the City's set of collection system grid maps. The City also furnished a copy of the previous Capital Improvements Plan updated by the City showing wastewater projects already in the planning stage.

Data on population and employment, which was used to generate wastewater flows in the model, was obtained from projections developed by the Capital Area Metropolitan Planning Organization (CAMPO) for the Hays-Travis-Williamson County area. CAMPO developed population and employment numbers by traffic serial zone for 2003, 2007, 2017, and 2027. Projections for some San Marcos traffic serial zones were revised by the City for local conditions. The same four years were used for modeling the San Marcos collection system for deficiencies and improvements, along with year 2014 for impact fee purposes.

Wallingford Software's state-of-the-art InfoWorks CS 5.5 software was chosen for dynamic hydraulic modeling of the wastewater system. The physical data gathered in the initial phase of the study was utilized to establish such existing system attributes as connectivity, pipe sizes and elevations, and lift station pumping rates and wet well operating volumes. The model generally included pipes 10-inch diameter and larger, with 596 pipes and 10 lift stations in the model of the existing system. Additional pipes were added for projects in the planning stage in order to develop 2007, 2014, 2017, and 2027 models for future conditions. Elevations and slopes for conceptual pipes were based on topographic maps for ground elevations with minimum slopes per Texas Commission on Environmental Quality standards.

Population and employment were added to the model to generate wastewater flows. The 19-meter basins in the flow monitoring program (flow meters and lift stations) were subdivided into 113 subbasins (or "subcatchments") for the existing system, each with a designated manhole point-of-entry. Modeling routines used intersection of the subcatchments and CAMPO's traffic serial zones to distribute population and employment to the subcatchments. The model calibration process used diurnal curves generated from flow monitoring data to develop unit flow rates for each meter basin as gallon/day per capita and gallon/day per employee. Each meter site's diurnal pattern (time-step flow relative to daily average) was input to the model, and the unit flow rates were adjusted so that simulated flows from the model matched the actual diurnal flow curve for each meter site. The per capita unit flows were typically in the 60 to 90 gallon/day range, and the per employee unit flows ranged from 10 to 360 gallon/day. The model of the existing system was also calibrated for the inflow characteristics of wet weather flow. A similar calibration process was used to establish leakage (volume) and routing (shape) parameters for each meter basin in order to match simulated flows with actual metered flows for a storm event that occurred on June 5, 2003. Each subcatchment was assigned the wastewater and inflow parameters derived for its meter basin.

For the 2007, 2017, and 2027 models, the aforementioned subcatchment/traffic serial zones polygon intersection routine was used to increase the future population and employment in the subcatchments of the existing system, and new subcatchments were added along with future mains in the planning stage to serve projected growth in outlying areas. Acreage for future subcatchments was based on the projected CAMPO population and employment at typical densities seen in current development projects. A peak inflow/infiltration rate of 750 gallon/day/acre was used for future subcatchments due to the obvious lack of actual flow monitoring data.

After the model of the existing (2003) system was calibrated to the June 5, 2003 storm, the model was analyzed with 2-year and 5-year design storms with 3-hour and 6-hour durations. The extent of overflows was not significantly sensitive to the design storm, so a 5-year 6-hour design storm was selected for analysis of the models for deficiencies and improvements. At that point new pipes were added to the model to reflect planned projects in the City's previous Capital Improvements Plan (CIP) planned for fiscal years 2003–2014. Ten projects were added for the 2007 model, and 14 projects were added for the 2017 and 2027 models. These models were run with the design storm and projected future population and

employment to identify overflow locations and timing. The model with 2027 flows was analyzed to identify improvements needed to eliminate overflows. As a result, some of the City's planned CIP projects were modified and other new projects were added to the CIP. In addition, portions of the collection system that appear to have the most severe inflow/infiltration problems were identified based on data from the flow monitoring program, and these areas were listed as highest priorities for the sanitary sewer evaluation survey and rehabilitation projects included in the proposed CIP. The results of this capacity and improvements analysis are presented in Section 5 of the report. The total estimated cost in the proposed CIP for these projects is $22.5 million. These projects are necessary independent of the alternative selected for future treatment facilities.

The City's existing River Road wastewater treatment plant is permitted for 9.0 million gallons per day (MGD) average flow (maximum month) and 18.0-MGD 2-hour peak flow. The current actual process capacity is 7.6 MGD. The projected 2027 flows for the City's wastewater system based on modeling are 14.9-MGD average flow, with 56.4-MGD peak flow for a 5-year 6-hour design storm. The study evaluated two alternatives for future treatment facilities—expansion of the existing River Road plant for all the future flow (Alternative 1) and construction of a second plant in the Cottonwood Creek watershed (Alternative 2) with sufficient diversion of flow to the new plant to limit future flows to the existing plant to its current treatment capacity. In Alternative 1 the existing treatment facility would be expanded to treat the projected future flows (14.9-MGD-average dry weather/56.4-MGD-peak wet weather for 2027). New interceptors, lift stations, and force mains to serve future development east of Interstate Highway 35 would require different routing to deliver flow to the existing plant versus to the Cottonwood Creek plant. In Alternative 2 the Cottonwood Creek ("Southeast") treatment plant would have to receive 7.4-MGD average flow (2027) in order to limit the existing plant to flows not exceeding its current capacity of 7.6 MGD. Flows must be diverted from both the north and south portions of the service area to deliver 7.4 MGD to the Southeast plant. System improvements necessary for these diversion options were thoroughly evaluated for each alternative. An Alternative 2A diverts the north flow from the Blanco River lift station to the future ByPass Creek interceptor, compared to Alternative 2 in which the Blanco River lift station continues to pump to the newly constructed Interstate Highway 35/Fairlawn Interceptor so that the north flow is diverted at the River Road lift station. The Alternative 2 variations sending 7.4-MGD average flow to the Southeast plant would have corresponding 19.5-MGD peak wet weather flow. Consequently, the existing plant would receive 36.8-MGD peak wet weather flow and require expansion from its current 18-MGD peak flow capacity (this expansion for peak flow would not be allowed to increase its 7.6-MGD treatment capacity). For both Alternatives 1 and 2 the effluent outfall location for the existing plant would be moved downstream of the confluence of the San Marcos River with the Blanco River due to endangered species and permitting considerations. There are some projects in the City's 2003–2014 CIP planned for serving future development in the southeast area whose capacity and sizing requirements are greatly affected by the alternatives for the location of future treatment plant construction. Section 6 of the report includes identification and cost comparison of projects associated with the future treatment alternatives. Estimated total CIP costs are $13.3 million for Alternative 1,

$15.9 million for Alternative 2, and $15.7 million for Alternative 2A (not including cost for future additional treatment capacity, which is assumed to be about the same regardless of location.) These CIP costs are in addition to the $22.5 million for the Section 5 improvements projects needed to address capacity deficiencies regardless of the future treatment alternatives.

As mentioned previously, the 2014 future collection system was evaluated to develop a basis for an impact fee update. The existing (2003) and future (2014) capacity utilizations were evaluated for the CIP projects that would be eligible for including in the charge of an impact fee. Impact fees can only be applied to new customers of the system, so only newly constructed and future projects (where new customers would be established) were considered for the calculation of impact fee. Percentages of capacity utilization were determined and are presented in Section 7 of the report. The actual impact fee will be calculated by the City using these capacity utilization percentages and service unit equivalents criteria that it is developing from customer billing records.

1.0 INTRODUCTION

1.1 PURPOSE

The objective of this study was to update the City of San Marcos (City) Wastewater Master Plan to reflect the need for upgrades and extensions of the sanitary sewer collection system based on the population and commercial growth that has occurred over the past 10 years and the projected growth into the future. The time horizon for this update is consistent with the update of the water master plan projecting future water system improvement needs out to the year 2027, which was completed in 2003 by Alan Plummer and Associates, Inc. The previous wastewater master plan, prepared by Black & Veatch in 1994, included a hydraulic model of the collection system with approximately 300 pipes. The master plan prepared in this study utilized a new hydraulic model with over 700 pipes (generally 10 inch and larger). A Capital Improvements Plan (CIP) was developed based on the evaluation of the existing condition of the system and the capacity required to handle future flows.

An additional objective in this study was an evaluation of two alternatives for meeting the treatment capacity needs for the future flows. The feasibility of expanding the existing plant to treat future flows was evaluated in the first alternative and the second alternative examined the option of constructing a new treatment facility in the Cottonwood Creek watershed. Each alternative examined the necessary line and lift station improvements to achieve the appropriate flow diversions.

Based on development of the 25-year master plan to year 2027, the study also needed to determine the proportionate cost of the improvements needed in the next 10 years and attributed to growth in the next 10 years, as basis for an updated wastewater impact fee.

1.2 SCOPE

In order to accomplish the above-mentioned objectives, the scope of the study included the following:

- Conduct flow monitoring to determine average dry weather flows and peak wet weather inflow and infiltration rates,
- Generate sewer model based on the data received from the City and the data gathered in the lift station and flow monitoring field studies,
- Establish criteria for capacity analyses and utilize the hydraulic model to evaluate the collection system at existing and projected flows,
- Analyze alternative improvements for existing deficiencies and future flows,
- Examine alternatives for location and amount of additional treatment capacity, and
- Complete impact fee analyses for each of the additional treatment capacity alternatives.

2.0 EXISTING WASTEWATER COLLECTION SYSTEM

2.1 OVERVIEW

The census population of San Marcos in 2000 was 34,733, which was a 20% increase from 1990. Robust growth is forecast to continue. Based on the population and employment data sources used in this study, by year 2027 the City is projected to provide wastewater service to a residential population of about 96,000 and employment of about 78,000.

The wastewater collection system of San Marcos includes approximately 275 miles of sewer lines ranging in size from 6 inches to 36 inches and over 40 lift stations. The system currently has a service area of approximately 60,000 acres and is projected to increase to over 122,000 acres by the year 2027. The flows from the service area are conveyed to the Main Lift Station via three main trunk sewers. Flow from the northeastern sections and central/western sections are conveyed to the Main Lift Station via two separate 24-inch lines and the flows from the southern section of the system are carried to the Main Lift Station by a 36-inch main. The Main Lift Station pumps the flows directly to the San Marcos Wastewater Treatment Facility via two 20-inch diameter force mains.

The San Marcos Wastewater Treatment Facility is permitted to receive an average daily flow (ADF) (maximum month) of 9 million gallons per day (MGD) and peak wet weather flow (PWWF) of 18 MGD, but the treatment process has been rated for a capacity of 7.6-MGD-ADF. Options for the expansion of this treatment capacity are discussed further in Section 6.0 of this report.

2.2 FLOW MONITORING PROGRAM

The wastewater master plan update has included measurement of wastewater flows at selected locations in the collection system. Flow monitoring data provides information on current operational conditions in the collection system and is also used for calibration of the hydraulic model. Flow data was collected at 15 manholes on trunk mains and at 9 lift stations.

2.2.1 Trunk Main Flow Monitoring

GSWW, Inc. installed, maintained, and removed the trunk main flow monitoring equipment and performed an engineering evaluation of the collected data. The flow monitoring period began April 7, 2003, and terminated June 27, 2003. This period included an extension from the original 60-day schedule in order to collect data during wet weather, which did not start until the first significant rainfall on June 2, 2003. The flow monitoring equipment was removed after five storms in June 2003 provided sufficient data for inflow/infiltration analysis. GSWW also provided installation and data collection for four recording rain gauges.

The flow monitoring study commenced with an evaluation of the collection system maps to identify potential gauging station sites. One consideration for site selection was to create meter basins that contained roughly the same linear footage of pipe upstream of the meter either to the perimeter of the collection system or to the next upstream meter on a trunk main. Site constraints control how well this goal is met. Five of the meters had one or more meters upstream.

Field investigations checked accessibility and hydraulic conditions at 22 manholes for installation of the 15 meters. One manhole on an 18-inch main (site GS-4) proved to have insufficient depths during nighttime low flows for useable data, and the meter was moved to an alternate gauging station (GS-12) to investigate the possibility of a suspected upstream wet weather bypass line. No crossover flow was detected during high wet weather flows, so the meter was moved to a final manhole (GS-12A) on June 12 for the remainder of the monitoring period.

The flow monitors were installed on pipes ranging in size from 10 inch to 36 inch in diameter. The site number, location, and pipe size are listed in Table 2.1. The flow meter locations and the meter basin boundaries are shown on Figure 2.1. Each meter basin was subdivided into smaller "subcatchments" which were used for generating flows in the hydraulic model based on the flow monitoring data. Figure 2.2 is a schematic flow diagram showing the relative locations of the gauging stations and the monitored lift stations.

TABLE 2.1. FLOW MONITORING SITES

Gaging Sta.	Address/Location	Pipe Size (inches)	Manhole ID#
GS-1	Interstate Highway 35 (IH 35) and CM Allen	18	260-030
GS-2	IH 35 east frontage opposite GS-1	36	261-037
GS-3	IH 35 north of San Marcos River	18	236-016
GS-5	Purgatory Ck at Gravel Street	12	283-001
GS-6	Linda Drive north of Sherbard	18	636
GS-10	Sessoms north of University	15	185-30
GS-11	River Road at Bugg Lane	24	238-011
GS-12A	McGehee and Lee	15	260-007
GS-13	Cheetham Street south of San Marcos River	18	235-029
GS-14	CM Allen and Mariposa	18	260-016
GS-15	IH 35 south of Highway 123	30	284-025
GS-17	Aquarena Springs Drive east of IH 35	24	165-11
GS-19	Nance Street south of San Antonio	10	234-006
GS-21	Gravel Street north of Mitchell	12	259-008
GS-22	Aquarena Springs Drive at Bobcat Stadium	15	162-17

Figure 2-1 Flow Meter Locations and Meter Basin Boundaries

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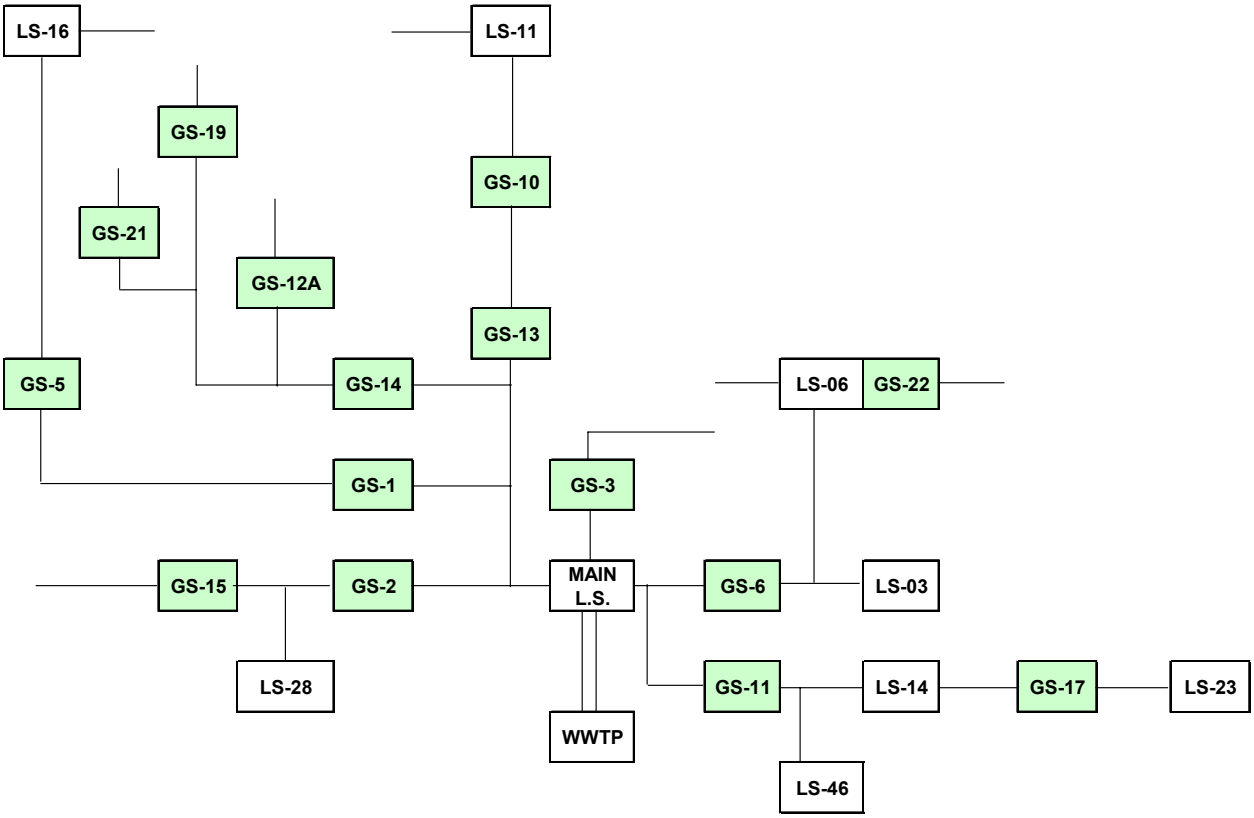

FIGURE 2.2. FLOW MONITORING SCHEMATIC

Flow monitor depth and velocity sensors were installed in the incoming pipe at a manhole. The data logger unit recorded the data and converted depth and velocity to flow rate. Details on the flow monitoring equipment, site conditions, calibration procedures, data evaluation, and results are presented in the *City of San Marcos Wastewater Master Plan Flow Monitoring Study Report* produced by GSWW (July 2003). The report contains graphical records of the continuous depths, velocities, and flow rates at each site for the duration of the monitoring program. The GSWW report has been submitted separately. The discussion in this section presents interpretative findings from the flow monitoring data.

The GSWW report also contains the data from rainfall monitoring, as shown below on Figure 2.3. Six storms on five days produced inflow responses at most of the flow monitoring sites (there were two separate storms on June 13).

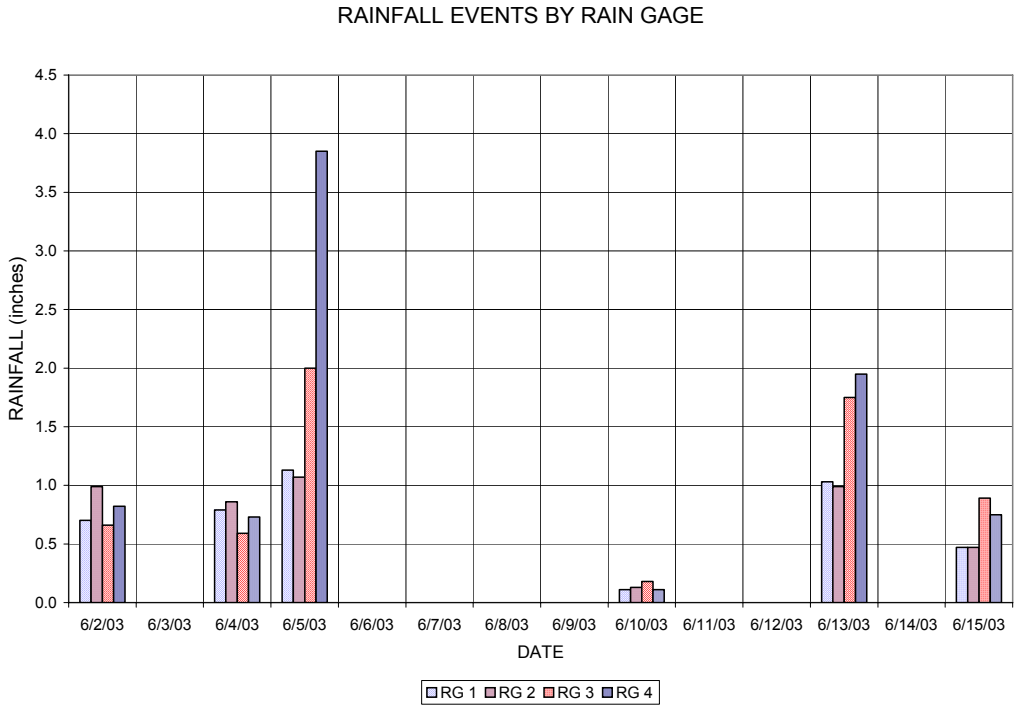

FIGURE 2.3. RAINFALL EVENTS BY RAIN GAUGE

Rain Gage Locations	
RG 1	911 River Road (Main Lift Station)
RG 2	Wonder World Drive east of IH 35 (Wonder World Lift Station)
RG 3	IH 35 and Aquarena Springs Drive (Old Plant Lift Station)
RG 4	1995 Bishop Street (Franklin Square Lift Station)

The June 5 storm not only produced the greatest rainfall amount and intensity, but it also had two antecedent storms to provide wet ground conditions that should have increased the rainfall-derived inflow and infiltration (RDII) generated by the June 5 storm. However, the storm was relatively short duration and did not have uniform widespread coverage. Figure 2.4 shows the rainfall intensity as inches per 15-minutes at each rain gage for the June 5 storm. RG 4, located on the western side of San Marcos at Franklin Square Lift Station, had 3.15 inches in one hour, which corresponds to about a 15-year design storm for one hour duration. The maximum 1-hour rain amount of 1.75 inches at RG 3 on the north side of the City (near IH 35 and Aquarena Springs Drive) is a storm with a return period of about 1 year, and 0.9 inch in one hour at RG 2 on the south side would occur on average four to five times a year.

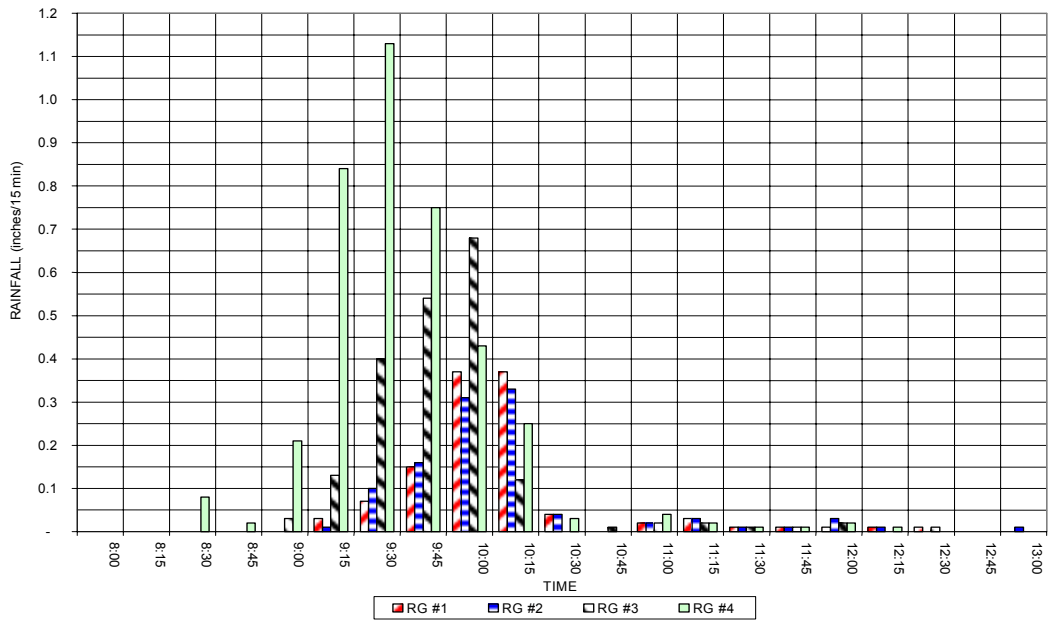

FIGURE 2.4. JUNE 5, 2003 STORM

The June 5 storm produced the highest flows in the collection system that were recorded during the flow monitoring period. Surcharging occurred at 6 of the 15 meter locations. Table 2.2 shows for each monitoring site the maximum manhole water level versus pipe diameter and the ratio of peak wet weather flow (PWWF) to average dry weather flow (ADWF).

TABLE 2.2. SUMMARY OF TRUNK MAIN FLOW MONITORING

Site #	Location	Pipe Size (inches)	Maximum Level (inches)	Average Dry Weather Flow (gallons per minute)	Peak Wet Weather Flow (gallons per minute)	PWWF/ ADWF
GS-1	IH 35 and CM Allen	18	17.0	535	1,750	3.3
GS-2	IH 35 opposite GS-1	36	11.8	500	2,765	5.5
GS-3	IH 35 north of San Marcos River	18	15.3	340	1,080	3.2
GS-5	Purgatory Ck at Gravel St	12	74.1	155	860	5.6
GS-6	Linda Dr north of Sherbard	18	19.6	675	1,640	2.4
GS-10	Sessoms north of University	15	44.9	570	1,170	2.1
GS-11	River Road at Bugg Lane	24	14.0	350	1,760	5.0
GS-12A	McGehee and Lee	15	4.9	115	510	4.4
GS-13	Cheetham south of San Marcos River	18	11.6	775	1,770	2.3
GS-14	CM Allen and Mariposa	18	20.7	520	2,280	4.4

Site #	Location	Pipe Size (inches)	Maximum Level (inches)	Average Dry Weather Flow (gallons per minute)	Peak Wet Weather Flow (gallons per minute)	PWWF/ ADWF
GS-15	IH 35 south of Highway 123	30	8.9	290	1,450	5.0
GS-17	Aquarena Springs east of IH 35	24	7.3	180	530	2.9
GS-19	Nance St south of San Antonio	10	8.6	100	635	6.4
GS-21	Gravel St north of Mitchell	12	42.0	33	374	11.3
GS-22	Aquarena Springs at Bobcat Lift Station	15	42.2	280	700	2.5

The wide range in values for the PWWF/ADWF ratio to some extent should reflect the uneven rainfall distribution over the system for the June 5 storm. This is in addition to the different extent of defects allowing infiltration/inflow (I/I) in older versus newer parts of the system. Also note that the meter at GS-12A was not installed until June 12 and did not catch the June 5 storm. The peak flow at GS-12A shown above occurred during the second storm on June 13.

At site GS-5 the June 5 storm apparently caused a change in the downstream hydraulics, possibly due to some debris deposition or movement causing a partial blockage. The manhole level surcharged to a depth of 74 inches during the June 5 storm. Surcharging then occurred daily after June 5 for the remainder of the monitoring period. Flow depths were at normal levels after June 5 only during low nighttime flows.

The metered peak flow at GS-22 shown above was controlled by downstream conditions, in effect, the pumping capacity of Bobcat Stadium lift station (LS 6). The lift station was being upgraded during the flow monitoring period, which prevented monitoring lift station influent flows by the method described in Section 2.2.2. The gauging station was on the north side of Aquarena Springs Drive at the first manhole upstream of the lift station due a junction of two trunk mains at the next manhole upstream. A main carrying flow to the lift station from a smaller area south of Aquarena Springs Drive was not metered. During the June 4 storm the flow meter logged a maximum water level of 124 inches (over 10 feet) and no velocity or flow, because the lift station was off-line due to the storm. The GS-22 manhole was surcharged at 33 to 42 inches for about an hour during the June 5 storm due to high wet well level as influent exceeded pumping rate. On account of the unmetered main into the lift station from the south, the GS-22 flow rate during surcharging does not reflect the pumping rate.

All of the flow monitoring sites except GS-12A and GS-21 had one or more lift stations upstream that to some degree could influence metered flows. The natural flow patterns generated upstream of a lift station are converted to periodic pumping rates experienced downstream of the station. In most cases the lift stations were too small or distant to produce a definite impact on metered flows. However, almost all of the flow recorded at GS-11 (manhole on 24-inch main in River Road) was delivered by upstream pumping from River Road lift station (LS 14) and from Gary Job Corps lift station (LS 46), but the

pumping cycles were not obvious in the meter data, probably due to having two stations with relatively short cycle times and the attenuating effect of the 24-inch line's flat slope. As for significant lift stations and other metering sites, GS-6 on the 18-inch main in Linda Drive was downstream of both Old Plant lift station (LS 3) and Bobcat Stadium lift station (LS 6), sites GS-10 and GS-13 were downstream of north LBJ lift station (LS 14), and sites GS-5 and GS-1 were downstream of Franklin Square lift station (LS 16). If the peak influent rates at a lift station exceed the pumping rate, the downstream flow meter does not see the true peak flow generated in the lift station's service area.

The peak wet weather flows indicated above are the single highest rates recorded at the 15-minute sampling interval of the flow monitors. The flow monitoring results typically show a rapid inflow response to the rainfall, as well as a fast recession in the inflow after the rainfall. This is illustrated in the charts showing flows at each monitoring site during the wet week of June 2, 2003, which are included in Appendix A. The smaller meter basins typically show a faster response due to shorter travel time. This type of inflow response, especially illustrated in the charts for GS-19 and GS-21, suggests defects providing a relatively direct connection between rainfall runoff and the collection system, e.g., at manhole and cleanout covers and roof drain connections. Smoke testing is a common method of identifying defects that can allow high inflow rates.

The flow monitoring data was analyzed to estimate an inflow rate for each meter basin for each storm. The inflow rate was calculated as the increase in metered flow rate due to the storm minus the normal change (plus or minus) in the flow rate in dry weather for the same time of day (from start of rain to peak metered flow). For example, if the flow during a storm increased by 100 gallons per minute (gpm) from midnight to 1:00 A.M. when the dry weather flow for the same time of day would normally drop by 50 gpm, the implied inflow component of the measured flow is 150 gpm.

The rain gage data was analyzed to determine the rainfall intensity for each storm for each site's selected time of concentration (time from start of rain to metered peak flow). Two methods were used to determine a relationship between inflow rate and rainfall intensity, which allows projection of inflow (Q) for selected design storms with higher rainfall intensity (I) than occurred during the monitoring period. The Q-I calculation sheets are included in Appendix B. One method uses regression analysis with the Q and I points to derive the coefficient ("a") and exponent ("b") for the relationship $Q = a * I^{b}$. The other method assumes that inflow rate is directly proportional to rainfall intensity and service area (A), i.e., Q = KIA, where K can be thought of as a "leakiness factor." There are pros and cons to each method. In both cases the inflow parameters (a, b, K) reflect conditions for the entire contributing area upstream of the meter including service areas for any upstream meters and lift stations.

Table 2.3 summarizes the results of the inflow analysis for rainfall intensities projected for a storm with a 5-year recurrence interval. The projected inflow rates (gpm) are converted to unit rates (gallons per day [gpd] per acre and gpm per 1,000 linear feet [lf]) for comparison of meter basins.

TABLE 2.3. INFLOW ANALYSIS

	Subbasin		Cumulative		Inflow by Q = a*I^b				Inflow by Q = KIA			
Meter	Acres	LF	Acres	LF	GPM	b	GPD/ Acre	GPM/ KLF	GPM	K	GPD/ Acre	GPM/ KLF
GS-1	545	66,519	1,323	162,095	1,421	0.868	1,550	8.8	1,662	0.73	1,810	10.3
GS-2	233	31,708	1,037	145,410	2,633	1.112	3,660	18.1	2,337	1.03	3,250	16.1
GS-3	301	27,528	301	27,528	1,129	0.628	5,400	41.0	2,046	3.52	9,800	74.3
GS-6	77	9,000	612	52,832	1,153	0.713	2,710	21.8	1,773	1.13	4,170	33.6
GS-10	677	85,063	939	119,695	778	1.289	1,190	6.5	608	0.30	930	5.1
GS-13	238	29,041	1,177	148,736	1,138	0.800	1,390	7.7	1,485	0.73	1,820	10.0
GS-14	211	37,795	913	124,435	1,723	0.752	2,720	13.8	2,509	1.75	3,960	20.2
GS-15	353	54,567	739	103,094	2,800	1.910	5,460	27.2	910	0.71	1,770	8.8
GS-17	276	22,296	442	35,342	1,121	1.263	3,650	31.7	763	0.67	2,490	21.6
GS-19	343	35,914	343	35,914	463	0.751	1,950	12.9	606	0.81	2,540	16.9
GS-21	168	23,650	168	23,650	439	1.370	3,770	18.6	284	0.88	2,440	12.0

Note that GS-5, GS-12A, and GS-22 had insufficient data for reliable inflow analysis for reasons mentioned earlier. Also, Table 2.3 does not include site GS-11 since its flows reflect pumping rates at two upstream lift stations rather than actual inflow in the lift stations' service areas.

The various unit inflow rates shown above indicate that meter basin GS-3 has the most significant inflow problems. This includes the Rio Vista area for which the City already has rehabilitation/replacement piping improvements planned (see Figure 2.1 for location of meter basin GS-3).

Not far behind is meter basin GS-6, which includes the service areas for Bobcat Stadium LS 6 and Old Plant LS 3, which are primarily on the west side of IH 35 just north of the Rio Vista area. (The overall ranking is based on assigning each of the eleven meter basins a value of 1 to 11 according to its rankings – best to worst – for the five "gpd/acre," "gpd/1,000 lf'," and "K-value" evaluations in Table 2.3. By this method, meter basin GS-3 has a rating of 54, meter basin GS-6 has a rating of 42, and meter basin GS-10 has a rating of 5 for the lowest.)

Inflow is also significant in meter basin GS-14 (rating of 38), which includes the contributing areas for meters GS-12A, GS-19, and GS-21. Since the K values (leakiness factors) for the GS-19 and GS-21 meter basins are 0.8 and 0.9, respectively, compared to an average of 1.75 for the cumulative GS-14 meter basin, it appears that the subbasin for GS-14/GS-12A downstream of GS-19 and GS-21 has the higher inflow rates. Also consider that the GSWW flow monitoring data definitely shows that the GS-21 and GS-19 meter basins have high inflow rates based on observed PWWF/ADWF factors of 11 and 6, respectively, for the June 5 storm (see Table 2.2 and flow charts in Appendix A). The flow monitoring data may not show as high a peaking factor for GS-14 because a portion of its service area was farther away from the part of the collection system that received the highest rainfall intensity for the June 5 storm. The projected unit inflow rates for ranking purposes use rainfall intensity for a 5-year design storm distributed uniformly over the service area.

The data collected in the June 2003 storms does not indicate significant residual rainfall-induced infiltration, although the June storms were not of long duration. Minimum nighttime flows were generally at near-normal levels a day after the storms. This does not preclude a steady base infiltration component of the flow due to high groundwater that is possible in some areas. An estimate of base infiltration can sometimes be made from the shape of the dry weather diurnal flow curve.

The flow monitoring data was reviewed to select days with typical dry weather flow patterns. A diurnal flow pattern was produced for each monitoring site by averaging the meter readings at each 15-minute interval for all of the selected days. This analysis also produced a dry weather average daily flow rate for each monitoring site. The diurnal curves are included in Appendix C.

Regarding a groundwater infiltration (GWI) component of dry weather flow, companies in the flow monitoring business sometimes use an empirical relationship to estimate base infiltration from metering data (see "Infiltration/Inflow Source Identification by Comprehensive Flow Monitoring," Journal WPCF, Vol. 53, No. 11, p.1620, November 1981). The relationship utilizes the assumption that the wastewater generation pattern (with no infiltration) for small residential areas is similar among different systems, based on typical human behavior and extensive flow monitoring experience. Base infiltration is estimated from the metered minimum nighttime flow and the average dry weather flow. The empirical relationship is that the wastewater component of total average daily flow equals {the average daily flow minus the minimum nighttime flow} divided by 0.9 (the "90% rule"). Therefore, the base infiltration component is the metered average daily flow minus the wastewater component as derived above. This relationship is not applicable for areas where a substantial non-residential wastewater component distorts the typical residential wastewater generation pattern or for collection systems where time of travel is out of the ordinary due to unusual slopes or network configuration.

It is likely that not much of the flow monitoring data for San Marcos is reliable for estimating base infiltration due to the widespread distribution of non-residential land use among the meter basins, given the commercial development along the IH 35 corridor and the extents of Texas State University. Nevertheless, with this caveat, the calculated base infiltrations are presented in Table 2.4.

Meter basin GS-3 shows the highest potential GWI rate in relation to average dry weather flow, although the 65% ratio may be high if the commercial development along Aquarena Springs Drive increases minimum nighttime flow. Meter basin GS-3 is on the north side of the San Marcos River in a low lying area, which increases the opportunity for a high groundwater table to produce infiltration. Also, piping defects are evident from the high rainfall-induced inflows observed during flow monitoring. The pipe rehabilitation/replacement project that is planned for the Rio Vista area should reduce the infiltration and inflow in meter basin GS-3.

TABLE 2.4. ESTIMATED BASE INFILTRATION BY THE "90% RULE"

Meter	ADWF (gpm)	Minimum Flow (gpm)	GWI (gpm)	GWI/ADWF
GS-3	342	234	221	65%
GS-13	774	511	482	62%
GS-14	521	327	305	59%
GS-15	290	178	166	57%
GS-22	282	146	130	46%
GS-5	154	77	69	45%
GS-6	675	331	293	43%
GS-21	33	15	14	42%
GS-1	535	251	219	41%
GS-11	352	162	141	40%
GS-17	181	68	55	30%
GS-19	102	36	29	29%
GS-10	571	190	147	26%
GS-12	113	36	27	24%
GS-2	389	115	85	22%
LS-11	140	59	50	36%
LS-16	80	25	19	24%
LS-28	24	8	7	28%

GS-13 is on the south side of the San Marcos River and also shows a high calculated base infiltration. Meter basin GS-13 includes the upstream meter basin GS-10, which shows a low calculated base infiltration, as expected due to its hillier terrain. This implies that the subbasin between GS-13 and GS-10 may have very substantial base infiltration, provided that high nighttime flows are not generated in the university area. The overall GS-13 meter basin shows less of an inflow problem that most of the other meter basins.

GS-14 is similar to GS-13 in that the calculated base infiltration for the overall meter basin is high while the upstream meter basins GS-12A, GS-19, and GS-21 are lower, suggesting that the GS-14 subbasin is worse than the overall GS-14 meter basin. The GS-14 subbasin is mostly a low lying area south of downtown in the vicinity of Purgatory Creek. Meter basin GS-12A is a mix of residential and downtown business district. A very low rate of nighttime wastewater generation in the business district could disguise a higher rate of base infiltration than suggested by the low calculated number. Meter basins GS-19 and GS-21 are primarily residential areas, so base infiltration estimated by the 90% Rule may be more reliable for these areas. Meter basin GS-5, which is south of basins GS-19 and GS-21, is also primarily residential and shows a calculated base infiltration ratio of 45% that is close to GS-21 at 42%.

The remaining meter basins contain a substantial amount of non-residential land use such as the restaurants, motels, shopping centers, and other businesses in the IH 35 corridor. The 90% Rule is not expected to produce useable infiltration estimates from this metered flow data.

The diurnal curves and average daily flows based on metering data were also used as input data for the hydraulic model. All of the model's subcatchments within a meter basin were assigned the flow pattern for that meter basin (as ratio of 15-minute flows to average daily flow). The calibration process for the hydraulic model generated values for gallon/day per resident and gallon/day per employee for each meter basin. The calibration procedure adjusted the unit flow rates for each meter basin so that the diurnal flows produced by the model at each monitoring site closely matched the flows seen in the meter data.

2.2.2 Flow Monitoring at Lift Stations

A lift station's service area is analogous to a subbasin generating wastewater that goes to a flow monitoring site. During the flow monitoring phase of the project, PBS&J derived influent rates at selected lift stations using (a) SCADA data that was supplied by the City, and (b) pumping rates that were determined by PBS&J. The recorded SCADA data was the time of each pump start and stop, which allowed calculation of the time-on and time-off durations for each pump cycle. The average influent rate during each on-off (and off-on) cycle is as follows:

$$I = D / (T_{off}/T_{on} + 1)$$

where I = influent rate, gal/min

D = pump capacity, gal/min

T_{off} = pump off time, minutes

T_{on} = pump run time, minutes

PBS&J conducted pumping tests to determine pump discharge rates. The wet well fill rate (inches per minute) was measured immediately before and after a pump run. With known wet well dimensions providing unit volume (as gallons per vertical inch), the measured fill rate was converted to influent rate in gallons per minute. The drawdown rate during a pump run was also measured in inches per minute and converted to a net discharge rate (gallon/minute) reflecting the pumping capacity minus the influent rate. Therefore, the pump capacity was taken to be the measured drawdown rate plus the average influent rate immediately before and after the pump run. (For example, if the wet well level drops at a rate of 200 gpm while the influent rate is 300 gpm, the pump capacity is 500 gpm.)

The pump capacities determined from the pumping tests were used with the T_{off} and T_{on} data from SCADA to derive lift station influent rates during the flow monitoring period. It should be noted that better data resolution occurs with short cycle times. Long pump run times yield the average influent rate over the pump cycle and cannot identify the peak influent rate during the pump run.

The original scope of the project intended to monitor five major lift stations that would be included in the hydraulic model. Initially the plan was to use data loggers installed at each station to collect the on/off pump data. When the flow monitoring phase commenced, the City determined that it could record and furnish the on/off data from its SCADA system. At that point, nine lift stations were selected for pump testing and flow monitoring. Table 2.5 lists these nine stations and the tested pumping rates.

TABLE 2.5. LIFT STATION PUMP TESTING

		Pump # and Capacity (gpm)		
LS#	Name	#1	#2	#3
3	Old Plant	340	385	–
5	Care Inn	135	100	–
11	North LBJ	880	765	–
12	Oakridge Drive	75	100	70
14	River Road	1,190	2,400	–
16	Franklin Square	685	685	340
23	CFAN	155	165	–
28	Hills of Hays	415	485	–
46	Camp Gary	–	600	620

Peak influent rates during the June storms could not be determined from the lift station SCADA data due a combination of factors: pump run times too long to derive a reliable peak flow; the storms causing interference/outage of the SCADA system; and power outage at some lift stations.

The SCADA data was used to determine average dry weather flow and the typical diurnal flow pattern at each lift station. These results are included in Appendix C for seven lift stations in the hydraulic model.

The dry weather diurnal curves can be used to estimate a base infiltration component of residential average daily flow, as discussed in Section 2.2.1. The service areas for three of the lift stations have primarily residential land use – North LBJ LS 11, Franklin Square LS 16, and Hills of Hays LS 28. The calculated base infiltration rates for these three areas are included in Table 2.4.

The flow monitoring data was used for calibration of the model. The unit flow rates for the population and employment in each lift station service area were adjusted during the calibration process so that the model-generated flows were close to the flows derived from the SCADA data. The observed diurnal patterns were input to the model to establish the patterns in the model results.

2.3 COLLECTION SYSTEM DATA

The PBS&J modeling team analyzed multiple sources of collection system data to develop the model of the existing San Marcos sanitary sewer system. Physical and hydraulic data was acquired for pipes identified for the model (generally 10 inch in diameter and larger plus hydraulically significant smaller

lines). This section covers the databases acquired and utilized to build the physical aspect of the existing collection system in the model.

The City furnished PBS&J with the paper and digital mapping of the collection system. The digital Geographic Information System (GIS) mapping replicated the paper maps and provided the location and size of all the pipes and force mains in the collection system, as well as the location of manholes and lift stations. Pipe elevation data was not available for the GIS database. Approximately 700 pipes were selected for the hydraulic model. The City also furnished the CAD drawing file and database for the 1994 Black & Veatch sewer model, which contained approximately 300 pipes and included pipe elevation data. Most of the network configuration in the Black & Veatch model matched the GIS network configuration very well, but manhole locations did not coincide in a few areas.

In order to determine pipe invert elevations (and slopes) for the pipes in the PBS&J model that were not in the Black & Veatch model, PBS&J reviewed construction plans. The City furnished copies of construction plans for about 40 sewer projects in either hardcopy or digital format. Some of the plans were for recent projects or projects under construction which were not yet in the City's GIS mapping but needed to be in the sewer model. Using whatever data was available on the construction plans (manhole coordinates, distances and bearings or deflection angles, etc.), PBS&J generated the alignments of the new lines digitally and added them to the model on the GIS coordinate system.

Some older sewers that are in the new hydraulic model did not have construction plans available. PBS&J furnished the City with maps identifying 14 line segments with about 140 manholes that still needed invert elevation data. The City utilized its GPS surveying equipment to obtain pipe invert and manhole rim elevations for these remaining pipes. About 20 of the targeted manholes could not be located in the field. The invert elevations for these pipes were estimated from the data available for upstream and downstream manholes, and ground elevations were estimated from topographic maps.

Lift station data provided by the City in construction plans and reports, along with pump test data acquired during the lift station evaluations by PBS&J (see Section 2.2.2), were incorporated in the model. Ten of the City's larger lift stations, out of over 40 lift stations in the collection system at the time, were used in the model of the existing system during the flow monitoring period. The 10 lift stations active in the initial model were Old Plant (LS 3), Bobcat Stadium (LS 6), North LBJ (LS 11), River Road (LS 14), River Ridge (LS 15), Franklin Square (LS 16), CFAN (LS 23), Hills of Hays (LS 28), Baptist Academy (LS 36), and Camp Gary (LS 46). Other lift stations were not included in the model generally because of (a) small pumping capacity, (b) location in a part of the collection system not in the model (the 6- and 8-inch mains), and/or (c) limited effect of the pumping on the flow patterns at the flow monitor gauging stations.

It should be noted that the "existing system" in regard to hydraulic modeling is not indicative of the system that exists today (2005). The starting version ("existing system") is the collection system as it was

during the flow monitoring period. This system, including the ten lift stations listed above, was needed for model calibration using the flow monitoring data, which included generating the population and employment wastewater unit flow rates for each meter basin. But parts of the calibration model have been changed significantly by projects that were under construction during the flow monitoring period. One of the recent projects added to the model is the Cottonwood Creek interceptor system. The Cottonwood Creek project has significantly rerouted flows in the southeast portion of the collection system. The interceptors and lift stations in this project were not in service yet during the flow monitoring period, and they are not included in the version of the hydraulic model used for calibration of wastewater flow generation parameters. The Cottonwood Creek project has allowed decommissioning of Civic Center LS 7, Factory Shop LS 39, and Clovis Barker LS 45, and it intercepted flows pumped from Hills of Hays LS 28. The new interceptors and the new Cottonwood Creek lift station and San Marcos River lift station are included in the hydraulic model going forward. The same scenario applies for the recently completed IH 35/Fairlawn interceptor and Blanco River lift station—excluded for model calibration but included for modeling future systems. Section 3 of this report provides details of projects included for development of the future modeling scenarios.

The existing system also differs from the future system regarding use of lift station lag pumps. In the model of the existing system for calibration, lag pumps were allowed to run when needed for peak flows to reflect actual operations. But in the models for future system improvements, lag pumps are locked out for the purpose of determining needed pumping capacity improvements, in keeping with requirements of the Texas Commission on Environmental Quality (TCEQ) for adequate pumping capacity with the largest pump out of service.

The Main Lift Station LS 1 was not included as a lift station in the hydraulic model. The Main Lift Station pumps directly to the wastewater treatment plant headworks structure through parallel 20-inch force mains. The station has four pumps nominally rated at 3,500 gpm. The actual individual pump and combined firm capacities of the station, for representation in the hydraulic model, have not be measured because the station does not have flow meters and submergence of influent lines eliminates the possibility of drawdown pump testing. However, a project is underway at the lift station at this time for piping improvements that will include flow meters. The City has reported that the lift station has sufficient pumping capacity to overload the treatment plant headworks. The flow interceptions going to the new San Marcos River lift station in the Cottonwood Creek interceptor project have reduced flows going to the Main Lift Station.

The Eastside Interceptor Phase I project (approximately 6,500 feet of 36- and 30-inch pipe on the east side of IH 35 south of Main Lift Station, constructed in 1993) included a triple barrel inverted siphon under the San Marcos River with 16-, 18-, and 22-inch high-density polyethylene (HDPE) siphon barrels. The inverted siphon is represented as such in the hydraulic model with HDPE pipe inside diameters and pipe elevations from the construction plans. Any degree of capacity reduction due to sediment deposition (smaller pipes, in effect) is unknown. A special study would be needed to use water level and flow rate

measurements for comparison to the expected capacity and head characteristics of the each barrel as a clean pipe. Such investigation might indicate the need for cleaning of the siphon barrels to eliminate surcharging and overflows under peak flow conditions. The hydraulic model, with clean siphon barrels, does not indicate that the inverted siphon causes upstream spills.

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3.0 MODEL DEVELOPMENT

The objective of the model development was to create a database that most accurately depicts the actual physical, hydraulic, and operating conditions that exist in the collection system of San Marcos. The two basic parts of the model are the physical component, which consists of the piping system and lift stations, and the flow component, which for the San Marcos system was developed from population and employment data for wastewater flows and from a design storm for the rainfall-induced inflow/infiltration flows. The model then provides an analytical tool that can be used to verify current performance of the collection system and project future performance of the system. Analyses of the system's future performance allow for forecasting of necessary system improvements and additions.

3.1 MODELING SOFTWARE

Wallingford Software's dynamic modeling software, InfoWorks, was selected as the computer program for the hydraulic modeling of the San Marcos sewershed. InfoWorks Collection System (CS) is a dynamic hydraulic modeling software package used to model sewage collection systems and was chosen for its versatility in modeling complex issues such as surcharged pipes, spilling manholes, lift station pumping cycles, and attenuation of pumped flows. The software serves as a master database of physical network and hydraulic data for sewer systems and provides the user with capabilities to store multiple networks of unlimited pipes and nodes, apply actual or synthetic rain events to the system, and view real time flow through pipes.

The InfoWorks CS software is compatible with database management, and geographical and computer aided design CAD software packages, including Microsoft (MS) Excel, GIS and AutoCAD. MS Excel can be used to create data charts and perform calculations outside of the modeling software. The GIS and CAD software compatibilities allow for views within the model to include aerial maps and other files generated outside of the model. These compatibilities also allow for straightforward importing and exporting of files for generation of charts and maps to display the modeling results.

3.2 POPULATION AND EMPLOYMENT DATA

Population and employment data was the basis for projected wastewater flows used for the wastewater master plan. Past, present and future population and employment data was gathered from the City of San Marcos and the Capital Area Metropolitan Planning Organization. The Capital Area Metropolitan Planning Organization (CAMPO) is the federally designated metropolitan transportation planning organization for Williamson, Hays, and Travis counties. CAMPO has been forecasting and estimating population and employment growth in Central Texas for over 40 years.

The population and employment data for the present and future years was gathered from the City of San Marcos and the CAMPO Population Forecasts and Estimates reports from 2000 and 2003, and the revised

report from 2002. Table 3.1 outlines the sources used for each year. A number of sources had concurring data, thus there are multiple sources listed for some years. The City's planning staff and the Water Master Plan consultant made minor revisions to some of the CAMPO projections based on better knowledge of local conditions. For consistency, the wastewater master plan and the water master plan use the same population and employment database and time intervals, although the service areas are different.

TABLE 3.1. SOURCES OF RESIDENTIAL AND EMPLOYMENT POPULATION DATA AND PROJECTIONS USED TO DEVELOP THE MODEL

	City of San Marcos	2000 CAMPO Report	Revised 2002 CAMPO Report	2003 CAMPO Report
2003 Residential		X		
2003 Employment		X		
2007 Residential			X	
2007 Employment				X
2017 Residential	X		X	X
2017 Employment	X			X
2027 Residential				X
2027 Employment	X			X

The population and employment data obtained from CAMPO was presented in traffic serial zones (TSZs). These zones are defined by CAMPO as the smallest geographically designated area used for analysis of transportation activity such as data collection and travel movements within, into, and out of the urban area. There are over 1,000 TSZs in Williamson, Hays, and Travis counties combined. Over 125 of the 1,000 TSZs were utilized in gathering the population and employment data for the wastewater master plan update.

Many fields of data were included in the TSZ database provided by CAMPO. The data fields utilized in our study of the collection system included the areas and the population and employment for 1990, 1997, 2000, 2007, 2017, and 2027 associated with each TSZ. Population and employment for years 2003 and 2014 were interpolated from 2000, 2007, and 2017 CAMPO data and used for generating flows for purposes of impact fee analysis.

As indicated in the "City of San Marcos Water and Wastewater Impact Fee Analysis 2002–2012" report prepared in 2002, San Marcos has a wastewater service area of approximately 59,600 acres with a population of approximately 51,350 and over 18,000 dwellings. According to calculations in the Impact Fee Analysis report, the wastewater service area is estimated to have a compounded average annual growth rate of 2.6%. The following chart on Figure 3.1 shows the projected population and employment.

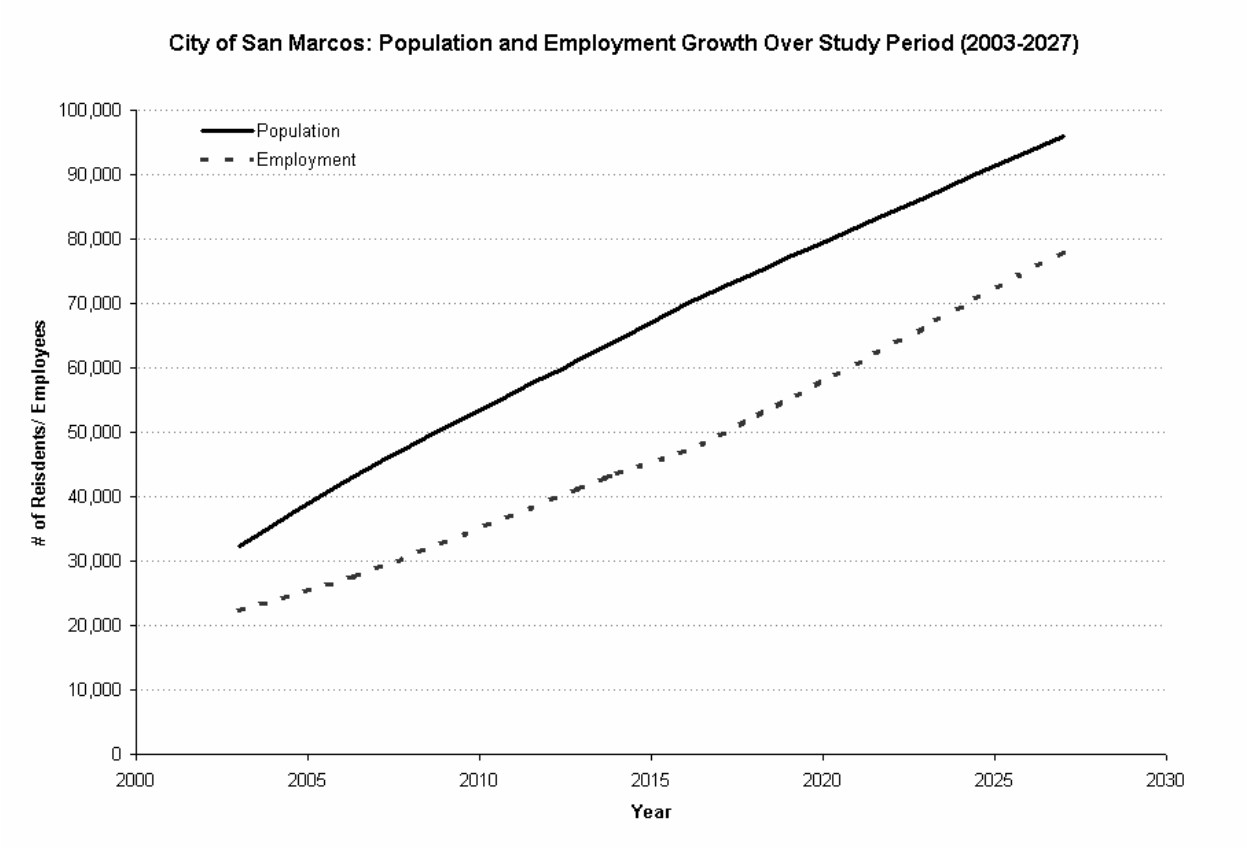


FIGURE 3.1. PROJECTED POPULATION AND EMPLOYMENT GROWTH

3.3 COLLECTION SYSTEM

The physical and hydraulic data gathered (from sources discussed previously in Section 2.3) for the existing conditions of the San Marcos collection system were compiled into a new GIS file and imported into the modeling software. The data imbedded in the GIS file and imported into InfoWorks CS included the spatial location of the pipes and manholes, the City's pipe and manhole identification numbers, pipe size, ground and invert elevation data, and delineated sewersheds. This database of existing conditions provided a base on which the model of the San Marcos collection system was built in InfoWorks CS.

The existing collection system was assembled in the InfoWorks CS software using the database imported via the GIS file. The connectivity of pipes was established in the model by importing the invert elevation data collected on the system. Problems with missing information and/or incorrect data, particularly information and data relative to the connectivity of pipes, were made obvious upon completion of the data merge into the modeling software. The data sources utilized to gather the information necessary to build the model were referenced to solve these issues and where the problems could not be resolved, the model was used to infer the data. Once the physical model of existing system was set, then the system improvements associated with the future systems could be established.

The existing system was used as a base on which to build the future systems for the years 2007, 2014, 2017, and 2027. The CIP timeline provided in the 2002–2012 San Marcos Water and Wastewater Impact Fee Analysis report dictated the order in which the projects were added to the future systems. In cases where available, construction plans were used to incorporate the alignments of current CIP projects to the system; otherwise, a GIS file provided by the City supplied the project locations for the planned CIP projects and TCEQ design criteria and U.S. Geological Survey (USGS) maps were used to approximate invert elevations, as discussed further below.

The GIS file provided by the City supplied the project locations for the CIP projects that are planned, but not yet designed. The x and y coordinates of these project locations were extracted from the GIS file and used to import the future lines into the model. USGS contour maps were used to estimate ground elevations in the project areas and the TCEQ design standards outlined in Table 3.2 were used to approximate the invert elevations of the future pipes.

TABLE 3.2. CHAPTER 317, TCEQ DESIGN CRITERIA FOR SEWER SYSTEMS

Pipe Size (inch)	Minimum Slope (%)	Maximum Slope (%)
8	0.33	8.40
10	0.25	6.23
12	0.20	4.88
15	0.15	3.62
18	0.11	2.83
21	0.09	2.30
24	0.08	1.93
27	0.06	1.65

The 2007 system built in the model included projects that are on the CIP schedule to be completed before 2007, as well as projects that have been built between 2003–2004 (since this study began). These projects have all been designed and are set for construction within the next couple of years. Table 3.3 provides an outline of the projects added and the data used to incorporate the improvements made to the existing system for 2007.

The CIP projects listed in Table 3.4 were included in the 2014, 2017, and 2027 systems, in addition to the projects already built-in to the 2007 system. Table 3.4 includes projects that are listed in the San Marcos CIP as scheduled to be completed between 2007 and 2014. The majority of these projects are dependent on land development in areas of preferred growth.

TABLE 3.3. CIP PROJECTS ADDED TO THE EXISTING SYSTEM FOR 2007

CIP Project Name	Description	Information Used for Alignment in InfoWorks CS
Craddock Avenue Improvements	8-inch line replacement/upsize	Craddock Avenue Improvements: Plan and Profile Sheets from Byrn & Associates Date on Plans: June 2004
Riverside MKT Interceptor	27-inch line installation from Bobcat lift station to Main lift station	Sink Creek Wastewater Interceptor Project: Plan and Profile Sheets from Bury & Partners Date on Plans: January 2002
IH 35 Fairlawn/Blanco River LS	36- and 30-inch line installation and installation of Blanco River lift station	Plans for Construction of IH 35/Fairlawn Force Main & Interceptor Turner, Collie, & Braden, Inc. Date on Plans: June 2003
North IH 35 Interceptor	24- and 18-inch line installation	Phase One North IH 35 Wastewater Improvements Baker-Aicklen & Associates Date on Plans: June 2003
Eastside Phase III	30- and 33-inch line installation	Willow Creek Wastewater Improvements: Plan and Profile Sheets from Turner, Collie, & Braden Date on Plans: September 2003
Cottonwood Creek Interceptor/ Cottonwood Creek LS and Force Main/San Marcos River LS	Line installation and installation of Cottonwood Creek LS and San Marcos River LS	Cottonwood Creek Sanitary Sewer Improvements Freese & Nichols Byrn & Associates Date on Plans: June 2002
Cottonwood Creek Subdivision Offsite Water and Wastewater Extension Improvements	18- and 15-inch line Installation	Cottonwood Creek Subdivision Offsite Water and Wastewater Extension Improvements: Baker-Aicklen & Associates Date on Plans: December 2003
McCarty Lane	12-inch line installation	McCarty Lane Wastewater Interceptor Byrn & Associates Date on Plans: December 2003
Sessoms/Loop 82	Line replacement (18- and 20-inch pipe)	Included details of this project as per a memo from the City of San Marcos
San Marcos Factory Shop Expansion	15-inch wastewater line	San Marcos Factory Shop Expansion Turner, Collie, & Braden Date on Plans: March 1999

TABLE 3.4. PROJECTS SCHEDULED FOR CONSTRUCTION BETWEEN THE YEARS OF 2007–2014 AND INCLUDED IN THE 2014, 2017 AND 2027 FUTURE SYSTEMS

CIP Project Name	Description as per City of San Marcos CIP	Information Used for Alignment in InfoWorks CS
Roosevelt Street Wastewater Improvements	Upsize 12-inch wastewater lines to 18-inch lines in the Roosevelt Street Neighborhood Area. This project was redefined in the proposed CIP in Section 5.0 of this report	City of San Marcos Wastewater System Improvements: Plan and Profile Sheets from Pledger Kalkomey, Inc. Date on Plans: March 2002
Sink Creek Interceptor Phase I	Construct 21- and 18-inch lines from Riverside MKT Interceptor (Bobcat LS) across Aquarena Springs and Post Roads and along Lime Kiln Road	
Sink Creek Interceptor Phase II	Construct 18- and 21-inch line from Sink Creek Phase I along Sink Creek to connect to various lift stations at N. LBJ & Holland, N. LBJ & Oak Ridge, and N. LBJ & Craddock	
Purgatory Creek Interceptor Phase I	Construct 24-inch line from the MKT RR tracks at Eastside III and Willow Creek sewer intersection to Hunter Road. Serves as a continuation of the interceptor sewers required to provide service along extension of Wonder World Drive	
Purgatory Creek Interceptor Phase II	Construct 24-, 21-, and 18-inch gravity sewer beginning at Purgatory Creek and Hopkins and extending to Columbia at Hazleton	
Uhland-River Road Interceptor	Upsize existing 24-inch sewer from Uhland Road to the River Road LS to carry wastewater flows from full development North of Blanco River. Project also increases the capacity of River Road LS. Timing of project will be coordinated with installation of larger pumps at new IH 35/Fairlawn LS	
Cottonwood Creek Interceptor Phase II	Abandon Cottonwood Creek Lift Station, turn 18-inch force main to gravity and extend a gravity interceptor along Cottonwood Creek to the new Southeast Wastewater Treatment Plant	Location from GIS file of the 2002–2012 City of San Marcos CIP; Ground Elevations approximated based on USGS Contours; Inverts calculated based on TCEQ Minimum Design Standards
Galisteo Ranch Wastewater Oversize	Oversize LS and force main associated with the Galisteo Ranch development to provide service beyond the subdivision	
Posey-Old Bastrop Wastewater	Install gravity wastewater line along Posey Rd. between IH 35 and Old Bastrop Highway. Install a LS at the intersection of Posey Road and Old Bastrop Highway, and a force main to bring the flow back toward IH 35	
Centerpoint Wastewater – East of Outlet Malls	City to participate in the oversize of wastewater extensions along Centerpoint Road east of the Outlet Malls to Old Bastrop Highway	
Airport Lift Station and Force Main	Construct 8-inch wastewater line along SH 21 from the proposed airport hangars to a new lift station on SH 21. Install a force main to connect to the existing lift station at the Gary Job Corps.	
Highway 21 Lift Station, Force Main and By Pass Creek Interceptor	Construct 1.5 MGD lift station near the intersection of Highway 80 and CR 101 with force main to River Road along Old Martindale Road. Construct gravity wastewater from lift station north along alignment of By Pass Creek across SH 21 in the vicinity of the River Ridge extension and continuing north to Saddlebrook and Harris Hill Road	
Oversize Old Bastrop Wastewater Main	Oversize wastewater main along Old Bastrop Highway from FM 621 toward Cottonwood Creek	
Staples Road Wastewater Oversize	Participate in oversize of wastewater gravity mains associated with development of Smith tract and others to provide minimum 10-inch main from the Smith Lift Station to properties in the sewershed extending on the south side of Staples Road	

Lift station improvements/abandonments were also made with the implementation of these CIP projects in the future systems. Table 3.5 provides a summary of the additional lift stations integrated into the system, initial year applied, and the information used to add them to the system.

TABLE 3.5. SUMMARY OF ADDITIONAL LIFT STATIONS INCORPORATED INTO FUTURE SYSTEMS

Lift Station	Year Applied to System	Information Used for Application in InfoWorks CS
San Marcos River LS	2007	Construction Plans and Factory Pump Curves
Cottonwood Creek LS	2007	Construction Plans and Factory Pump Curves
Blanco River LS	2007	Construction Plans and Factory Pump Curves
Posey Road/Old Bastrop Highway LS	2014	Location of LS from GIS Map of CIP; Approximate On/Off Elevations from USGS Maps; and Estimated Pumping Rates of LS based on Incoming Flows
Galisteo Ranch LS	2014	
Airport LS	2014	
SH 21 LS	2014	Approximate location determined through discussions with City personnel; Elevations from USGS; Pumping Rates based on Future Flows Generated

Table 3.6 summarizes the redirection of flow from lift stations that were either abandoned or intercepted with the implementation of new lift stations and/or CIP projects.

TABLE 3.6. SUMMARY OF ABANDONED/REDIRECTED LIFT STATIONS AND UPDATED FLOW DIRECTIONS IN FUTURE SYSTEMS

Lift Station Abandoned/Redirected	Year Applied to System	Flow Redirection
Bobcat Stadium LS	2007	Riverside MKT Interceptor to receive flows and convey to Wastewater Treatment Plant (WWTP) via Main LS
CFAN LS	2007	Blanco River LS to replace CFAN LS, Fairlawn Interceptor to receive flows and convey to River Road LS; River Road LS to convey flows to WWTP via Main LS
Hills of Hays LS	2007	Hills of Hays LS flow re-directed from Main LS to the Cottonwood Creek Interceptor downstream of the Cottonwood Creek LS and conveyed to WWTP via San Marcos River LS

In summary, the 2003 existing system hydraulic model has 596 pipes and 577 manholes and grows to 883 pipes and 839 manholes for the year 2027 model. The number of subcatchments increases from 226 in the 2003 model to 441 in the 2027 model. Subcatchment areas are the basis for flow generation as described in the next section.

3.4 FLOW GENERATION

3.4.1 Wastewater Component

InfoWorks CS uses subcatchments to define the flow generation characteristics of the areas from which wastewater is collected. The pipe configuration of lines 10 inches and larger in the San Marcos sanitary sewer network was evaluated and used, along with the boundaries of 26 meter basins (flow monitors and lift stations), to delineate 113 subcatchments. The boundaries of the subcatchments were determined

based on the connectivity and branching of the pipes and common points of entry to main lines of the sewer system. The areas of the subcatchments delineated in the model range from below 10 acres to more than 250 acres. Locations of the flow monitors, rain gauges, and lift stations were also taken into account in defining the subcatchments.

Two identical subcatchments were created for each area in order to separately represent the flow generation of population and employment. With two subcatchments per area, a total of 226 subcatchments were included in the model of the existing system. Given that the total acreage was used for the population and employment subcatchments, the runoff coefficients that control the I/I characteristics of the areas were halved to compensate for the additional area represented in using the two subcatchments.

The population data within the TSZs was integrated into the subcatchments using a GIS file merge in the model. In order to accurately represent the population and employment distribution of San Marcos, the TSZ boundaries were adjusted to only include populated areas (i.e., areas with wastewater connections) before merging the GIS file into the model. Aerial image files provided by the City of San Marcos were superimposed onto the TSZs to facilitate the boundary edit. Areas excluded in the revised TSZs included:

- Plots of open land and wooded areas
- Water bodies (i.e., the San Marcos River and the Blanco River)
- Major roadways, parking lots and cemeteries

The subcatchments were merged with the revised TSZs to obtain the proportionate population distributions for the subcatchments. The population distributions determined by the merge were based on the area of the TSZ falling within the perimeter of a given subcatchment. For example, if 20% of the TSZ falls in Subcatchment A, then Subcatchment A is assigned 20% of the population associated with the TSZ. The population and employment associated with the TSZs were merged separately to the appropriate subcatchments. In the case of the outer subcatchments where the TSZ boundaries extend beyond the collection system and subcatchments, the population distributions were calculated manually using a scale to approximate the percent of TSZ area represented in the outer subcatchments.

Meter basins were formed based on the location of the flow meters during the flow monitoring period. Each subcatchment was assigned to a meter basin based on location with respect to the flow meters. The diurnal flow curve developed for each meter basin from flow monitoring data was subsequently applied to all of the subcatchments in the meter basin.

3.4.2 Dry Weather Calibration

The InfoWorks Wastewater Generator (WWG) generates dry weather wastewater flows in the model at the subcatchment level using population and employment data, wastewater unit flow rates (average gallon/day per resident or employee), and dimensionless diurnal curves (ratio of flow at 15-minute

intervals to average daily flow rate). The diurnal curves are derived from flow monitoring data as discussed in Section 2, and the unit flow rates are derived in the calibration process.

The observed dry weather flow diurnal curves are stored in the model and used to calibrate the modeled dry weather flow. Working through the meter basins from upstream to downstream, the gpd/capita and gpd/employee unit flows are adjusted for each meter basin so that the diurnal flow curve generated by the model closely matches the observed flow monitoring diurnal curve stored in the model. Each meter basin's wastewater generation rates thus derived are applied to all of the subcatchments in the meter basin. The residential population generation rates were assumed to have relatively little variance throughout the collection system (approximately 80 gpd/capita), and the employment generation rates (gpd/employee) were modified to a greater extent to achieve calibration. The gpd/employee rate can vary considerably due to the range of potential commercial/industrial land use characteristics.

3.4.3 Inflow/Infiltration – Wet Weather Calibration

Using the Thiessen Polygon method, rain gauge zones were delineated and each meter basin was assigned a rain gauge based on the zones. During the flow monitoring period there were several small rainfall events, but only one event that had a substantial impact on the San Marcos collection system. The rainfall event used to calibrate the model for wet weather flows was the June 5, 2003 event.

The two model parameters used to calibrate the model to the observed wet weather event of June 5 were the fixed runoff coefficients and the routing values associated with the meter basins. The fixed runoff coefficient is a factor that governs the percentage of the rainfall volume that gets into the sewer. This parameter is used within the model by multiplying the area of the subcatchment by the inches of rainfall. The routing value is a factor that affects the peak and lag of the inflow hydrograph. This parameter governs the elapsed time before the rainfall induced inflow/infiltration enters the system.

Similar to the dry weather calibration, the wet weather calibration began with the meter basins and subcatchments furthest upstream followed by those downstream. The flow monitoring data from the June 5, 2003, rainfall event was stored in a flow survey and was superimposed onto the model-simulated flow patterns to facilitate calibration. The lift station subcatchments were assigned the wet weather parameters of the nearest downstream meter due to the fact that the June 5 storm caused a SCADA outage that resulted in a lack of real time data for the lift stations. The runoff coefficients and routing values of the meter basins were adjusted until the flow patterns simulated by the model closely matched the rainfall flow survey (observed) patterns of the June 5 event.

3.4.4 Future Development

The City of San Marcos provided a map of properties for which land development applications had been submitted. This map was used to evaluate and properly distribute the future population and employment, and acreage to the model. The increases in population across TSZs were evaluated to determine the future

acreage to be added to each subcatchment. The factors applied to the population for adding acreage to subcatchments were as follows:

- Population: 6 person increase in population = 1 additional acre
- Employment: 4 person increase in population = 1 additional acre

The future population and employment were added to the existing subcatchments using the GIS file merge method used previously. Due to the fact that acreage cannot be added to existing subcatchments in the model, supplementary subcatchments with the same input node were added to incorporate the additional future acreage to the existing subcatchments.

The future population and acreage not associated with the existing subcatchments were added to the model as new subcatchments. Since these new catchments were hypothetical, they did not have exact areas to represent their spatial orientation to the existing collection system. The new future subcatchments were assigned a point of entry to represent their contribution to the flow generated in the model. An additional flow generation profile was created to represent inflow and infiltration characteristics of the new future subcatchments. An inflow and infiltration rate of 750 gpd per acre was applied to the future flow generation profile and future population and employment generation rates of 80 gpd/capita and 60 gpd/employee, respectively, were included in the WWG for the profile.

The future population and employment were added to the existing and new subcatchments in the future systems. Table 3.7 provides a summary of the existing population, as well as the additional future populations added to the existing population for the 2007, 2014, 2017, and 2027 systems.

TABLE 3.7. SUMMARY OF ADDITIONAL RESIDENTIAL AND COMMERCIAL POPULATION INCORPORATED INTO FUTURE SYSTEMS THROUGH EXISTING AND NEW SUBCATCHMENTS

System	Total Population	Total Employment	Future Population		Future Employment	
			Added to Existing Subcatchments of 2003 Model	*Added to 2003 Model as New Subcatchments*	*Added to Existing Subcatchments of 2003 Model*	*Added to 2003 Model as New Subcatchments*
Existing	32,443	22,399				
2007	45,196	28,877	5,982	6,771	4,514	1,964
2014	64,149	43,513	12,828	18,878	13,576	7,538
2017	72,282	49,571	15,768	24,071	17,251	9,921
2027	95,943	78,189	21,316	42,184	34,845	20,945

To account for area growth in 2007, 2014, 2017, and 2027, there were additional acres integrated into the existing (2003) subcatchments (using the supplementary subcatchments with additional area and no spatial orientation) and additional future acres added as new subcatchments. Since there is only a 3-year difference in the two systems, the same area growth representation was used for 2014 and 2017. The growth areas for all of the future systems were based on the population and employment increases

calculated from the CAMPO data. As discussed previously, 1 acre was added for every six-person increase in residential population and for every four-person increase in employment. Table 3.8 summarizes the additional acreage added to each system.

TABLE 3.8. ADDITIONAL ACREAGE INCORPORATED INTO FUTURE SYSTEMS TO REPRESENT AREA GROWTH OF THE CITY OF SAN MARCOS

System Year	Additional Acreage Added to Existing (2003) Subcatchments	Future Acreage Incorporated as New Subcatchments
2007	685	1,606
2014	1,223	5,021
2017	1,223	5,021
2027	1,332	6,484

3.5 CRITERIA FOR CAPACITY EVALUATION

Two primary criteria for capacity evaluation for the existing system and for proposed improvements are the selected design storm and the allowance of any pipe surcharging for additional capacity.

Since the inflow component of peak flow is a function of rainfall intensity and duration, the design storm is a key element for capacity analysis. A 5-year 6-hour design storm is frequently used in the Texas region. However, in order to evaluate the sensitivity of capacity requirements to the design storm, the hydraulic model was run with both 2- and 5-year storms and 3-hour and 6-hour durations. The source of IDF (intensity-duration-frequency) rainfall data for the San Marcos area was the "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, U.S. Weather Bureau. Synthetic hyetographs for these four design storms were developed using the alternating block method with a 15-minute time interval (*Applied Hydrology*, Chow, Maidment and Mays). The 2-year design storm has a total volume of 3.03 inches and a peak 15-minute amount of 1.14 inches. The 5-year design storm has a total volume of 4.04 inches and a peak 15-minute amount of 1.41 inches. When these design storms were applied to the hydraulic model for the existing system, the number of projected overflows for the 5-year 6-hour storm was not significantly greater than for the 2-year 3-hour storm. Consequently, in consultation with the City it was decided to use the 5-year 6-hour storm for the capacity analysis for system improvements. The following hyetograph on Figure 3.2 represents the synthetic rainfall event utilized for system evaluation.

The surcharge criteria used in the capacity analysis of the San Marcos collection system allowed pipes to surcharge during the 5-year 6-hour design storm, but surcharged pipes causing sewer overflows were flagged for improvements. The spilling manholes and surcharged pipes were identified and improvements were made to allow surcharging, but no spilling.

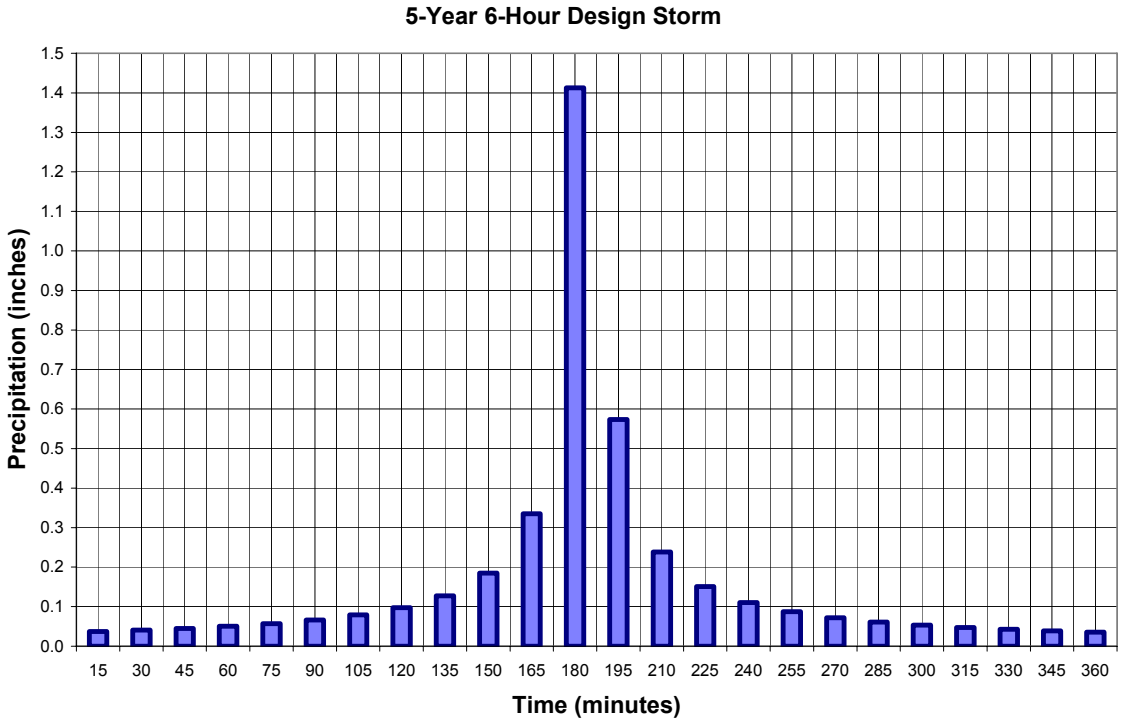


FIGURE 3.2. SYNTHETIC DESIGN STORM HYETOGRAPH

4.0 MODEL RESULTS AND IMPROVEMENTS ANALYSES

This section summarizes deficiencies identified in the modeling results for all of the scenarios analyzed - 2003, 2007, 2014, 2017, and 2027- with focus on surcharged pipes and manhole overflows. The various modeling scenarios include improvements already planned in the City's previous 2003–2014 wastewater CIP. Ten CIP projects are added for the 2007 scenario, and 14 more for the 2014 scenario. Hydraulic modeling evaluates the effectiveness and timing of the projects as planned and provides the basis for any needed modifications and additional CIP improvements as discussed in Section 5.0. The sizing for piping improvements indicated as needed through 2017 is based on design flows for the 2027 scenario to take advantage of the cost effectiveness of capacity versus pipe size.

Results of the 5-year 6-hour design storm simulations were analyzed for surcharged pipes and sanitary sewer overflows (SSOs or "spills"). Maps were generated to display the existing and future systems and the locations of SSOs occurring under peak wet weather conditions. An analysis of pipe surcharging was accomplished using the 'surcharge state' feature generated for each pipe by the modeling software. Pipes were evaluated for improvements if they had a surcharge state of 1, which indicates that surcharging is caused by downstream conditions, or 2, which indicates that the pipe does not have sufficient capacity to handle the flow. The concept of "surcharge state" is demonstrated on Figure 4.1. A surcharge state of 1 indicates that the pipe has enough open channel flow capacity to handle the flow, but downstream conditions cause the hydraulic grade line (HGL) to be above the pipe. Surcharge state 2 indicates that the pipe's capacity (size and/or slope) is not adequate to handle the incoming flow without the HGL rising above the pipe, i.e., surcharging. Figure 4.1 illustrates the surcharge states and how an overflow might be caused in surcharge state 1 by downstream conditions.

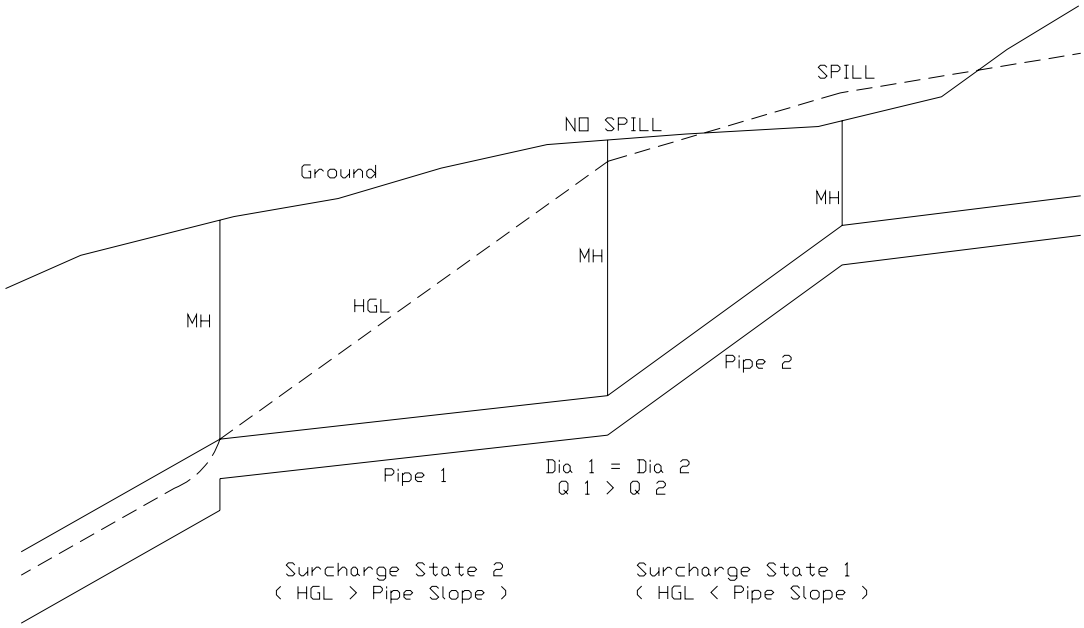


FIGURE 4.1. HYDRAULIC MODEL SURCHARGE STATES 1 AND 2

4.1 EXISTING SYSTEM

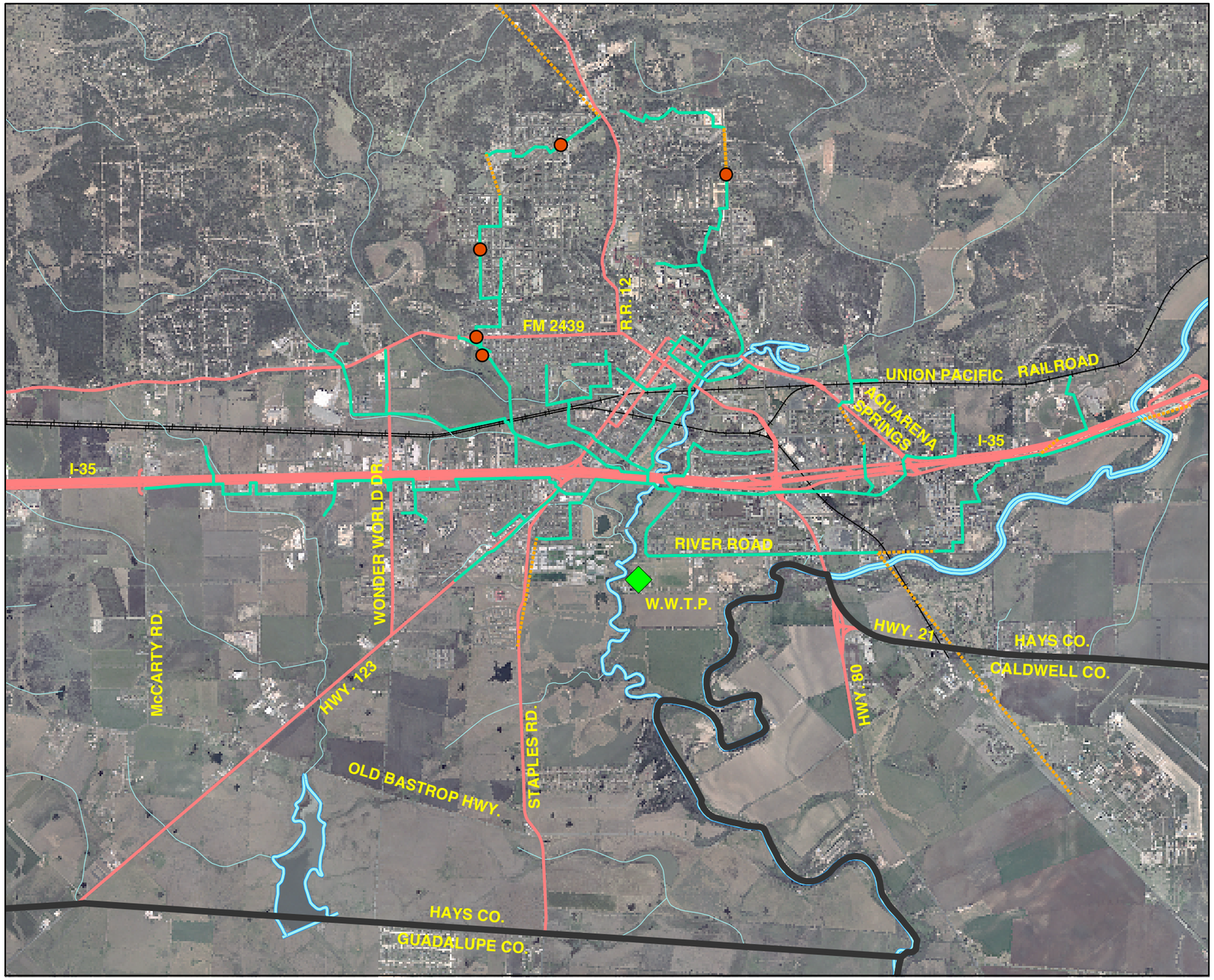
The existing system modeled refers to the system in place during the flow-monitoring period (Spring/Summer 2003). This system was used to calibrate the flows generated by the model to the flow results obtained during the monitoring period. Following the calibration of the existing system, analyses were performed to identify problem areas and evaluate consistency with the problem areas noted by the City that exist in the current collection system.

A map showing the sewer overflow locations in the model simulation of the June 5, 2003, storm conditions for the existing system is shown on Figure 4.2. The overflow results of the modeled storm event on the existing system revealed consistencies with sewer overflows reported by the City, which supported the validity of the calibrated model. The model results indicated that there were 5 instances of sewer overflows and 112 surcharged pipes, 53 of which were at a surcharge state of 2.0. A listing of overflows received from the City indicated that there was only one reported overflow (at manhole ID 185-12, Peques and Sessoms) during the June 5, 2003, rainfall event. In the model simulation of the June 5, 2003, storm event over 1,600 linear feet of the pipe directly downstream of manhole 185-12 is shown to be surcharged, and manhole 133-008 upstream of 185-12 is shown to overflow. Though the City did not report an overflow at manhole 133-008 on June 5, 2003, it did reveal a history of overflows at manhole 133-008. Another manhole that the model simulation of the June 5 event showed to be overflowing was 231-008 (Columbia and Hazelton), which the City's overflow report indicates has spilled in the past.

The response of the existing system to the 5-year-6-hour design storm is shown on Figure 4.3. This map shows the spills at maximum peak conditions during the 5-year 6-hour design storm. Again, all of these spills do not occur at one time. Over 250,000 gallons of flow was lost from 20 manholes and 3 lift stations in the design storm. The areas with the highest volumes of flow loss were from a 12-inch wastewater line near the Roosevelt Street subdivision (flow lost from 3 manholes), a 12-inch line near the intersection of Bishop and Hopkins Streets (flow lost from 2 manholes), a 10-inch line through the Rio Vista subdivision (flow lost from 2 manholes), and an 8-inch line receiving flow from the North LBJ lift station (LS 11) (flow lost from 1 manhole). Surcharging lines lacking the capacity needed to handle incoming flows caused the manhole spills on the lines near Roosevelt Street, the Rio Vista subdivision, and the North LBJ lift station. The manholes losing flow on the 12-inch line near Bishop and Hopkins were spilling due to downstream surcharged lines (downstream constraint).

Identified in the following are other areas where flow volume was lost in the system with existing conditions and the proposed cause of the flow loss:

- At the Old Plant lift station (LS 3) and three manholes that lost flow from a 10-inch line upstream of the Old Plant lift station; flow was lost due to inadequate pumping capacity at the station.



Legend

- SEWER OVERFLOW LOCATIONS
- EXISTING W.W.T.P.
- EXISTING WASTEWATER SYSTEM (F.M.)
- EXISTING WASTEWATER SYSTEM (WWL)
- COUNTY LINE
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

PBS&J
6504 Bridge Point Pkwy, Ste. 200
Austin, Texas 78730
Phone: (512) 329-8342 Fax: (512) 327-2453

FIGURE 4.2

EXISTING SYSTEM: CONDITIONS UNDER JUNE 5, 2003 STORM EVENT

Prepared for: CITY OF SAN MARCOS	
Job No.: 441201.00	Scale: 1"=4000'
Prepared by: S.MULLER	Date: 10/08/05
File: K:/Hi1/441201/Docs/Report/Revised Maps 07_2005/shp/SM-SP-01	

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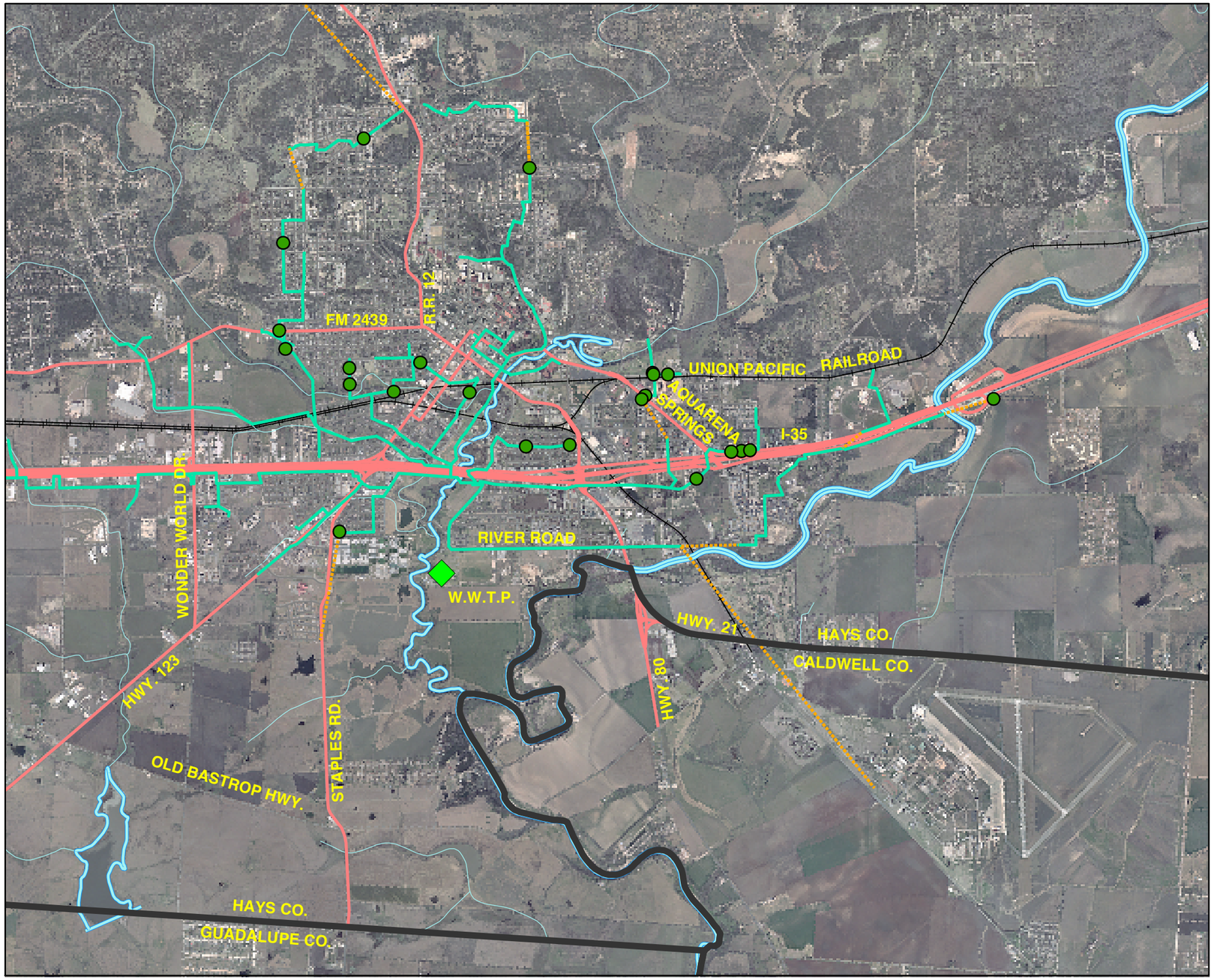

Legend

- EXISTING W.W.T.P.
- SEWER OVERFLOW LOCATIONS
- EXISTING WASTEWATER SYSTEM (F.M.)
- EXISTING WASTEWATER SYSTEM (WWL)
- COUNTY LINE
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

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6504 Bridge Point Pkwy, Ste. 200
Austin, Texas 78730
Phone: (512) 329-8342 Fax: (512) 327-2453

FIGURE 4.3
EXISTING SYSTEM: 5 YEARS 6 HOUR DESIGN STORM CONDITIONS

Prepared for: CITY OF SAN MARCOS	
Job No.: 441201.00	Scale: 1"=4000'
Prepared by: S.MULLER	Date: 10/08/05
File: K:/Hi1/441201/Docs/Report/Revised Maps 07_2005/shp/SM-SP-02	

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- At the Bobcat Stadium lift station and three manholes along a 15-inch line and one manhole on a 12-inch line, all upstream of the Bobcat lift station; spills were caused by inadequate pumping capacity at Bobcat lift station and low manholes along the 15-inch line.

- At the CFAN lift station; due to lack of sufficient pumping capacity.

- Two manholes along 10- and 12-inch lines near MLK Drive/CM Allen Parkway/McGehee Street; flow lost due to insufficient capacity in the 10- and 12-inch lines and in the downstream lines (downstream constraint).

- One manhole along a 12-inch line in the area of Bishop and Belvin; flow lost due to surcharging in lines downstream.

- One manhole along a 6-inch line near Craddock Avenue; flow lost due to inadequate capacity in the line.

- One manhole along a 6-inch line receiving flow from the Hills of Hays lift station (LS 28); flow lost due to lack of capacity in the line.

4.2 FUTURE SYSTEMS

The collection systems modeled for the future were modified from existing conditions to reflect the projected future conditions of the City of San Marcos, including sewer system improvements, population growth and residential and commercial developments. Sewer system improvements were made to the future systems according to the CIP issued by the City of San Marcos for the fiscal years 2003–2014. The detailed improvement projects added to the existing system are outlined in Section 3.2. For modeling purposes, the backup lag pumps present in the existing system were removed from the future systems leaving only the lead pumps to convey flow at all of the lift stations. This is in accordance with the TCEQ requirement for lift stations to have sufficient capacity with the largest pump out of service. In order to account for the increased flows associated with future growth of the City of San Marcos, population and land area were added to the future systems. The procedures used to add future population and land area were discussed previously in Section 3.3 of this report. The following sections discuss the responses of the 2007, 2014, 2017, and 2027 systems to the 5-year 6-hour design storm.

4.2.1 Design Storm – 2007 System Response

The modeling results of the design storm application to the 2007 conditions yielded a total flow loss of over 500,000 gallons from 26 manholes and 2 lift stations. There were a total of 203 surcharged pipes, 109 of which were at a surcharge state of 1.0 and 94 were at a surcharge state of 2.0. The areas with the highest volumes of flow loss were from surcharged lines just upstream of the Old Plant lift station (flow lost from the lift station and 8 manholes) and from 12-inch wastewater lines near the Roosevelt Street subdivision (flow lost from 3 manholes). The surcharged lines upstream of the Old Plant lift station were caused by inadequate pumping capacity at the station and lack of capacity in the pipes. The Roosevelt Street spills were due to surcharged lines lacking the capacity needed to handle incoming flows.

Shown on Figure 4.4 is the 2007 system and the spills associated with the system's response to the design storm. The 2007 system response was similar to the response of the system's existing conditions with a few exceptions. As noted in the previous Section 3.0, there were ten improvement projects from the City's 2003–2014 CIP added to the system between the existing and 2007 scenarios. These projects relieved some of the spilling manholes and lift stations that occurred under existing conditions. The Craddock Avenue Improvements eliminated spills occurring in lines that lost flow under existing conditions and the addition of the Riverside MKT Interceptor relieved some spilling occurring at the manhole directly upstream of and at the Bobcat Stadium lift station. The Riverside MKT also eliminated one of two spilling manholes on the line through the Rio Vista subdivision. The addition of the Cottonwood Creek Interceptor re-directed flow from the Hills of Hays lift station and eliminated the spill in the 6-inch line that previously received flow from the lift station. The spill at the CFAN lift station was relieved under the 2007 conditions with the addition of the IH 35/Fairlawn interceptor and Blanco River lift station.

Even with the improvements/additional projects, some of the spills occurring under existing conditions still lose flow in the 2007 system. The following are the spills in the 2007 system that were consistent with the spills occurring under existing conditions and additional spills in the same areas occurring in 2007:

- At the Old Plant lift station (LS 3) and three manholes that lost flow from a 10-inch line upstream of the Old Plant lift station; flow was lost due to the lack of pumping capacity at the station. Five additional manholes on 6-, 8-, 10-, and 12-inch lines upstream of the Old Plant lift station lost flow under 2007 conditions due to lack of capacity in the pipes and inadequate pumping capacity at the Old Plant lift station.

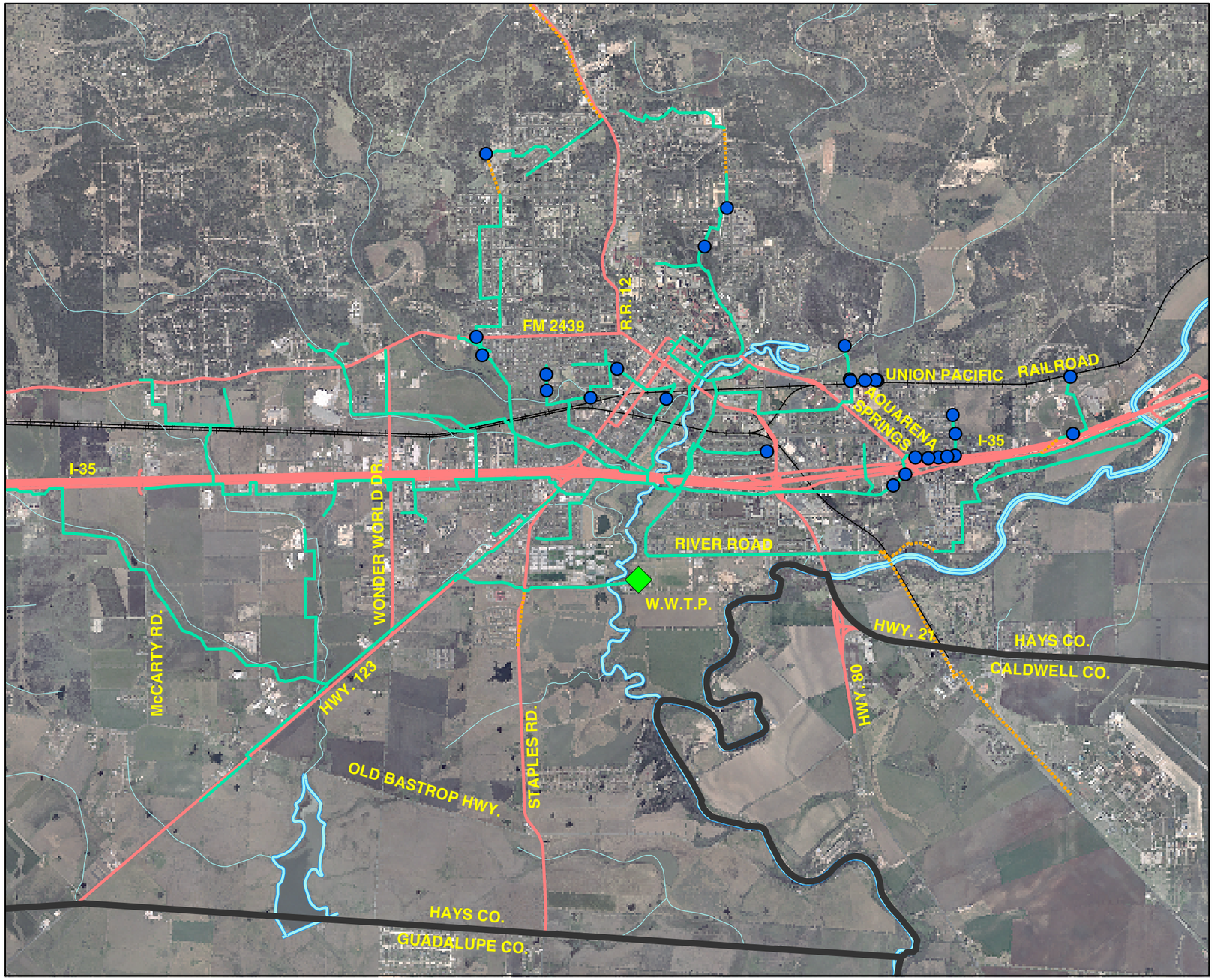
- Two manholes along a 15-inch line and one manhole on a 12-inch line, all upstream of the Bobcat Stadium lift station; spills were caused by low manholes along the 15-inch line and lack of capacity in the lines. One additional manhole on an 8-inch line upstream of the 15-inch and two additional manholes on the 12-inch line lost flow under 2007 conditions due to lack of capacity in the pipes downstream.

- Two manholes along a 12-inch line near the intersection of Bishop and Hopkins Streets; due to resulting surcharging from a downstream constraint.

With the added flow resulting from the increase in population and land area served under the 2007 conditions, there were additional areas where volume was lost from the system. The following is a listing of the spills occurring in additional areas under the 2007 conditions and the proposed cause of the flow loss:

- Two manholes along a 6- and 8-inch line in the area of Sessoms and Loop 82; due to lack of capacity in the pipes.

- Two manholes upstream of the River Ridge lift station; due to inadequate pumping capacity at the River Ridge lift station.



Legend

- EXISTING W.W.T.P.
- SEWER OVERFLOW LOCATIONS
- 2007 WASTEWATER SYSTEM (WWL)
- 2007 WASTEWATER SYSTEM (F.M.)
- COUNTY LINE
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

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FIGURE 4.4

2007 SYSTEM: 5 YEAR 6 HOUR DESIGN STORM CONDITIONS

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- At the Franklin Square lift station; due to inadequate pumping capacity at the Franklin Square lift station and downstream capacity issues.
- Three manholes along 12-inch lines near Roosevelt Street; due to lack of pipe capacity.
- One manhole along a 10-inch line through the Rio Vista subdivision; due to lack of capacity in the line.
- Two manholes along 10- and 12-inch lines near MLK Drive/CM Allen Parkway/McGehee Street; flow lost due to lack of capacity in the 10- and 12-inch lines and in the downstream lines.

4.2.2 Design Storm– 2014 System Response

The application of the 5-year-6-hour design storm to the 2014 system resulted in a flow volume loss of over 475,000 gallons from a total of 19 manholes and two lift stations. There were 80 pipes at a surcharge state of 1.0 and 74 pipes at a surcharge state of 2.0, for a total of 154 surcharged pipes. As under 2007 conditions, the areas losing the most flow were lines just upstream of the Old Plant lift station and lines near Roosevelt Street. The points of flow loss in the 2014 system were consistent with flow loss points under 2007 conditions for the Old Plant lift station and the Roosevelt Street areas; these points included the flow lost at the Old Plant lift station and eight manholes upstream and three manholes in the Roosevelt Street area.

Figure 4.5 illustrates the 2014/2017 system and spills occurring with the system's response to the design storm. As outlined in the previous Section 3.2, there were 14 projects (in addition to the 10 projects added to the 2007 system) added to the collection system for the 2014 conditions. These projects relieved some of the spills seen in the previous 2007 system response to the design storm, but not all. Areas relieved by improvements included upstream of the Bobcat Stadium Lift Station, lines near Sessoms/Loop 82, and lines near the intersection of Bishop and Hopkins Streets. There were six spilling manholes upstream of the Bobcat Stadium Lift Station in the 2007 system. With the installation of the Sink Creek Interceptor Phase II in the 2014 system, the count of manholes losing flow was cut in half with only three manholes spilling upstream of the Bobcat Stadium lift station. The Sink Creek Interceptor Phase II also relieved the two manholes in the lines of Sessoms/Loop 82 that were losing flow in the 2007 system. The Sessoms/Loop 82 line was relieved in the 2014 system by the redirection of flow from the North LBJ lift station to the Phase II Sink Creek Interceptor. The manholes located in the line near the intersection of Bishop and Hopkins Streets that were losing flow in the 2007 system were relieved by the Purgatory Creek Interceptor in the 2014 system.

There were areas of flow loss in the system response to the design storm with the 2014 conditions that were consistent with the 2007 conditions. The flow lost in the 2014 system, compared to the 2007 system,

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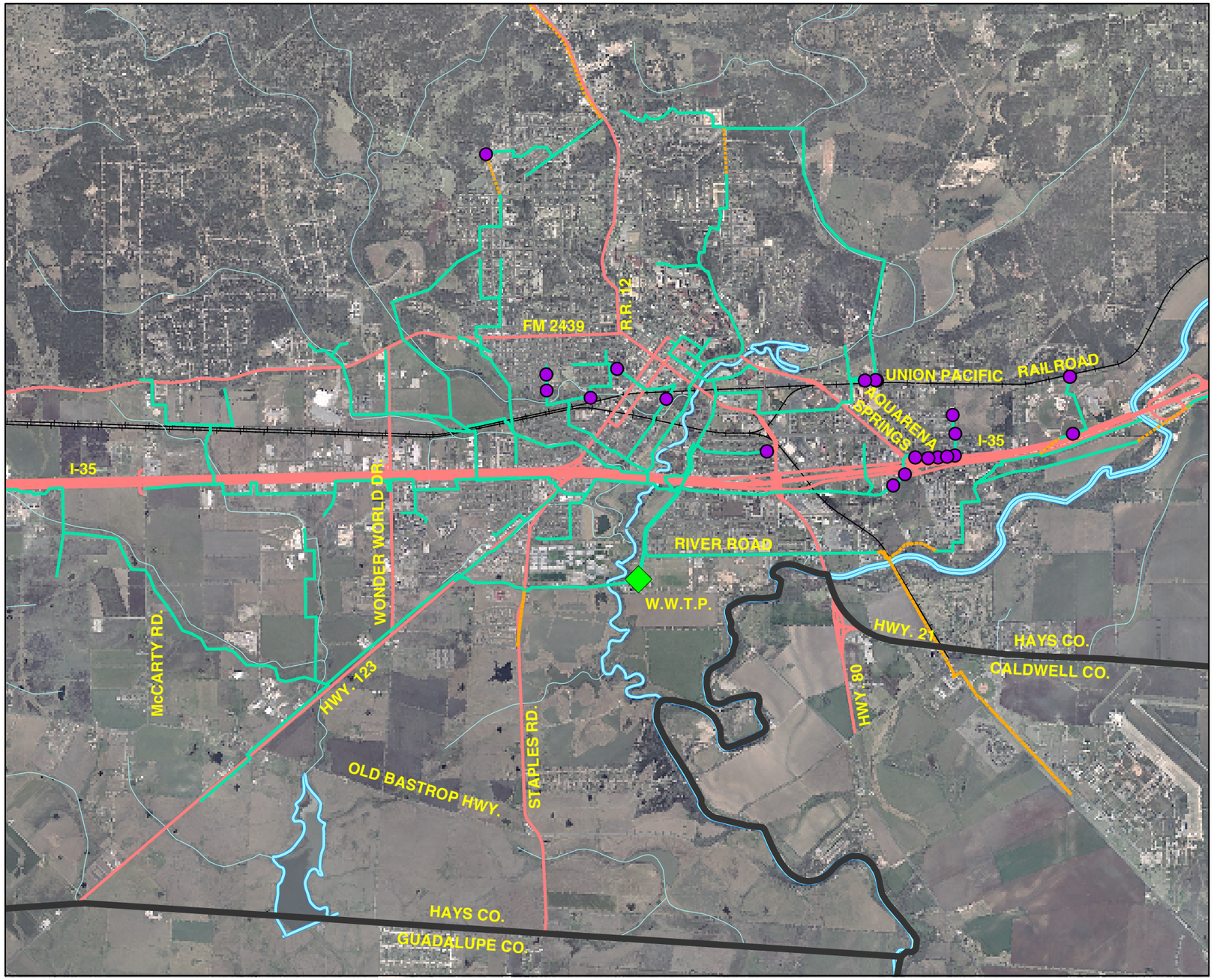

Legend

- EXISTING W.W.T.P.
- SEWER OVERFLOW LOCATIONS
- 2014 WASTEWATER SYSTEM (F.M.)
- 2014 WASTEWATER SYSTEM (WWL)
- COUNTY LINE
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

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FIGURE 4.5

2014 & 2017 SYSTEMS WITH COSM CIP: 5 YEAR 6 HOUR DESIGN STORM CONDITIONS

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was higher in most of these areas due to the increased flows resulting from population growth and land development. There were no areas with increased amounts of spilling locations, as seen in the transition from the existing conditions to the 2007 conditions, but as mentioned in the above paragraphs, there were areas where spills in the 2007 system were relieved by the capital improvements projects added to the 2014 system. The following are the points of flow loss (manholes and lift stations) in the 2014 system that were consistent with spills occurring under 2007 conditions and the proposed causes of flow loss, also listed are manholes that were relieved by projects added to the 2014 system:

- At the Old Plant lift station (LS 3) and three manholes that lost flow from a 10-inch line and five manholes on 6-, 8-, 10-, and 12-inch lines upstream of the Old Plant lift station; flow was lost due to the lack of pumping capacity at the station and lack of capacity in the pipes upstream of the lift station.

- Three manholes along 12-inch lines near Roosevelt Street; due to lack of pipe capacity.

- One manhole along a 10-inch line through the Rio Vista subdivision; due to lack of capacity in the line.

- Three manholes on a 12-inch line upstream of the Bobcat Stadium lift station lost flow due to lack of capacity in the pipes downstream. Two manholes along a 15-inch line and a manhole on an 8-inch line upstream of the 15-inch, all upstream of the Bobcat Stadium lift station, were relieved (and not spilling under 2014 conditions) by the installation of Phase II of the Sink Creek Interceptor.

- Two manholes along 10- and 12-inch lines near MLK Drive/CM Allen Parkway/McGehee Street; flow lost due to lack of capacity in the 10- and 12-inch lines and in the downstream lines.

- Two manholes along a 6- and 8-inch line in the area of Sessoms/Loop 82; relieved and not spilling under 2014 conditions with the addition of the Phase II Sink Creek Interceptor.

- Two manholes upstream of the River Ridge lift station; due to inadequate pumping capacity at the River Ridge lift station.

- At the Franklin Square lift station; due to inadequate pumping capacity at the Franklin Square lift station and downstream capacity issues.

4.2.3 Design Storm – 2017 System Response

The system response to the 5-year 6-hour design storm with the 2017 conditions yielded a flow loss of over 490,000 gallons from 19 manholes and 2 lift stations. A total of 168 pipes were shown to surcharge. Of these pipes, 85 lines surcharged to a surcharge state of 1.0 and 83 lines reached the 2.0 surcharge state. The highest volumes of flow loss occurred in the same areas as under the 2007 and 2014 conditions; upstream of the Old Plant lift station and in lines near Roosevelt Street. The causes of flow loss were also

the same; a lack in pumping capacity at the Old Plant lift station and a lack of capacity in the pipes upstream of the lift station and in the lines near Roosevelt Street.

There were no additional projects added to the system in the transition from the 2014 system to the 2017 system. The points of flow loss in the 2017 system were the exact same as the spills occurring in the 2014 system, as shown on Figure 4.5 and detailed in Section 4.2.2. Due to an increase in flow resulting from added population and land area, the volumes of flow lost from these spills increased in the 2017 system.

4.2.4 Design Storm – 2027 System Response

Over 500,000 gallons of flow was lost from 20 manholes and two lift stations in the 2027 system response to the 5-year 6-hour design storm. There were 181 pipes surcharged, 85 lines reached a surcharge state of 1.0 and 96 lines reached the 2.0 surcharge state. The locations of flow loss under the 2027 conditions were identical to the flow loss locations in the 2014 and 2017. The locations of the highest volumes of flow loss under 2027 conditions were also identical to the 2014 and 2017 conditions; at and upstream of the Old Plant lift station and pipes near Roosevelt Street. The flow lost under 2027 conditions at and upstream of the Old Plant lift station and from the lines in Roosevelt Street reached over 275,000 gallons (over half of the total volume lost from the system).

No additional projects were included in the transition from the 2017 system to the 2027 system. The manholes and lift stations losing flow were the same under 2027 conditions as they were under 2014 and 2017 conditions as detailed in Section 4.2.2, with one addition. Figure 4.6 illustrates the overflow locations associated with the 2027 system. The additional spill was located on the 15-inch line upstream of the Bobcat Stadium lift station. The spill was caused by a lack of pipe capacity downstream and low-lying manholes in the area of the spill.

4.3 IMPROVEMENTS ANALYSIS

Over the duration of the 5-year 6-hour design storm, there were 28 sewer overflows under the future 2007 conditions, 20 under 2014 conditions, 20 under 2017 conditions, and 21 under 2027 conditions. It is important to note that the maximum peak conditions referred to here do not occur at a single point in time, thus these overflows did not all occur simultaneously during the design storm. The overflows observed at the maximum peak conditions of the 2014, 2017, and 2027 systems all resulted in the same general areas of the system. Although the overflow locations were the same, the volume lost in the overflows increased with time.

After consulting with the City, it was decided that improvements analyses would be performed only on the 2027 system. The system was evaluated at maximum peak conditions using the criteria described in Section 3.4 of this report. These criteria mainly focused on the assessment of overflows to determine the cause of the spill. Downstream conditions, lack of pipe capacity, and lift station back ups were common problems resulting in overflows.

The methods used in fixing the overflows varied from case to case. To fix the downstream constraint and pipe capacity issues, the hydraulic grade line (HGL) was compared to the slope of the pipes. If the HGL was steeper that the pipe slope, then that pipe segment was upsized. Where the HGL could not be used as a tool to determine pipe improvements, Manning's equation was applied to determine the necessary capacity based on the maximum upstream flow.

The lift stations in all future systems were analyzed using only the pumping capacity of the lead pumps. The lag pumps were modeled for analysis of the existing conditions, but were removed from the model for analysis of improvements for the 2007, 2014, 2017, and 2027 systems per TCEQ requirements for backup capacity. Lift station improvements were made in the system where overflows were caused by insufficient pumping capacity. Lift station back ups were resolved by increasing the pumping capacity to keep up with the incoming flows.

Modifications and additions were applied to the City CIP projects as needed to eliminate spills occurring in the improved areas. These project modifications, as well as the recommended system improvements, are included in the CIP outlined in Section 5.0 of this report.

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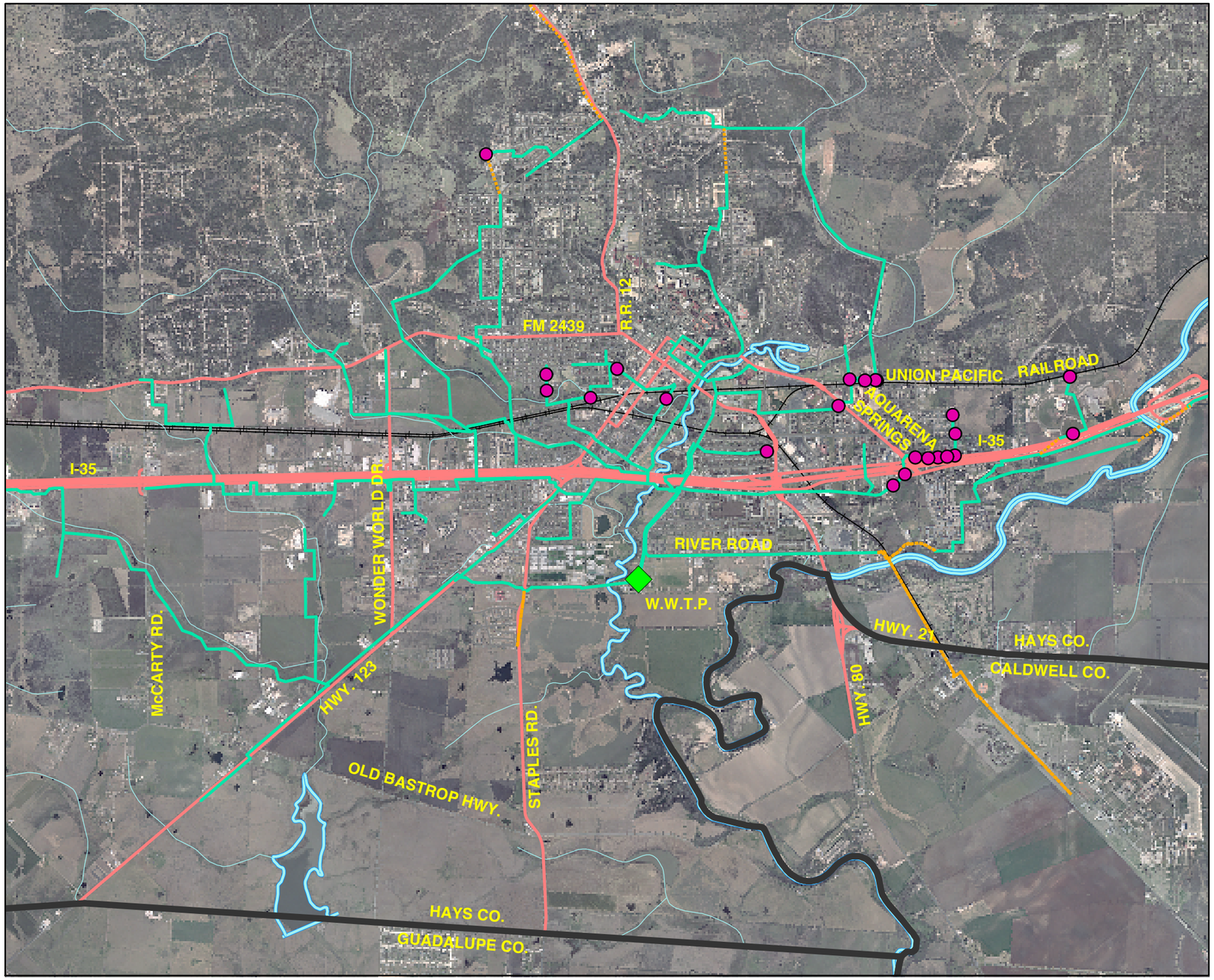

Legend

- EXISTING W.W.T.P.
- SEWER OVERFLOW LOCATIONS
- 2014 WASTEWATER SYSTEM (F.M.)
- 2014 WASTEWATER SYSTEM (WWL)
- COUNTY LINE
- HIGHWAYS
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- WATERWAYS
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FIGURE 4.6

2027 SYSTEMS WITH COSM CIP:
5 YEAR 6 HOUR DESIGN STORM CONDITIONS

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5.0 CIP DEVELOPMENT

The modeling results provided the information necessary to identify future problem areas as growth occurs in the San Marcos collection system and, in turn, to develop a CIP tailored to preventing sewer overflows in these problem areas. The CIP developed in this section is based on the modeling results and the CIP included in the 2002-2012 San Marcos Water and Wastewater Impact Fee Analysis report prepared by City staff. The improvement projects listed in the 2002–2012 CIP were included and reviewed in the future sewer models, and revisions to these projects were made as necessary. These planned projects, as well as additional system improvement projects, have been prioritized, and a CIP has been developed to provide sewer capacity and support future growth through the year 2014. Improvements are sized to provide capacity for projected 2027 flows. Projects are scheduled through year 2014 for purposes of developing applicable costs for the wastewater impact fee (10 year timeframe) and because the timing and distribution of growth beyond 10 years has higher uncertainty.

The CIP developed in this section reflects the needs of the collection system regardless of how flows will be diverted for treatment in the future. The two main treatment alternatives discussed in Section 6.0 involve the expansion of the existing treatment facilities to treat all future flow or construction of a new treatment facility in the Cottonwood Creek watershed. Each of these treatment alternatives has a unique diversion plan that involves additional projects to the CIP outlined in this section. The projects included in the CIP presented in this section are required to provide sufficient capacity to convey future flows through the collection system.

The intentions of the CIP projects developed in this study were to provide relief to problem areas identified in the model of the existing system by incorporating additional sewer capacity. Construction has commenced and/or been completed on CIP projects listed in Table 5.1 since the existing collection system was evaluated for this study. These projects were incorporated in the models of the future systems with beneficial results. The Craddock Avenue Improvements provided additional capacity and relief to surcharging lines upstream of the Franklin Square lift station. The Sessoms/Loop 82 Improvements aided a recurring sewer overflow issue in the area of the intersection of Sessoms Drive and Loop 82. The Cottonwood Creek Interceptor intercepted flows from the Outlet Malls and provided additional sewer capacity in the southeastern portion of the service area. The San Marcos River lift station provided a means of transferring the flows from Cottonwood Creek Interceptor and other lines on the south side of the river over to the wastewater treatment facility on the north side of the river. The IH 35/Fairlawn interceptor, force main and Blanco River lift station project provides for relief of the CFAN lift station, which was found to overflow in the model simulation of design storm conditions on the existing collection system. The North IH 35 Interceptor relieves an existing 8-inch pipe and increases sewer capacity for future development north of the Blanco River. The implementation of the 12-inch wastewater line near McCarty Lane established sewer service in an area in southwest San Marcos experiencing residential growth.

TABLE 5.1. CIP PROJECTS COMPLETED/UNDER CONSTRUCTION THROUGH 2003 AND 2004

CIP Projects Under/Completed Construction (2003-2004)
Craddock Avenue Improvements (8-inch Lines Installed)
Sessoms/Loop 82 Improvements (18- and 20-inch Lines Installed)
North IH 35 Interceptor (18- and 24-inch)
IH 35/Fairlawn Interceptor (30- and 36-inch)/Blanco River LS
Cottonwood Creek Interceptors/Cottonwood Creek LS/San Marcos River LS
San Marcos River LS
McCarty Lane 12-inch Wastewater Line
Centerpoint Road Wastewater Line
Cottonwood Creek Subdivision Wastewater Line

The prioritized list of CIP projects in Table 5.2 includes projects listed in the 2002–2012 San Marcos Water and Wastewater Impact Fee Analysis report, as well as necessary improvements projects identified by the modeling results. Figure 5.1 illustrates the locations of the prioritized CIP projects required regardless of treatment alternative chosen (treatment alternatives presented in Section 6.0), as well as the completed/under construction projects listed in Table 5.1. The simulation results of the modeled design storm on the existing system show surcharged lines and sewer overflows in the areas of all CIP projects listed in Table 5.2. The priority order of the projects is based on the severity of the pipe surcharging and sewer overflows, as well as the importance of the project with regard to timing and rerouting of flows. These projects are presented in greater detail in Figures 5.2, 5.3, 5.4, and 5.5.

TABLE 5.2. LIST OF PRIORITIZED CIP AND IMPROVEMENT PROJECTS NECESSARY TO FIX CAPACITY ISSUES IDENTIFIED BY THE CITY AND THE RESULTS OF THE EXISTING SYSTEM MODEL

Priority Rank	Proposed Year of Construction	Project Title	Project Description
1	2005–2006	Modified Riverside MKT Interceptor	Identified in the City CIP: Install 27- and 30-inch lines to relieve Bobcat Lift Station (LS 6), which will relieve flow from and increase capacity in the Linda Drive WWL for development south of Uhland Road
2	2005–2006	Sanitary Sewer Evaluation Survey (SSES) of selected Drainage Basins	Identified in the City CIP and Flow Monitoring Study: Conduct SSES, including smoke testing, dye water testing, cleaning and CCTV, on all sewer lines in selected basins
3	2005–2006	Sessoms/Loop 82	Identified in the model: Pipe burst 3 line segments to relieve blockages caused by insufficient capacity
4	2005–2006	Sink Creek Phase I	Identified in the City CIP: 27-inch Upsize of 15-inch WWL Upstream of Bobcat LS. This upsize will relieve spills identified in the model
5	2006–2007	Rio Vista Wastewater Improvements	Identified in the City CIP as a Rehab Project, but replacement of the 10-inch WWL with 12-inch is suggested to eliminate two upstream manhole spills identified in the model

Priority Rank	Proposed Year of Construction	Project Title	Project Description
6	2006–2007	Old Plant Lift Station Improvements	Identified in the model: Increase pumping capacity at Old Plant LS (LS 3) from 0.5 MGD to 1.0 MGD. Replace 6-, 8-, 10- and 12-inch pipe upstream of LS 3 with larger pipes to relieve manhole spills occurring in the existing system identified in the model
7	2006–2007	Modified Roosevelt Wastewater Improvements	Identified in the City CIP modified with model results: Replace 12-inch WW Lines with 18-inch to eliminate manhole spills. Additions to the planned Roosevelt Improvements eliminate manhole spills identified in the model
8	2007–2008	Rehab/Replacement for I/I Control	Repair and replace defects identified through SSES
9	2007–2008	Eastside Interceptor Phase III	Identified in the City CIP: Construct 24-, 30-, and 33-inch WWL to connect existing 18-inch WWL and future Purgatory Creek Interceptor to the Eastside II Interceptor. This line along with the Purgatory Creek Interceptor will reroute flow to eliminate spills in the existing system caused by downstream capacity constraints
10	2007–2008	Purgatory Creek Interceptor	Identified in the City CIP: Construct 21- and 24-inch WWL to relieve flow from lines in the Bishop Belvin area. This line will connect to the Eastside III Interceptor and will eliminate spills that were identified in the model as occurring in the existing system due to downstream capacity constraints
11	2009–2010	Rehab/Replacement for I/I Control	Repair and replace defects identified through SSES
12	2009–2010	MLK/CM Allen/ McGehee Improvements	Identified in the model: Upsize lines to 12-, 15-, 18-inch to provide capacity necessary to handle incoming flow. Needed to relieve spills occurring at design storm conditions in the existing system according to the model
13	2010–2011	River Ridge Lift Station	Identified in the model: Upsize River Ridge Lift Station from 0.25 MGD to 0.60 MGD. Needed to relieve future spills according to the model
14	2011–2012	Rehab/Replacement for I/I Control	Repair and replace defects identified through SSES.
15	2012–2013	Modified Sink Creek Phase II	Identified in the City CIP and modified with model results: Construct 18- and 21-inch line from Sink Creek Phase I along Sink Creek to relieve lift stations at N. LBJ & Holland, N. LBJ & Oak Ridge, and N. LBJ & Craddock. Incorporate an additional 12-inch line connecting the Sink Creek Interceptor to 12-inch line along Post Road Date of construction for this project is dependent on future development in this area

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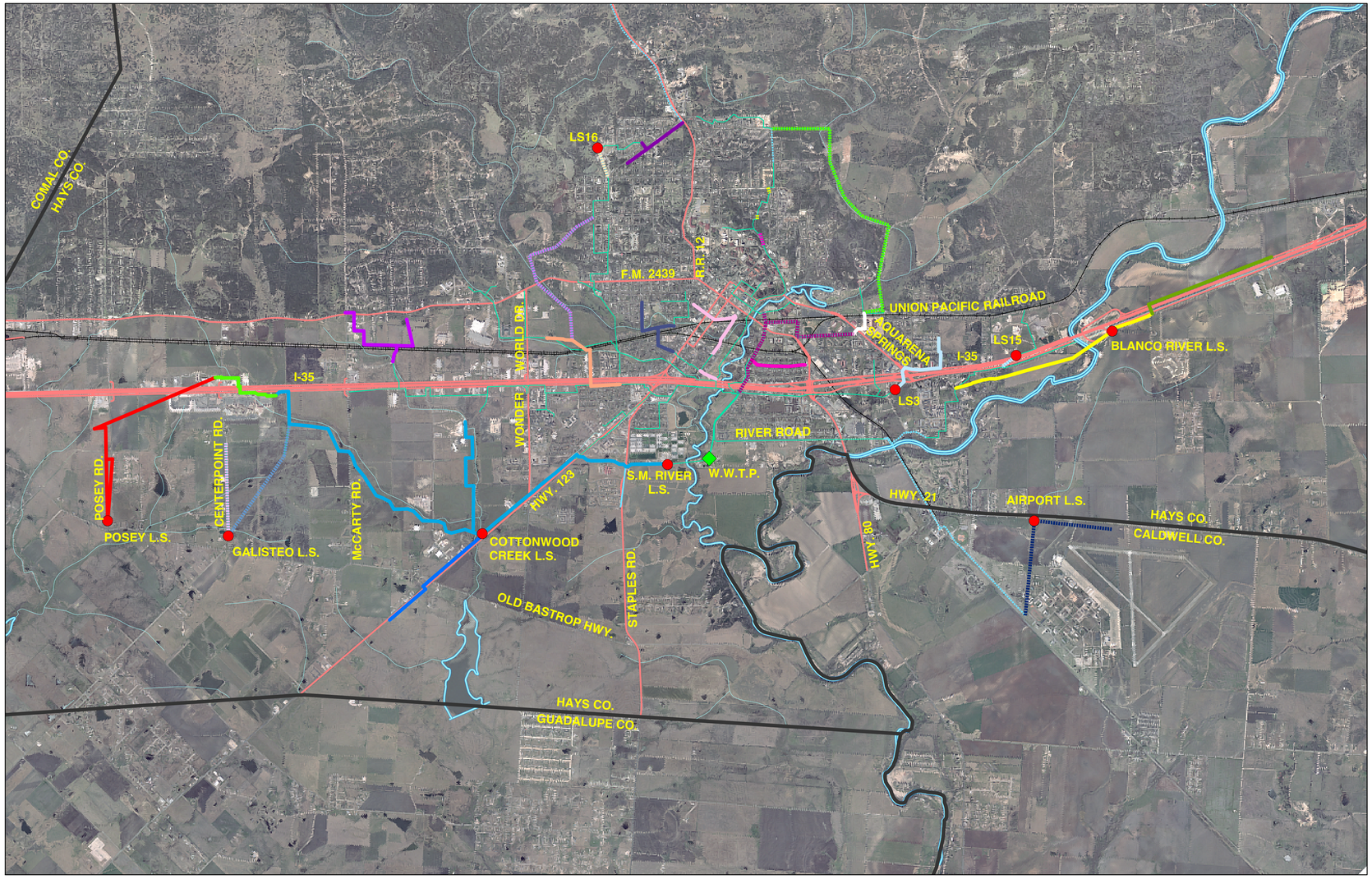

Legend

- EXISTING W.W.T.P.
- LIFT STATIONS
- COMPLETED: CENTERPOINT RD. IMPROVEMENTS
- COMPLETED: CRADDOCK AVE. IMPROVEMENTS
- COMPLETED: McCARTY LANE WASTEWATER LINE
- COMPLETED: COTTONWOOD CREEK SUBDIVISION WWL
- COMPLETED: COTTONWOOD CREEK SYSTEM (WWL & 2 L.S.)
- COMPLETED SESSOMS/ LOOP 82
- COMPLETED: NORTH I-35 INTERCEPTOR
- COMPLETED: I-35 FAIRLAWN/ BLANCO RIVER L.S.
- F.M. UNDER INVESTIGATION LS-16
- 2005-2006: MODIFIED RIVERSIDE MKT INTERCEPTOR
- 2005-2006: SESSOMS/ LOOP 82
- 2005-2006: SINK CREEK PH-1
- 2006-2007: RIO VISTA WW IMPROVEMENTS
- 2006-2007: OLD PLANT L.S. IMPROVEMENTS
- 2006-2007: MODIFIED ROOSEVELT WW IMPROVEMENTS
- 2007-2008: EASTSIDE INTERCEPTOR PH-3
- 2007-2008: PURGATORY CREEK INTERCEPTOR
- 2009-2010: MLK/CM ALLEN/MCGEHEE IMPROVEMENTS
- 2010-2011: RIVER RIDGE LS-15
- 2012-2013: MODIFIED SINK CREEK PH-2
- FUTURE: POSEY RD. - OLD BASTROP HWY. (WWL/ F.M./ L.S.)
- FUTURE: GALISTEO RANCH OVERSIZE
- FUTURE: AIRPORT L.S., F.M. & WWL
- FUTURE: CENTERPOINT WW EAST OF OUTLET MALLS
- FUTURE WASTEWATER SYSTEM (F.M.)
- FUTURE WASTEWATER SYSTEM (WWL)
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS
- COUNTY LINE

NOTE:

THE PROJECTS SHOWN ARE INDEPENDENT OF ALTERNATIVES FOR ADDITION OF FUTURE WASTEWATER TREATMENT CAPACITY. SEE FIGURES 6.1, 6.2, & 6.3 FOR CIP PROJECTS ASSOCIATED WITH ALTERNATIVES 1, 2, & 2A FOR FUTURE TREATMENT CAPACITY.

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FIGURE 5.1
CIP PROJECTS: PRIORITIZED, CONSTRUCTED/ UNDER CONSTRUCTION & FUTURE

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Prepared by: S. MULLER	Date: 9-06-05
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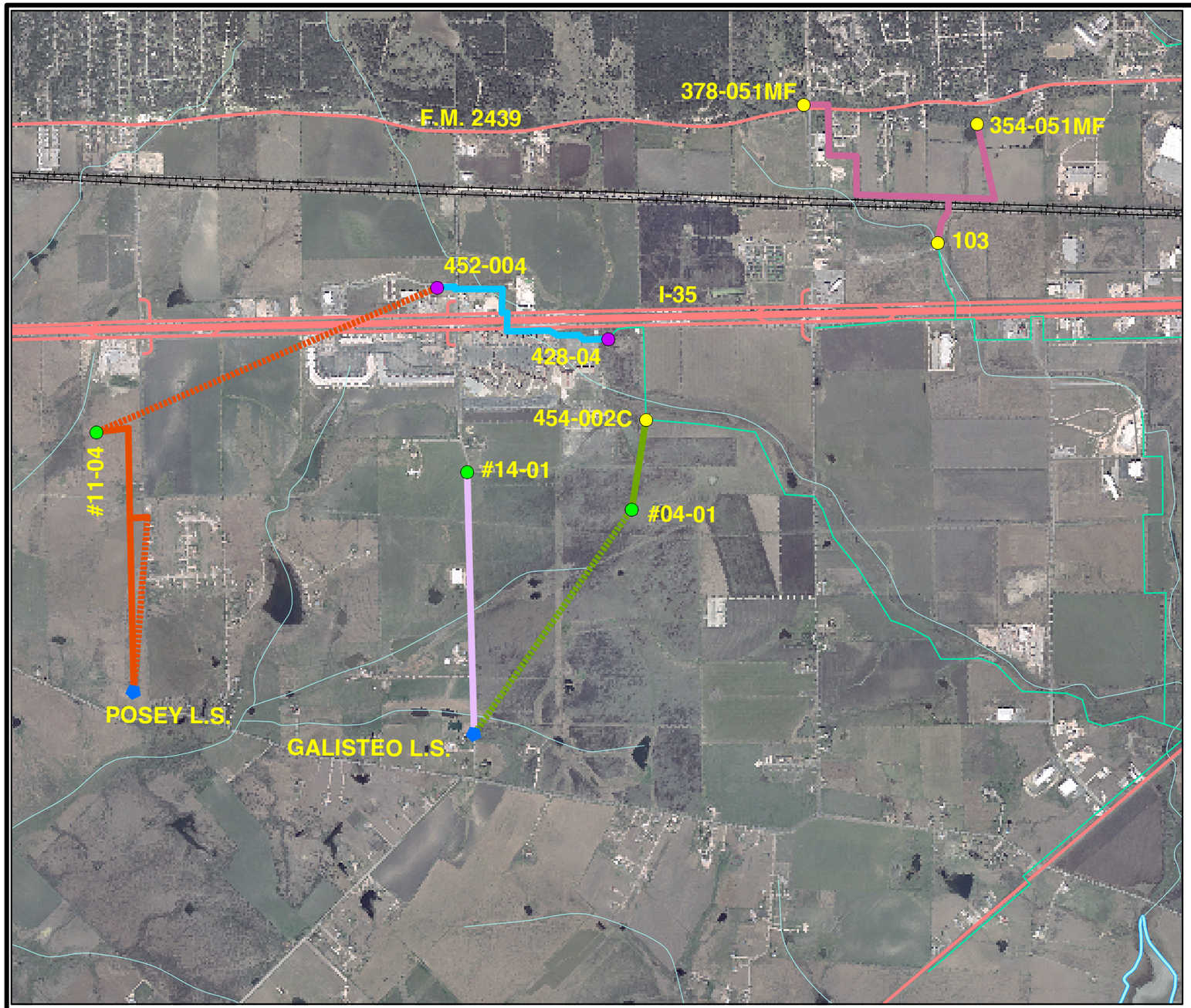

Legend

- EXISTING MANHOLES (MODEL)
- EXISTING MANHOLES (GIS)
- CIP MANHOLES (FUTURE)
- LIFT STATIONS
- COMPLETED: CENTERPOINT 2003
- COMPLETED: McCARTY LANE WASTEWATER LINE
- FUTURE GALISTEO RANCH OVERSIZE
- FUTURE POSEY RD. - OLD BASTROP HWY. WASTEWATER
- FUTURE GALISTEO RANCH OVERSIZE
- FUTURE POSEY RD. - OLD BASTROP HWY. WASTEWATER
- WASTEWATER SYSTEM WITH CIP (WWL)
- WASTEWATER SYSTEM WITH CIP (F.M.)
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

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FIGURE 5.2

CIP PROJECTS - SOUTH

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File: K:/Hi1/441201/Docs/Report/Revised Maps 07_2005/shp/SM-FG-01	

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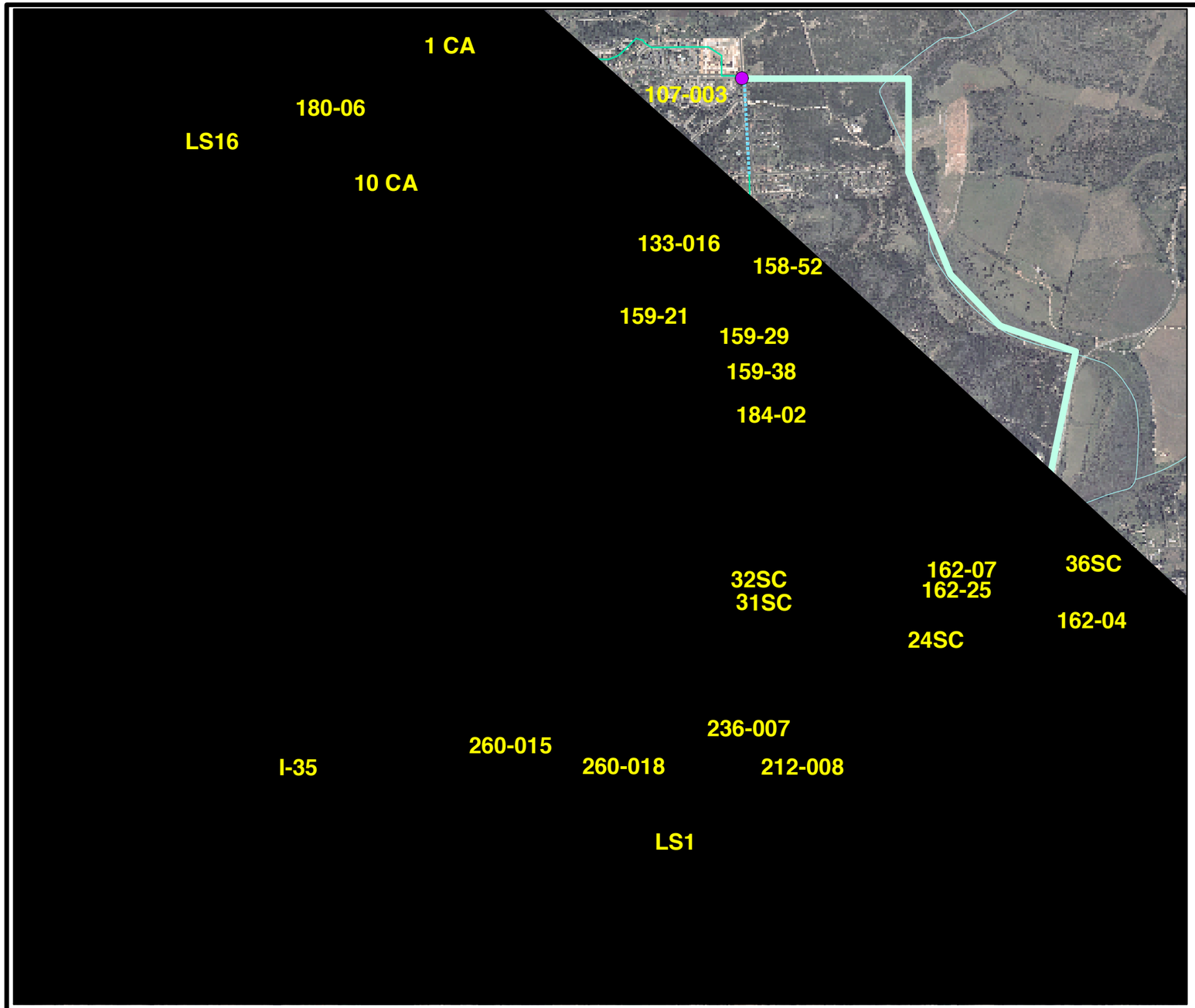

Legend

- LIFT STATIONS
- EXISTING MANHOLES (MODEL)
- EXISTING MANHOLES (GIS)
- CIP MANHOLES
- COMPLETED: CRADDOCK AVE. IMPROVEMENTS
- COMPLETED_SESSOMS/ LOOP 82
- 2005-2006 SESSOMS/ LOOP 82
- 2005-2006 MODIFIED RIVERSIDE MKT INTERCEPTOR
- 2005-2006 SINK CREEK PH-1
- 2006-2007 MODIFIED ROOSEVELT WW IMPROVEMENTS
- 2006-2007 RIO VISTA WW IMPROVEMENTS
- 2007-2008 EASTSIDE INTERCEPTOR PH-3
- 2007-2008 PURGATORY CREEK INTERCEPTOR
- 2009-2010 MLK/CM ALLEN/MCGEHEE IMPROVEMENTS
- 2012-2013 MODIFIED SINK CREEK PH-2
- F.M. UNDER INVESTIGATION
- WASTEWATER SYSTEM WITH CIP (WWL)
- WASTEWATER SYSTEM WITH CIP (F.M.)
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

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FIGURE 5.3

CIP PROJECTS - CENTRAL

Prepared for: CITY OF SAN MARCOS	
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Prepared by: S. MULLER	Date: 09/08/05
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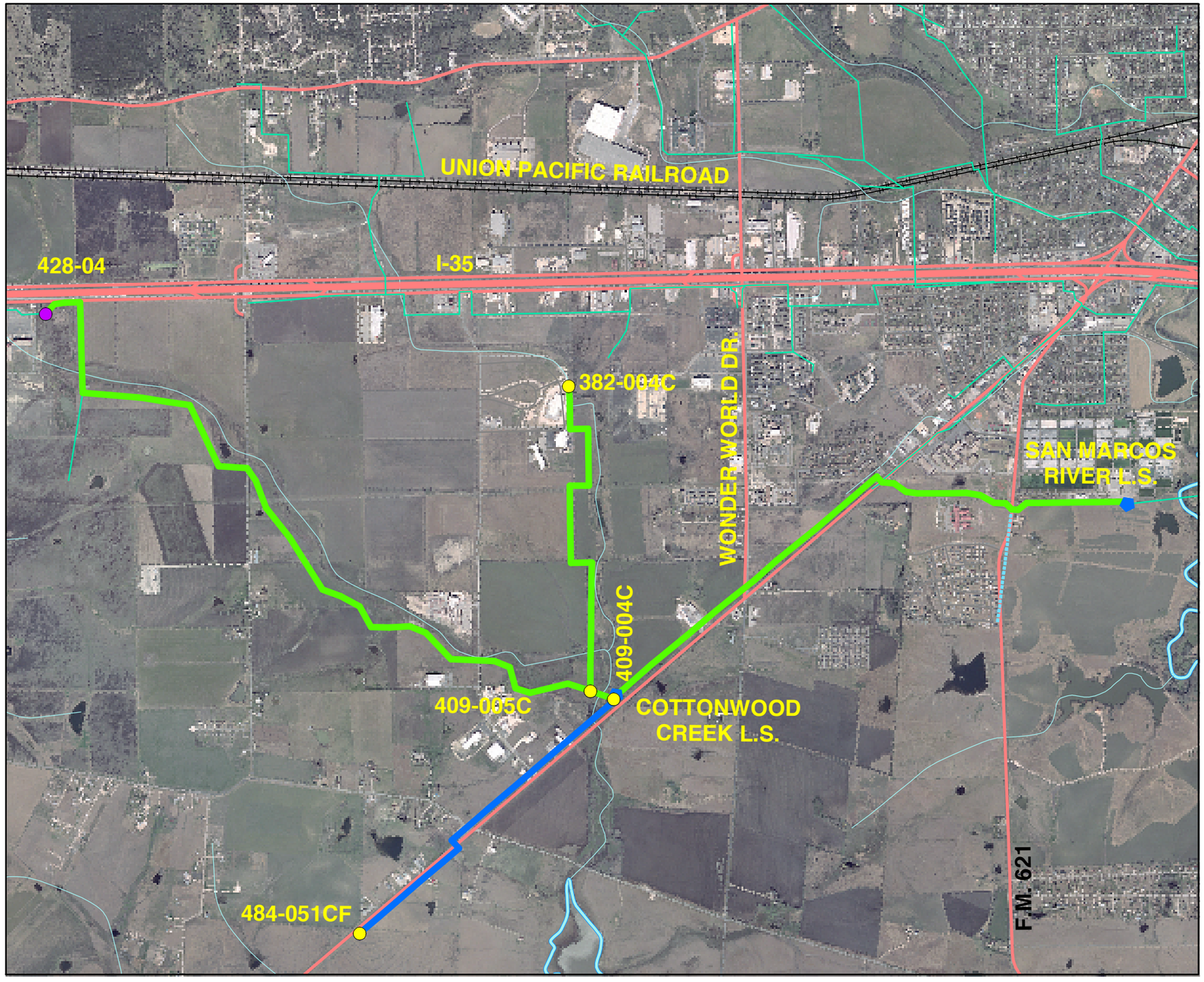

Legend

- EXISTING MANHOLES (MODEL)
- EXISTING MANHOLES (GIS)
- LIFT STATIONS
- COMPLETED: CN. WD. CRK. SUB. OFF SITE WL. & WWL. EXT. IMP.
- COTTONWOOD CREEK INTERCEPTOR
- WASTEWATER SYSTEM WITH CIP (WWL)
- WASTEWATER SYSTEM WITH CIP (F.M.)
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

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FIGURE 5.4

CIP PROJECTS - EAST

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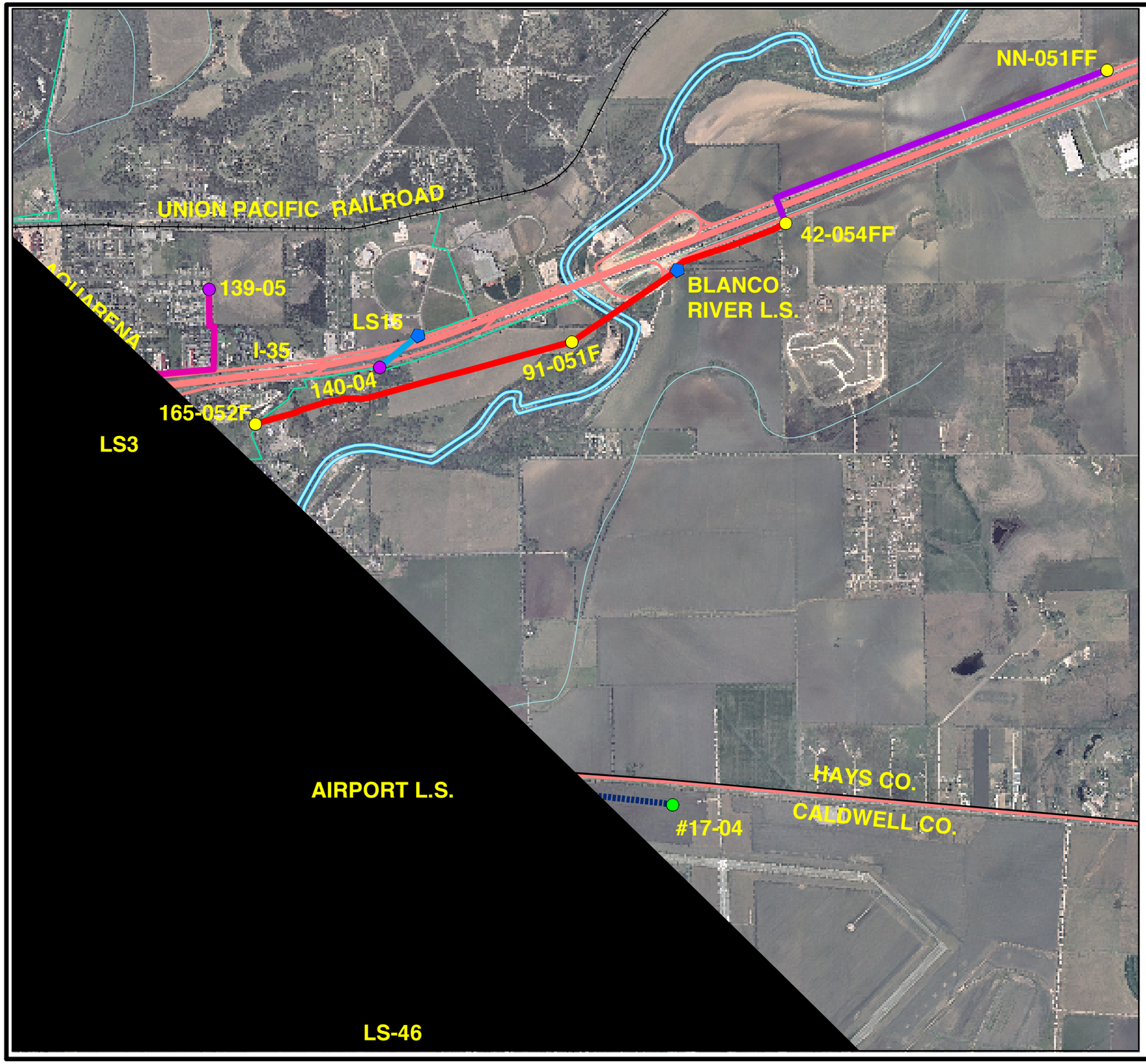

Legend

- EXISTING MANHOLES (MODEL)
- EXISTING MANHOLES (GIS)
- LIFT STATIONS
- CIP MANHOLES
- COMPLETED: NORTH I-35 INTERCEPTOR
- COMPLETED: I-35 FAIRLAWN/ BLANCO RIVER L.S.
- 2006-2007 OLD PLANT L.S. IMPROVEMENTS
- 2010-2011 RIVER RIDGE L.S.
- FUTURE AIRPORT L.S., F.M. & WWL
- WASTEWATER SYSTEM WITH CIP (WWL)
- WASTEWATER SYSTEM WITH CIP (F.M.)
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS
- COUNTY LINE

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FIGURE 5.5

CIP PROJECTS - NORTH

Prepared for: CITY OF SAN MARCOS	
Job No.: 441201.00	Scale: 1"=2000'
Prepared by: S. MULLER	Date: 09/08/05
File: K:/Hi1/441201/Docs/Report/Revised Maps 07_2005/shp/SM-FG-04	

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Project 1. The Riverside MKT project is listed as the first priority because this project is essential to development on the north side of the City. The Riverside MKT Interceptor will increase the available capacity in the northwest area of the City and allow for the Bobcat lift station to be taken out of service. Flows from future Sink Creek developments will be conveyed to the Main lift station via the Riverside MKT Interceptor. The original Riverside MKT Interceptor was designed as a 27-inch main, but with model results showing higher flows to be anticipated thorough the downstream section of the main, approximately 3700 feet of the downstream section was upsized to 30-inch line.

Project 2. The flow monitoring program indicated that some portions of the collection system experience high extraneous inflow rates during storm events, both directly in the flow monitoring data and also by statistical analysis of the flow monitoring data to predict inflow rates as a function of design storm rainfall intensity. A Sanitary Sewer Evaluation Survey (SSES) project is recommended in order to identify inflow source defects as the first step in a program for selected system rehabilitation measures to theoretically eliminate the worst rainfall-induced inflow/infiltration, but more practically, to prevent I/I from increasing over time. (It is usually found that rehab does not significantly reduce overall I/I. Although some I/I sources are eliminated by rehab, others develop new or un-repaired minor ones worsen over time.) The SSES should include manhole inspection, smoke testing, dye water flooding, and cleaning and closed circuit television inspection, as methods of identifying various defects. The SSES is required to determine appropriate defect repair methods to control I/I.

Various methods were used to analyze the 2003 flow monitoring data in order to identify the inflow and infiltration components of the total flow, as described in Section 2.2.1. However, the results were not entirely conclusive for identifying the highest priority SSES meter basins for several reasons. For example, the observed PWWF/ADWF ratios from flow data for the June 5, 2003 storm confirm high inflow rates in some areas, but the non-uniform rainfall intensity for that storm does not put all meter basins on equal footing for comparison. Even the two methods for statistical analysis of the flow data ($Q = a*I^{b}$ and $Q = KIA$) may respond differently to different types of defects producing inflow. Most meters showed inflow response to most of the monitored storms, but a few (e.g., GS-21) only showed significant response to the June 5 storm. Some meters showed a very sharp inflow spike with very short duration (e.g., GS-21), while others showed lower peaking with longer duration. The inflow response at GS-21 suggests an open manhole cover, located in a low-lying area subject to flooding only during more severe storm events, as the predominant inflow source. The SSES program needs to utilize various investigation methods to identify sources and to produce the most cost-effective rehabilitation program. Also, as discussed earlier, an upstream lift station can distort a flow meter's data so that it does not reflect true inflow characteristics in the overall meter basin. The same applies to any upstream overflows.

Although analysis of the flow monitoring data is not conclusive as to ranking of meter basins for SSES, it suggests the following priorities: (1) GS-3 meter basin; (2) GS-6 meter basin including LS-3 and LS-6 service areas; (3) GS-14 subbasin downstream of GS-12A, GS-19, and GS-21; (4) GS-13 subbasin downstream of GS-10; (5) GS-19 meter basin; (6) GS-21 meter basin; and, (7) GS-2 subbasin

downstream of GS-15. The meter subbasin boundaries are shown on Figure 2.1. The data and discussion presented in Section 2.2.1 are the basis for this listing, in addition to the following:

- It should be noted that some of the piping in the GS-3 meter basin is scheduled to be replaced in the Rio Vista CIP project, which could support SSES downgrading or else limiting SSES to the remaining pipes.

- The GS-6 meter subbasin itself is very limited in extent, which suggests the two upstream lift station service areas as significant inflow sources. As discussed in Section 2.2.2, inflow analysis for the lift stations was hindered by wet weather data problems due to different methodology.

- The implied high groundwater infiltration rate derived from dry weather flow monitoring data (see Section 2.2.1) supports the leakiness conclusion for the GS-14 meter subbasin, as well as for the GS-3 meter basin.

- GS-13 rates rather low in the inflow analysis calculations, but there is a history of overflows at an upstream location which might have distorted the flow monitoring data. Moreover, the base infiltration analysis ranks the GS-13 subbasin as second worst after GS-3, which is not unreasonable since a good portion of it is situated in a low-lying area along the San Marcos River.

- GS-19 and GS-21 showed the two highest PWWF/ADWF ratios for the June 5, 2003 storm as conclusive evidence of significant inflow problems. However, the projected total gal/min inflow rates for a 5-year design storm are not overly significant for the overall system downstream.

- GS-2 has a very large overall meter basin (including the GS-15 meter basin) which generates a high projected total gal/min inflow rate for a 5-year design storm, and it is consistent between the two Q vs. I methodologies. The GS-2 meter subbasin is much smaller and is proposed for SSES for cost efficiency and because the GS-15 flow monitoring data and Q vs. I results are not as reliable as for GS-2.

Projects 8, 11, 14. Three rehabilitation and replacement projects are included in the CIP to repair defects in the collection system found through the SSES CIP project. Through the SSES program, the necessary repairs should be identified and rehabilitated with this repair and replacement CIP. A set budgeted item has been included for alternating years to facilitate ongoing repair work necessary to preserve the collection system.

Project 3. The Sessoms/Loop 82 project listed in this CIP is an extension of the line improvements made in the completed Sessoms/Loop 82 project mentioned previously in Table 5.1. Even with the previous Sessoms/Loop 82 project, there are still two manholes shown to be spilling in the existing system under design storm conditions. The model indicates that the overflows are caused by insufficient capacity in the

lines just downstream of the manholes losing flow. The Sessoms/Loop 82 project listed in Table 5.2 will eliminate these overflows by pipe bursting downstream line segments to increase the pipes by one size.

Project 4. The next CIP project listed is Phase I of the Sink Creek project. The Sink Creek Phase I project connects to the Riverside MKT Interceptor and relieves sewer overflows occurring upstream of the Bobcat Stadium lift station. In addition to relieving sewer overflows, there are logistical benefits of simultaneous construction of the Riverside MKT Interceptor and the Sink Creek Phase I project.

Project 5. The Rio Vista Wastewater Improvements project includes lines that will provide flow to the Riverside MKT Interceptor. This project is included in the San Marcos CIP as an improvement/ replacement project for the entire Rio Vista subdivision. Details are unknown as to plans of upsizing any of the existing lines, so it is important to note that it is recommended that the 10-inch line included in the model be upsized to a 12-inch line. It is suggested that these improvements be made concurrently with the construction of the Riverside MKT Interceptor so that the Rio Vista tie-in line can be connected during construction of the Riverside MKT Interceptor.

Project 6. Improvements to the Old Plant lift station and the pipes upstream are high on the priority list due to the amount of flow lost from the sewer overflows occurring at and upstream of the lift station. The model indicates that the flow delivered to the Old Plant lift station exceeds the pumping capacity that exists at the station. This lack of capacity causes lines upstream of the lift station to surcharge in the existing system under design storm conditions and causes sewer overflows with the additional flows associated with the future conditions. The model results show that there is a substantial amount of flow lost from the manholes upstream of the lift station in the application of the design storms on the future systems.

Project 7. The Modified Roosevelt Wastewater Improvements project will replace and upsize older sewer mains to provide additional capacity needed in a residential area. The model results show that the lack of capacity causes three sewer overflows during design storm conditions in the existing system. These same three overflows exist in the model of the future conditions with the volume lost from the overflows increasing with time. The City's original plan for the Roosevelt Wastewater Improvements was modified to include an additional 2,000 linear feet of pipe improvements upstream of the project's location to provide more capacity and eliminate a spilling manhole.

Downstream of the Bishop Belvin residential area, there are two sewer overflows that occur due to lack of pipe capacity. The construction of the Eastside III Interceptor and Purgatory Creek Interceptor would allow for the Bishop Belvin flows to be re-routed, thus solving the overflow problems. The Eastside III Interceptor is listed before the Purgatory Creek because in order to re-route the flows through pipes with sufficient capacity, the construction of the Eastside III project is necessary prior to the construction of the Purgatory Creek Interceptor. It is understood by PBS&J from discussions with the City that the Eastside III project has been put on hold due to easement issues. If the easement issues were to cause further delay

of this project, then the overflows could be eliminated by pipebursting the surcharging lines causing the overflows downstream of the Bishop Belvin area. The pipebursting would include replacing approximately 3,000 linear feet of 12-inch pipe with 15-inch pipe.

Project 12. The MLK/CM Allen/McGehee Improvements increase capacity in lines located in the central part of San Marcos. Sewer lines along MLK Drive, CM Allen Parkway, and McGehee Street are all shown to surcharge in the model of the design storm in the existing and future systems. There are also two sewer overflows that occur along the surcharged lines. The line replacements recommended for the MLK/CM Allen/McGehee WWL would eliminate the sewer overflows and provide sufficient capacity for the future flows that will be conveyed by this line. This project also includes upsizing approximately 1,000 linear feet of 18-inch WWL just downstream of the MLK/CM Allen/McGehee WWL to 24-inch WWL in order to provide additional capacity necessary to convey the higher flows to the Main lift station.

Project 13. The River Ridge lift station improvements are a low priority because currently the station has the capacity necessary to pump the incoming flows; however, the lines upstream of the lift station are shown to be surcharging during the design storm conditions of the existing system. With the future flows applied, the pipes shown to surcharge in the existing system cause two sewer overflows to occur in the future systems. It is recommended that for the lift station to handle future incoming flows the pumping capacity should be increased to 0.6 MGD. This recommended increase in pumping capacity is based on projected future conditions governed by estimated increases in population. It should be noted that this increase in pumping capacity will not be necessary until the future flows generated in the model occur in the system. Flows to the River Ridge lift station should be monitored until such future flows exist and the increase in pumping capacity is needed.

Project 15. Phase II of construction of the Sink Creek Interceptor is listed as the last priority on the CIP list because this project is largely dependant on the future growth of the northwest section of the City. It is recommended that construction of Phase II of the Sink Creek Interceptor be driven by development. The timing of construction of this project should also take into consideration the relief of the North LBJ Lift Station (LS11). Wastewater lines downstream of LS11 will be overloaded if future flows from developments in this area are not diverted with the Phase II Sink Creek Interceptor.

The model indicates that an increase in pumping capacity is necessary at the Franklin Square lift station, but this improvement has not been prioritized as a project in the CIP listed in Table 5.2 due to the current evaluation underway. The Franklin Square lift station is currently under investigation as part of a lift station assessment and evaluation study that PBS&J and the City of San Marcos initiated in the spring of 2005. Through this study, the pumping capacity and additional improvements needs of the Franklin Square lift station will be assessed. At present, there is space available in the wet well of the lift station for installment of an additional (fourth) pump. The study will further assess the condition of the lift station and force main at Franklin Square and PBS&J will make recommendations on how to proceed with improvements to these facilities.

The costs associated with the prioritized CIP projects are outlined in Table 5.3.

TABLE 5.3. PRELIMINARY COST ESTIMATES FOR THE PRIORITIZED CIP PROJECTS

PRIORITIZED CIP PROJECTS			
Line Size (inch)	LF	Cost per LF	Total Cost
Riverside MKT			
27	5,178	$ 224	$ 1,160,000
30	3,791	256	970,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			781,000
		TOTAL COST:	**$ 3,011,000**
Task	Unit	Cost per Unit	Total Cost
SSES Selected Basins			
Review of Existing Data	1 EA	$ 1,500	$ 1,500
Above Ground Physical Inspection	70,000 LF	0.04	2,800
Smoke Testing	70,000 LF	0.33	23,000
Dyed Water Flooding/Leak Quantification	10 EA	125	1,300
Sanitary Sewer Main Cleaning and Internal Inspection	35,000 LF	0.27	9,500
CCTV	35,000 LF	2	84,000
Review and Engineering Recommendations of CCTV Data	35,000 LF	0.26	9,100
Engineering Recommendations			10,000
Overall Project Report			10,000
Mobilization			1,500
		TOTAL COST:	**$ 153,000**
Line Replacement Size (inch)	LF	Cost per LF	Total Cost
Sink Creek Phase I			
27	1,267	$ 168	$ 213,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			109,000
		TOTAL COST:	**$ 422,000**
Rio Vista			
12*	2,566	$ 82	$ 210,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			109,000
		TOTAL COST:	**$ 419,000**

Line Replacement Size (inch)	LF	Cost per LF	Total Cost
Line US of Old Plant LS			
10	742	$ 38	$ 28,000
12*	764	41	31,000
15	599	55	33,000
18	356	68	24,000
21	1,810	83	150,000
21*	1,325	83	110,000
24	261	97	25,000
Increase Pumping Capacity of LS from 0.6 mgd to 1.0 mgd			350,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			297,900
		TOTAL COST:	**$ 1,148,900**
Modified Roosevelt WW Improvements			
18	4,592	$ 136	$ 625,000
Misc. (Crossings, Borings)			200,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			324,000
		TOTAL COST:	**$ 1,248,000**
Rehab/Replacement for I/I Control			
Repair and replace defects identified through SSES			$ 500,000
		TOTAL COST:	**$500,000**
Eastside III			
24	862	$ 97	$ 84,000
30	1,564	128	200,000
33	2,953	159	470,000
Misc. (Borings, Crossings)			500,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			474,000
		TOTAL COST:	**$ 1,827,000**
Purgatory Creek			
21	2,314	$ 166	$ 384,000
24	6,229	194	1,208,000
Mobilization:			100,000
Contingency & Design Fees (35% of Construction Costs)			592,000
		TOTAL COST:	**$ 2,285,000**
Rehab/Replacement for I/I Control			
Repair and replace defects identified through SSES			$ 500,000
		TOTAL COST:	**$500,000**

Line Replacement Size (inch)	LF	Cost per LF	Total Cost
MLK, CM Allen, McGehee			
12*	1,668	$ 82	$ 137,000
15*	515	110	57,000
18*	384	136	52,000
18	1,671	136	227,000
24	868	194	168,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			259,000
		TOTAL COST:	**$ 1,001,000**
River Ridge Lift Station			
Increase Pumping Capacity to 0.6 MGD		**TOTAL COST:**	**$ 350,000**
Rehab/Replacement for I/I Control			
Repair and replace defects identified through SSES		**TOTAL COST:**	**$ 500,000**
Sink Creek Phase II			
18	8,876	$ 136	$ 1,207,000
21	5,073	166	842,000
Tie overs from LS 12, LS 44, LS 19, LS 38, LS 27			660,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			983,000
		TOTAL COST:	**$ 3,792,000**

**: Indicates that pipebursting is optional for this linear footage.*

Table 5.4 outlines projects included in the City of San Marcos CIP for the Fiscal Years 2004–2013. The construction dates of these CIP projects are dependent on future developments and/or the flow diversions associated with the future wastewater treatment alternatives. Further details of the future wastewater treatment alternatives and the associated CIP are discussed in Section 6.0. The timing and location of specific future development projects will depend on the general economy as well as local public infrastructure development. For example, growth on the east side of the City will be influenced by projects that are included in the San Marcos Transportation Master Plan such as the Outer Loop 110 and the State Highway 21 Extension to Posey Road.

The existing Uhland Road Interceptor is currently 24-inch and has an average capacity of 6.8 MGD. The IH 35/Fairlawn Interceptor contributes flow to the Uhland Road Interceptor and has an average capacity of 8.8 MGD. The Blanco River lift station conveys flow from the North IH 35 Interceptor service area to the IH 35/Fairlawn Interceptor and Uhland Road Interceptor, and it is planned that this lift station will have three phases of pumping capacities with an initial pumping capacity of 3.3 MGD and ultimate of 9.9 MGD. Before the ultimate pumping capacity is reached at the Blanco River lift station and when approximately 75% of the capacity of the IH 35/Fairlawn interceptor is utilized, the Uhland Road Interceptor will need to be upsized. The model results indicate that upsizing the Uhland Road Interceptor from 24 to 30 inch provides an average capacity of 11.1 MGD. With this capacity, the Uhland Road

Interceptor will not only be able to handle flows from the IH 35/Fairlawn Interceptor at its full capacity, but will also have additional capacity to handle flows from other surrounding areas.

TABLE 5.4. FUTURE CIP PROJECTS WHOSE CONSTRUCTION DATES ARE DRIVEN BY DEVELOPMENTS THAT ARE SCHEDULED AND/OR FUTURE TREATMENT CAPACITY ALTERNATIVES

Future Development Driven CIP Projects	
Uhland Road Interceptor Upsize*	Project timing dependent on: *Development in the northern section of San Marcos and increased flows from the IH 35/Fairlawn Interceptor, Blanco River lift station, and the North IH 35 Interceptor*
Cottonwood Creek LS Abandonment and 18-inch Force Main Turnaround to Gravity*	Project timing dependent on: *Construction of a new Southeast Wastewater Treatment Facility*
Galisteo Ranch Oversize	Project timing dependent on: *The development of Galisteo Ranch area*
Posey Road-Old Bastrop Highway Wastewater	Project timing dependent on: *Developments in southeast San Marcos*
Cottonwood Creek Interceptor Phase II*	Project timing dependent on: *Construction of a new Southeast Wastewater Treatment Facility*
Centerpoint WW East of Outlet Malls	Project timing dependent on: *Developments in south San Marcos*
Airport Lift Station and Force Main	Project timing dependent on: *Developments associated with the airport expansion*
State Highway 21 Lift Station, Force Main and By Pass Creek Interceptor*	Project timing dependent on: *Developments associated with the construction of Loop 110 and the airport expansion*
Oversize Old Bastrop Highway Wastewater Main*	Project timing dependent on: *Developments in southeast San Marcos*
Staples Road WW Oversize*	Project timing dependent on: *Smith Tract development and future developments on the south side of Staples Road*

*Indicates project that varies in detail depending on the treatment alternative chosen. These projects and the treatment alternatives are discussed further in Section 6.0.

The Galisteo Ranch Oversize project will provide sewer service to areas beyond the Galisteo Ranch development, which is located in the vicinity of Centerpoint Road and Old Bastrop Highway. The Galisteo Ranch Golf Course Community is a planned development in south San Marcos with sewer service being provided by facilities constructed and financed by the developers. The Galisteo Ranch Oversize CIP project is planned to upsize the lift station and force main associated with this development in order to provide sewer service beyond the limits of the Galisteo Ranch Community. Date of construction of the Galisteo Ranch Golf Course Community is unknown, thus this CIP project should be scheduled as needed.

The installation of a gravity wastewater line is planned in the vicinity of Posey Road and Old Bastrop Highway in order to serve existing and developing areas in south San Marcos that currently do not receive

sewer service. Two gravity main segments along Posey Road between IH 35 and Old Bastrop Highway will serve developments in south San Marcos, such as La Hacienda and the H&H Industrial Park. One segment will connect to the existing Toyota lift station and the other will connect to a future lift station in the vicinity of Centerpoint Road and Old Bastrop Highway. An additional lift station will be installed near the intersection of Posey Road and Old Bastrop Highway and a force main will be constructed to convey the flow back to IH 35. Construction of this CIP project should be planned to commence as developments progress.

Phase II construction of the Cottonwood Creek Interceptor refers to the installation of a wastewater line extending east of the Cottonwood Creek lift station to a proposed treatment plant in southeast San Marcos. This phase of construction is dependent on the construction of a new treatment facility in southeast San Marcos. The Phase II Cottonwood Creek Interceptor project also involves abandoning the Cottonwood Creek lift station and turning the lift station's 18-inch force main into a gravity main which will convey flow to the newly installed wastewater line, thence to a new southeast treatment facility. This project will also support growth in the southeast area of the City. Planned developments in this area include Lago Ranch, El Camino Real, Cottonwood Creek Business Park, Highway 123 Business Park, R&R Subdivision, and the Cottonwood Creek Subdivision. If the City chooses not to build a new southeast treatment facility and to instead implement the future treatment capacity at the existing wastewater treatment facility, then Phase II of the Cottonwood Creek Interceptor as currently planned is not applicable (see Section 6.0 of this report for details of the future treatment capacity alternatives).

The Centerpoint wastewater CIP project includes oversizing gravity lines east of the outlet malls to serve areas of undeveloped land. The gravity lines along Centerpoint Road to Old Bastrop Highway will be oversized in this CIP and the additional capacity will be available for new developments in this area including La Hacienda, H&H Industrial Park, Galisteo Ranch, and the future expansion of the outlet malls. The timing of this CIP will be governed by the need for additional wastewater capacity for development in south San Marcos.

The proposed expansion of the airport will govern the timing of the Airport lift station and force main CIP project. This project includes the construction of a wastewater line along SH 21 from the expanded hangars at the San Marcos Airport to a new lift station on SH 21. A force main will also be installed to tie the new lift station to the existing lift station at the Gary Job Corps site. These facilities will support future developments and growth with the expansion of the airport and construction of the FM Loop 110.

The SH 21 Lift Station, Force Main and By Pass Creek Interceptor CIP project is located in the same area as the Airport lift station and force main CIP project, but is scheduled to be constructed following the Airport CIP and will eventually relieve the lift station of the Airport CIP project. The SH 21 lift station, force main and By Pass Creek Interceptor CIP project will provide sewer service to future developments associated with FM 110 and the airport expansion. According to the future land use maps provided by the City, these areas expect to be occupied by industrial and public/institutional developments. The CIP

project includes the construction of a lift station, force main, and gravity line. The lift station is planned to have a pumping capacity of 1.5 MGD and will be located near the intersection of SH 80 and CR 101. The gravity wastewater line will extend north from the lift station along By Pass Creek across SH 21 continuing north to Saddlebrook and Harris Hill Road. The force main will run along Old Martindale Road to River Road and its discharge location will be determined by the location of additional future wastewater treatment facilities. See Section 6.0 for treatment alternatives and the associated CIP.

The Old Bastrop Highway and Staples Road Oversize projects are planned in the City's CIP as projects they will participate in to upsize wastewater lines originally constructed and funded by developers of the areas. The sizes of these lines will be determined based on how flows are routed for future treatment. The Old Bastrop Highway wastewater line will need to be oversized considerably if flows are diverted to a new Southeast Wastewater Treatment plant. The flow direction of the force main associated with the Staples Road project will also be decided by the location of the future treatment facilities. These lines will serve developments in southeast San Marcos including Lago Ranch and El Camino Real. Further discussion of the details of these projects with respect to treatment alternatives is provided in Section 6.0.

The costs associated with the development driven CIP projects (projects independent of the future treatment alternatives) are outlined in Table 5.5. Costs for the projects that are driven by the future treatment alternatives will be presented in the following section.

TABLE 5.5. PRELIMINARY COST ESTIMATES FOR DEVELOPMENT DRIVEN PROJECTS THAT ARE NOT DEPENDENT ON THE FLOW DIVERSION ASSOCIATED WITH FUTURE WASTEWATER TREATMENT PLANT ALTERNATIVES

DEVELOPMENT DRIVEN CIP*			
Line Size (inch)	LF	Cost per LF	Total Cost
Galisteo Ranch Oversize			
12 (Force Main)	4,694	$ 50	$ 235,000
15	1,552	55	85,000
1.0 MGD Lift Station			600,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			357,000
		TOTAL COST:	**$ 1,377,000**
Posey-Old Bastrop Wastewater			
10	5,358	$ 38	$ 204,000
8 (Force Main)	2,995	40	120,000
0.86 MGD Lift Station			600,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs):			358,000
		TOTAL COST:	**$ 1,382,000**

Line Size (inch)	LF	Cost per LF	Total Cost
Centerpoint WW - East of Outlet Malls			
12	4,485	$ 60	$ 269,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			129,000
		TOTAL COST:	**$ 498,000**
Airport Lift Station and Force Main			
12	8,966	$ 80	$ 717,000
4 (Force Main)	4,584	25	115,000
0.89 MGD Lift Station			600,000
Mobilization			100,000
Contingency & Design Fees (35% of Construction Costs)			536,000
		TOTAL COST:	**$ 2,068,000**

** All projects listed are not dependent on flow diversions associated with location of future treatment capacity. Costs associated with projects not listed will be based on chosen WWTP alternative.*

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6.0 ADDITIONAL TREATMENT CAPACITY ALTERNATIVES

A "special studies" task was performed to evaluate the need and potential sites for additional treatment capacity to handle the projected future wastewater flows. The existing treatment capacity, planned CIP projects, estimated future growth, and projected future flows were all taken into account in developing strategies to provide additional treatment capacity.

The existing treatment plant on River Road has current permitted capacity of 9 MGD for average daily flow (maximum month) and 18 MGD for 2-hour peak flow. The City has indicated that the plant is rated for process at 7.6 MGD for average dry weather and has a capacity to treat 18 MGD of peak wet weather flow. Studies by others are currently underway to address some capacity issues within the plant. According to the modeling analysis of the 2027 system, the average dry weather flow in 2027 will be 14.9 MGD, and the peak wet weather flow for a 5-year 6-hour design storm will be 56.4 MGD. Consequently, the 7.6 MGD process capacity for the existing plant is only about one-half of the projected 2027 average flow of almost 15 MGD. In the past the City has anticipated a new future Southeast treatment plant for future flows. Major issues for a new Southeast treatment plant are (a) the possible collection system locations at which sufficient flow can be diverted away from the existing plant, and (b) the piping and pumping improvements needed to deliver the flows to the new plant.

Initially four scenarios were evaluated for the future wastewater treatment program for San Marcos. All four of the scenarios included maintaining or expanding the existing River Road treatment facility and rerouting the treated effluent to a new discharge point downstream of the Cummings Dam on the San Marcos River. None of the scenarios considered abandoning the existing facility. The new discharge point for the existing plant is related to endangered species issues as discussed below.

Scenario A. The first scenario opted to expand the existing facilities to treat all future flows, i.e., the City would not have a second treatment plant. However, in the past the City has indicated to the community in the vicinity of the plant that it would not be expanded, so that issue would need to be addressed if this alternative moves forward. The remaining scenarios involved diverting flows to new treatment facilities located in southeast San Marcos in either the San Marcos River watershed or the Cottonwood Creek watershed.

Scenario B. This scenario diverts only the flows in the service area for Cottonwood Creek Interceptor system to a new treatment facility that would be located on and discharge to Cottonwood Creek. The service area would include new wastewater lines built in southeast San Marcos with future growth. The purpose of this scenario was to reduce the amount and cost of new piping and pumping improvements needed to divert flows from other parts of San Marcos to the new plant. However, in this scenario, the projected 2027 average daily flow to the Cottonwood Creek treatment plant would only be about 2.7 MGD, so the existing River Road treatment facilities would still have to be expanded to handle all remaining flows. The community issue about expanding the existing plant is again a factor for this scenario.

Scenario C. Another scenario considered transporting all of the wastewater to the existing treatment facilities, treating the amount of flow allowed at the existing capacity, and pumping the remaining untreated flow to a new treatment facility located in the San Marcos River watershed near where the proposed Outer Loop roadway will cross the San Marcos River. The new treatment facility and the existing plant would have effluent outfalls to the same discharge point (with a single permit) near the confluence of the San Marcos and Blanco Rivers.

Scenario D. The final scenario is essentially Scenario B (new treatment plant on Cottonwood Creek receiving flow from the Cottonwood Creek Interceptor service area), plus additional flow diversions from (a) a new lift station on the east side of IH 35 south of the San Marcos River to divert flow from the Eastside II and future Eastside III/Purgatory Creek interceptors, (b) rerouted flows from the San Marcos River lift station, (c) new wastewater lines in the vicinity of SH 21 and the San Marcos Airport, and (d) the area upstream of River Road lift station (via the new lines near the airport). These widespread interceptions from the existing collection system are needed to increase the amount of flow diverted to the new treatment plant. In this scenario, the existing plant's current treatment capacity of 7.6 MGD will be adequate for the average dry weather flows generated in its reduced 2027 service area.

In developing the potential scenarios for diversion of flows to a new treatment plant, a major consideration was the possible ability of flow diversions to provide the additional benefit of eliminating the need for capacity improvements for overloaded lines in the existing system, especially in the older central part of the system where relief main construction can be difficult. However, this was not practical due to the configuration of the collection system. The potential flow diversion points closest to a new Southeast treatment plant are in downstream portions of the trunk mains carrying flow to the existing plant, but eliminating some relief mains would require flow diversions farther upstream, which extends the length of the diversion mains, reduces the amount of flow intercepted in some cases, and requires construction of diversion mains in the same area where the relief mains would be avoided. In addition, the extensive branching of the trunk mains transporting flows to the existing plant (mostly along and west of IH 35) does not provide just one or two potential upstream diversion points that would produce the required amount of flow for the new plant.

At the request of the City, two of the future treatment scenarios were further evaluated for meeting treatment capacity needs for 2027 flows. **Alternative 1** (from Scenario A) involves the expansion of the existing plant to treat all future flows with a new discharge point downstream of the Cummings Dam on the San Marcos River. **Alternative 2** (from Scenario D) includes the diversion of all future flows greater than the existing plant's capacity to a new facility on Cottonwood Creek. A variation, **Alternative 2A**, investigates diverting the flows from the North IH 35 corridor at the Blanco River Lift station rather than the River Road Lift Station. The scenarios for a small Cottonwood Creek facility and expanded River Road plant (Scenario B) and for a second plant in the San Marcos River watershed (Scenario C) are not pursued further.

Cost estimates of gravity lines, force mains, and lift stations needed for conveyance of the flows to the treatment sites in both of the alternatives were assessed and are presented in Table 6.1. Cost of future treatment capacity for the purpose of this current evaluation is considered to be the about the same whatever the location, relative to differences in the costs of the conveyance alternatives.

6.1 ALTERNATIVE 1

Expansion of the existing treatment facilities to treat all future flows was further examined as Alternative 1. In order to treat the projected flows of the year 2027, the existing plant will need a 7.3-MGD process capacity increase for average dry weather flow and a 38.4 MGD hydraulic capacity increase for peak wet weather flow (based on 5 year 6 hour design storm conditions). A limiting factor for expansion at the current site is aesthetic impacts to adjacent residential properties. The City already owns land surrounding the plant as buffer, but acquisition of additional land to extend the buffer may be required for the physical plant expansion given the community involvement issues, which would be a factor for this alternative to move ahead.

The proposed expansion of the existing River Road treatment facility in Alternative 1 includes relocation of the effluent discharge to a point further downstream on the San Marcos River, downstream of its confluence with the Blanco River. Currently, the existing plant discharges effluent to the San Marcos River at the plant site. However, this reach of the river is habitat for endangered species. The City has a history of involvement with regulatory agencies, environmental groups, and the downstream property owners on this issue. In response to the uncertainty of obtaining, and renewing, a revised discharge permit for higher flows at the existing location, Alternative 1 includes a new outfall, namely, the construction of an effluent lift station and force main with capability to pump and carry effluent peak flows. The force main length, depending on the routing and discharge location, would range between approximately 4,000 feet (cross-country to the confluence) and 14,000 ft (following road right-of-way to the San Marcos River at Old Bastrop Highway downstream of the Cummings dam). The worst case is assumed for cost comparison purposes with Alternative 2. It might be possible to keep the existing discharge at its current permit levels and somewhat reduce the size and cost for the new outfall under a second permit, but there are advantages to maintaining a single permit for a discharge downstream of the restricted portion of the San Marcos River. Considering the significant cost of a new outfall, the feasibility (including public involvement, treatment level requirements, and costs) of obtaining permitting for higher future flows at the existing outfall, as the only discharge point, should be evaluated before defaulting to a new outfall for significant expansion of the existing treatment plant under Alternative 1.

The planned CIP projects specific to Alternative 1 will need to be tailored to transport all flow to the existing treatment plant as outlined in Table 6.1 for the existing service area and future growth areas. The table provides details of each project necessary for conveyance of flow to the existing plant and the estimated cost associated with each. Figure 6.1 labels the CIP projects required for the flow diversions associated with Alternative 1.

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TABLE 6.1. SUMMARY OF CIP AND COSTS OF FUTURE TREATMENT ALTERNATIVES

DESCRIPTIONS AND ESTIMATED PROBABLE COSTS: PROJECTS REQUIRED FOR EACH OF THE ADDITIONAL WASTEWATER TREATMENT ALTERNATIVES

		Alternative 1		Alternative 2		Alternative 2 A	
		Take all flow to existing WWTP, incl Cottonwood Ck; expand plant for 14.9/56.4 yr 2027 flows; pump effluent to new outfall downstream of confluence & dam		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via River Road LS)		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via Blanco River LS)	
Project ID	Project Location	Description	Construction Cost	Description	Construction Cost	Description	Construction Cost
1	Existing WWTP	Expand to 14.9 avg/56.4 peak ***Note:*** *Expansion of existing facilities discussed with cost in report by CH2M Hill*	$$$*	Maintain 7.6 avg/increase capacity for peak 36.8 MGD	$$$*	Maintain 7.6 avg/increase capacity for peak 36.8 MGD	$$$*
2	Future Cottonwood Ck WWTP	NA	NA	Build 7.4 avg/19.5 peak WWTP	$$$*	Build 7.4 avg/19.5 peak WWTP	$$$*
3	Cottonwood Ck Interceptor south of Old Bastrop Highway	NA	NA	6,750 LF of 42-inch WWL Gravity Line to C'wood Plant Peak Flow = 19.5 MGD	$ 1,289,250.00	6,750 LF of 42-inch WWL Gravity Line to C'wood Plant Peak Flow = 19.5 MGD	$ 1,289,250.00
4	Cottonwood Ck main between OBH and CCLS	3,600 LF of 8-inch Force Main Force main to convey flow from OBH to CCLS Peak Flow = 0.5 MGD	$ 126,000.00	3,600 LF of 30-inch WWL Gravity line flow from CCLS to OBH Peak Flow = 7.0 MGD	$ 461,000.00	3,600 LF of 30-inch WWL Gravity line flow from CCLS to OBH Peak Flow = 7.0 MGD	$ 461,000.00
5	Linda Drive Improvements	Upsize existing WWL from 18 inch to 21 inch and 24 inch to provide necessary capacity to convey flow from Old Plant LS	$ 1,398,000.00	NA	NA	NA	NA
6	CIP #21** – Old Bastrop Highway	6,300 LF of 8-inch WWL Peak Flow = 0.5 MGD; 0.5 MGD Lift Station to Pump Flow from OBH to the CCLS	$ 620,500.00	6,300 LF of 27-inch WWL Peak Flow = 12. 6 MGD	$ 706,000.00	6,300 LF of 27-inch WWL Peak Flow = 12.6 MGD	$ 706,000.00

* Costs indicated by '$$$' assume that the cost of this component will be the same regardless of the treatment alternative chosen, thus a quantitative cost is not provided for reasons of comparison.

** The City CIP for 2002–2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines.

*** The new outfall line (listed as project 9) may not be required depending on the future possible discharge permitting issues at the current outfall location.

TABLE 6.1 (CONT'D)

		Alternative 1		Alternative 2		Alternative 2 A	
		Take all flow to existing WWTP, incl Cottonwood Ck; expand plant for 14.9/56.4 yr 2027 flows; pump effluent to new outfall downstream of confluence & dam		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via River Road LS)		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via Blanco River LS)	
Project ID	Project Location	Description	Construction Cost	Description	Construction Cost	Description	Construction Cost
7	CIP #22** Smith Tract WW line	8,400 LF of 12-inch WWL; 3,900 LF of 4-inch FM to SMRLS Peak Flow = 0.2 MGD	$ 441,900.00	8,400 LF of 12-inch WWL Peak Flow = 0.2 MGD	$ 344,400.00	8,400 LF of 12-inch WWL Peak Flow = 0.2 MGD	$ 344,400.00
8	New WWTP outfall line***	14,000 LF of 48-inch FM Peak Flow = 56.2 MGD	$ 3,150,000.00	14,000 LF of 42-inch FM Peak Flow = 36.8 MGD	$ 2,674,000.00	14,000 LF of 42-inch FM Peak Flow = 36.8 MGD	$ 2,674,000.00
9	New lift station at existing WWTP	1- Effluent Lift Station with Capacity to Pump 56.4 MGD	$ 3,500,000.00	1- Effluent Lift Station with Capacity to Pump 36.8 MGD	$ 2,800,000.00	1- Effluent Lift Station with Capacity to Pump 36.8 MGD	$ 2,800,000.00
10	Abandon Cottonwood Ck LS	NO – Do Not Abandon	NA	YES – Abandon Lift Station	$ 20,000.00	YES – Abandon Lift Station	$ 20,000.00
11	Force Mains from Main LS to Existing WWTP; Currently 2- 3,960 LF 20-inch FM	Add'l 48-inch FM needed for capacity of 48.5 MGD: 3,960 LF of 48-inch FM	$ 891,000.00	Add'l 42-inch FM needed for 36.8 MGD capacity: 3,960 LF of 42-inch FM	$ 756,360.00	Add'l 42-inch FM needed for 36.8 MGD capacity: 3,960 LF of 42-inch FM	$ 756,360.00
12	SH 21 Lift Station	Peak Flow = 0.5 MGD	$ 400,000.00	Peak Flow = 5.1 MGD	$ 900,000.00	Peak Flow = 5.1 MGD	$ 900,000.00
13	Force Main from SH 21 LS to Existing WWTP	10,800 LF of 4-inch FM Peak Flow = 0.5 MGD	$ 270,000.00	NA	NA	NA	NA
14	Force Main from SH 21 LS to CIP #21-OBH	NA	NA	7.700 LF of 18-inch FM Peak Flow = 5.1 MGD	$ 523,600.00	7.700 LF of 18-inch FM Peak Flow = 5.1 MGD	$ 523,600.00
15	Force Main from River Road LS to ByPass Interceptor	NA	NA	7,200 LF of 18-inch FM Peak Flow = 4.6 MGD	$ 489,600.00	NA	NA
16	Improvements to River Road LS	NA	NA	Increase Pumping Capacity to Provide Firm Pumping Capacity = 4.6 MGD	$ 250,000.00	NA	NA

* Costs indicated by '$$$' assume that the cost of this component will be the same regardless of the treatment alternative chosen, thus a quantitative cost is not provided for reasons of comparison.

** The City CIP for 2002–2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines.

*** The new outfall line (listed as project 9) may not be required depending on the future possible discharge permitting issues at the current outfall location.

TABLE 6.1 (CONT'D)

		Alternative 1		Alternative 2		Alternative 2 A	
		Take all flow to existing WWTP, incl Cottonwood Ck; expand plant for 14.9/56.4 yr 2027 flows; pump effluent to new outfall downstream of confluence & dam		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via River Road LS)		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via Blanco River LS)	
Project ID	Project Location	Description	Construction Cost	Description	Construction Cost	Description	Construction Cost
17	Cape Road LS	NA	NA	Corner of Cape Road and Luciano – LS w/capacity to divert Eastside II and III flows to new facility Peak Flow = 6.5 MGD	$ 900,000.00	Corner of Cape Road and Luciano – LS w/capacity to divert Eastside II and III flows to new facility Peak Flow = 6.5 MGD	$ 900,000.00
18	Force Main from Cape Road LS to SMRLS	NA	NA	4,080 LF of 18-inch FM Peak Flow = 6.5 MGD	$ 277,440.00	4,080 LF of 18-inch FM Peak Flow = 6.5 MGD	$ 277,440.00
19	Force Main from SMRLS to Smith Tract LS	NA	NA	3,900 LF of 18-inch FM Peak Flow = 7.4 MGD (v = 4.19 fps through FM)	$ 265,200.00	3,900 LF of 18-inch FM Peak Flow = 7.4 MGD (v = 4.19 fps through FM)	$ 265,200.00
20	Increase Capacity at San Marcos River LS	Improve to meet capacity of Peak Flow = 7.9 MGD	$ 225,000.00	NA	NA	NA	NA
21	Smith Tract LS (Part of CIP #22)	Lift Station to handle Peak Flow = 0.2 MGD	$ 500,000.00	Lift Station to handle Peak Flow = 7.4 MGD	$ 900,000.00	Lift Station to handle Peak Flow = 7.4 MGD	$ 900,000.00
22	Force Mains from SMRLS to WWTP; Currently 2- 2,400 LF 10-inch FM	Add'l 24-inch FM needed for capacity of 7.93 MGD: 2,400 LF of 24-inch FM	$ 232,800.00	NA	NA	NA	NA
23	Force Main from Smith Tract LS to #21 WWL in Old Bastrop Hwy	NA	NA	6,000 LF of 24-inch FM Peak Flow = 7.4 MGD	$ 408,000.00	6,000 LF of 24-inch FM Peak Flow = 7.4 MGD	$ 408,000.00
24	Increase Capacity at Cottonwood Creek LS	Add'l Capacity needed to handle Peak Flow = 7.1 MGD	$ 200,000.00	NA	NA	NA	NA
25	By-Pass Creek Interceptor	3700 LF of 8" WWL, 15,500 LF of 12" WWL	$ 765,000.00	3,700 LF of 8-inch WWL, 21,215 LF of 12-inch WWL	$ 999,315.00	3,700 LF of 8-inch WWL, 6,615 LF of 18-inch WWL, 5,000 LF of 21-inch WWL, 9,600 LF of 24-inch WWL 3,500 LF of 16-inch FM from Blanco River LS	$ 2,118,000.00

* Costs indicated by '$$$' assume that the cost of this component will be the same regardless of the treatment alternative chosen, thus a quantitative cost is not provided for reasons of comparison.

** The City CIP for 2002–2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines.

*** The new outfall line (listed as project 9) may not be required depending on the future possible discharge permitting issues at the current outfall location.

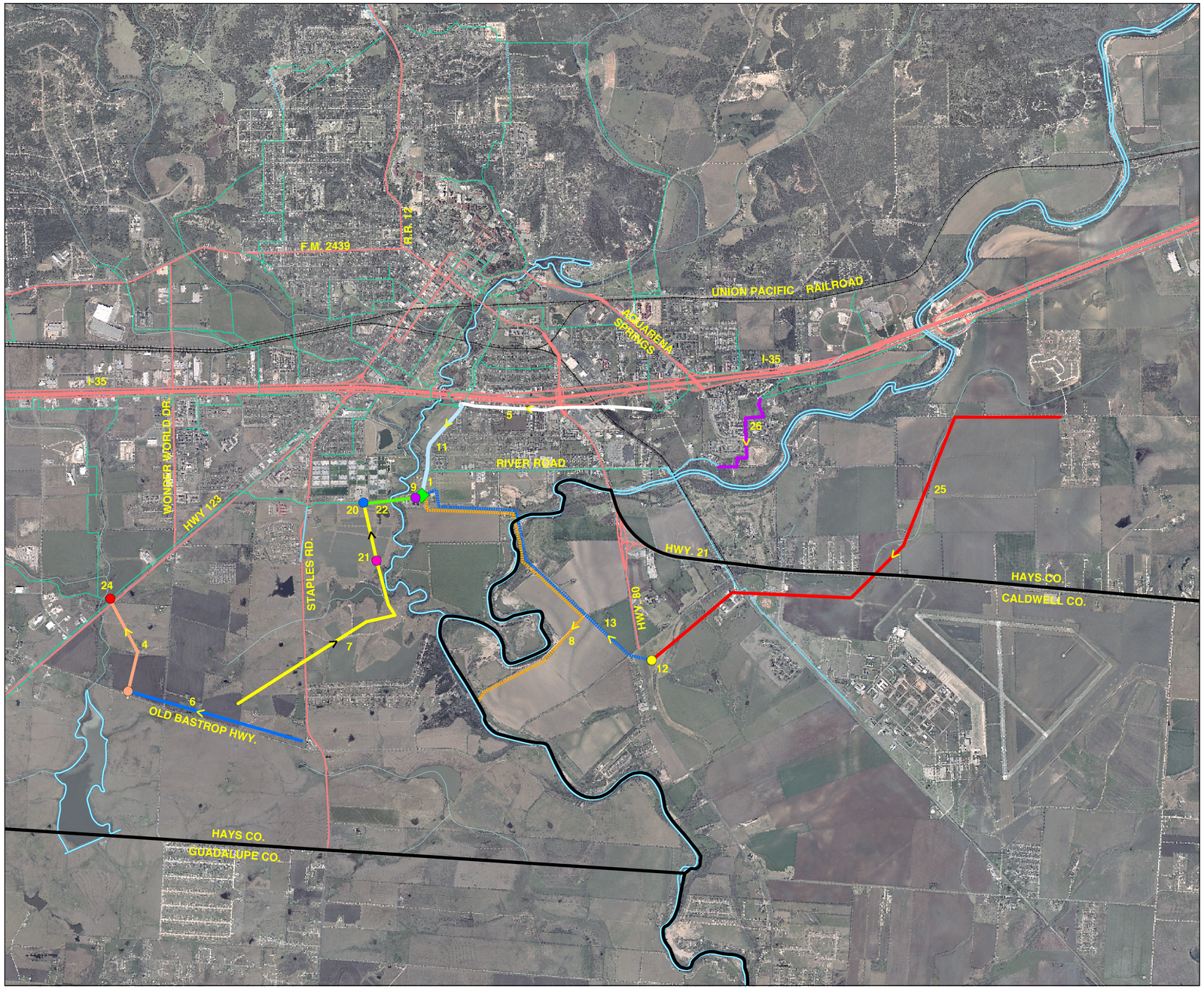
TABLE 6.1 (CONT'D)

		Alternative 1		Alternative 2		Alternative 2 A	
		Take all flow to existing WWTP, incl Cottonwood Ck; expand plant for 14.9/56.4 yr 2027 flows; pump effluent to new outfall downstream of confluence & dam		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via River Road LS)		Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek (North flow diverted via Blanco River LS)	
Project ID	Project Location	Description	Construction Cost	Description	Construction Cost	Description	Construction Cost
26	Uhland Road Interceptor	Upsize existing 3888 LF 24" WWL with 30" WWL	$ 583,200.00	Upsize existing 3888 LF 24" WWL with 30" WWL	$ 583,200.00	NA	NA
27	Old Plant LS Divervsion – WWL to River Road LS	NA	NA	3,580 LF of 12-inch WWL	$ 333,150.00	3,580 lf of 12-inch WWL	$ 333,150.00
Total			$ 13,303,400.00		$ 15,880,515.00		$ 15,676,400.00

* Costs indicated by '$$$' assume that the cost of this component will be the same regardless of the treatment alternative chosen, thus a quantitative cost is not provided for reasons of comparison.

** The City CIP for 2002–2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines.

*** The new outfall line (listed as project 9) may not be required depending on the future possible discharge permitting issues at the current outfall location.



Legend

- 1. EXISTING W.W.T.P. EXPANSION
- 4: COTTONWOOD CREEK (BETWEEN OBH & CNWOOD CREEK L.S.) F.M. & L.S.
- 5. LINDA DRIVE IMPROVEMENTS
- 6. OLD BASTROP HWY - FM 621 WWL
- 7. SMITH TRACT WWL & F.M.
- 8. NEW W.W.T.P. OUTFALL LINE
- 9. NEW EFFLUENT L.S. AT EXPANDED W.W.T.P.
- 11. F.M. FROM MAIN L.S. TO EXIST. W.W.T.P.
- 12. SH21 L.S.
- 13. F.M. FROM SH21 L.S. TO EXIST. W.W.T.P.
- 20. SAN MARCOS RIVER L.S. EXPANSION
- 21. SMITH TRACT L.S.
- 22. F.M. FROM S.M. RIVER L.S. TO EXIST. W.W.T.P.
- 24. INCREASE CAPACITY AT COTTONWOOD L.S.
- 25. BY PASS CREEK INTERCEPTOR
- 26. UHLAND ROAD INTERCEPTOR UPSIZE
- FUTURE WASTEWATER SYSTEM (F.M.) WITH CIP FROM SECTION 5
- FUTURE WASTEWATER SYSTEM (WWL) WITH CIP FROM SECTION 5
- COUNTY LINE
- HIGHWAYS
- CITY STREETS
- WATERWAYS
- CREEKS
- RAILROADS

NOTES:

1. REFERENCE TABLE 6.1 FOR PROJECT DETAILS.

2. DOES NOT SHOW CIP PROJECTS INDEPENDENT OF TREATMENT ALTERNATIVES (SEE FIGURE 5.1)

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FIGURE 6.1
WW TREATMENT DIVERSION ALTERNATIVE 1

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Job No.: 441201.00	Scale: 1"=2000'
Prepared by: S.MULLER	Date: 10/08/05
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6.2 ALTERNATIVE 2

Previous long-range planning has anticipated a new Southeast Wastewater Treatment Plant in the Cottonwood Creek watershed when future flows exceed the existing plant's treatment capacity, thus the evaluation of Alternative 2. The current CIP shows a wastewater interceptor extending between the existing Cottonwood Creek lift station and a possible treatment plant site near the community of Redwood downstream of the Dam No. 13 impoundment on Cottonwood Creek.

Modeling analysis for the wastewater master plan shows that in year 2027 the Cottonwood Creek lift station would have average dry weather flow of 2.7 MGD and peak wet weather flow of about 7.0 MGD (based on 5-year 6-hour design storm conditions). With flows generated in the Cottonwood Creek watershed (including along Old Bastrop Highway) delivered directly to a new Cottonwood Creek treatment plant, keeping the existing River Road treatment plant within its current rated capacity of 7.6 MGD would require diverting an additional average flow 4.2 MGD from the existing plant.

A number of alternatives were investigated for locations in the collection system to divert the additional flow. Figure 6.2 illustrates the flow diversion required for Alternative 2. The proposed diversions are as follows.

- South – Intercepting flow in the Eastside I and the Eastside III/Purgatory Creek interceptors would divert 2.13 MGD of average dry flow and 6.49 MGD of peak wet weather flow from the existing plant. This diversion to the new Cottonwood Creek plant would require a new lift station on the east side of IH 35 between Highway 123 and the San Marcos River.

- North – Diverting the flow in the Fairlawn and Uhland Road Interceptors to the new Cottonwood Creek plant (via the River Road Lift Station, the proposed ByPass Creek interceptor, and a new lift station located near the intersection of Highway 80 and CR 101) would redirect average dry flow of 2.17 MGD and 5.10 MGD of peak wet weather flow based on a 5-year 6-hour design storm.

A variation of Alternative 2 would be to divert the north-side flow to the By Pass Creek interceptor using the Blanco River lift station instead of the River Road lift station. This "Alternative 2A" would avoid the need for a new force main and pump upgrade for the River Road lift station (projects #15 and #16 in Table 6.1) and also eliminate the need for a future relief main for the Uhland Road interceptor (project #26). However, it would require a new force main for the Blanco River lift station and upsizing of the ByPass Creek interceptor (project #25), and it would not utilize the recently completed 30/36-inch IH-35/Fairlawn interceptor. These flow diversions for Alternative 2A are shown in Figure 6.3.

With both Alternatives 2 and 2A redirecting the flow from Old Plant lift station to go to River Road could avoid the need for a future relief main for the 18-inch Linda Drive main. As indicated in Section 5, a future upgrade is required for the Old Plant lift station. With Alternative 1 this pumping increase will require a future relief main for the 18-inch Linda Drive main (project #5 in Table 6.1). The Linda Drive

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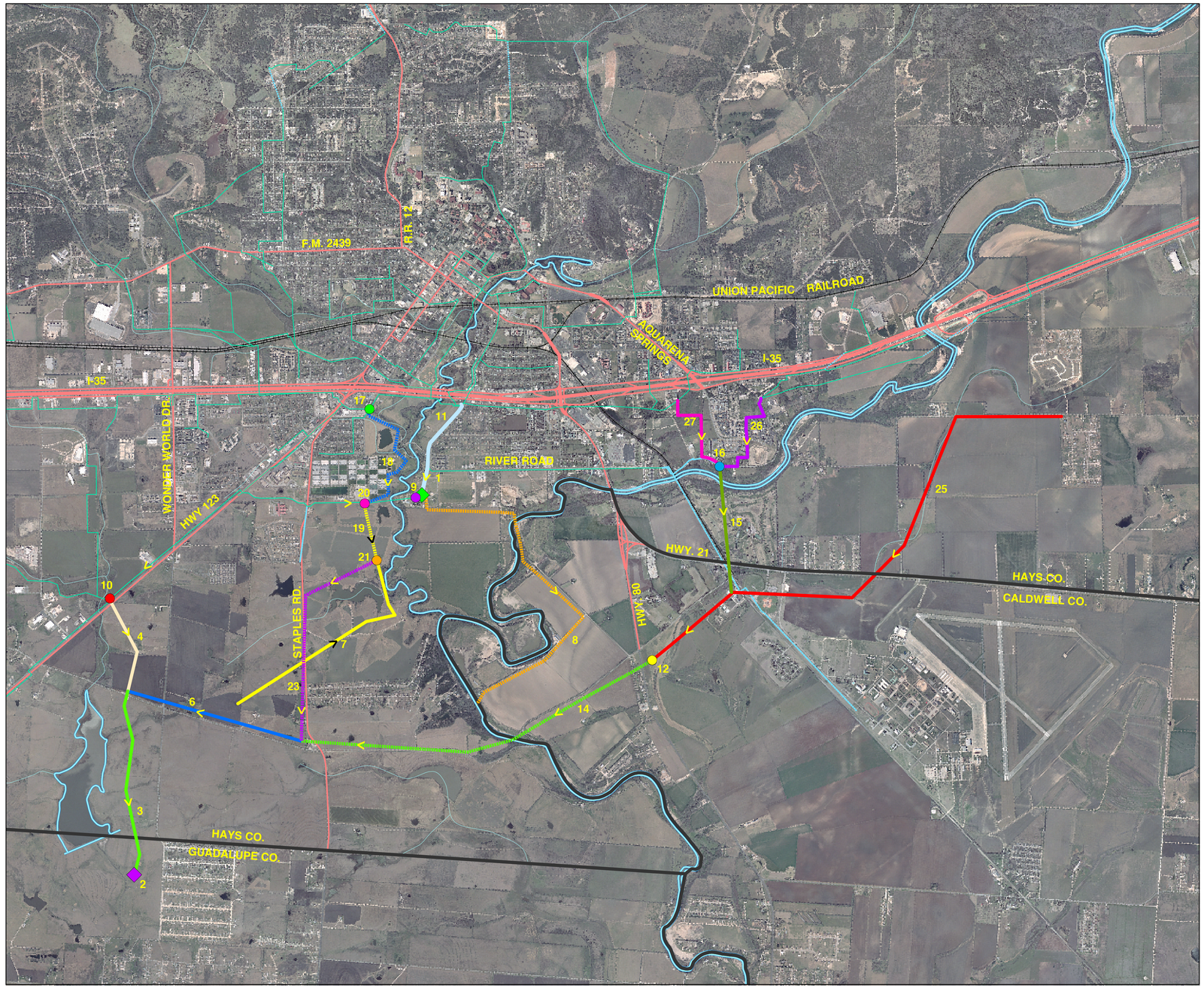

Legend

- 1. EXISTING W.W.T.P. EXPANSION (FOR PEAK FLOW)
- 2. COTTONWOOD CREEK W.W.T.P.
- 3. COTTONWOOD CREEK INTERCEPTOR - SOUTH OF OLD BASTROP HWY.
- 4: COTTONWOOD CK WWL BETWEEN OBH & CCLS
- 6. OLD BASTROP HWY/ FM621 WWL
- 7. SMITH TRACT WWL
- 8. NEW W.W.T.P. OUTFALL LINE
- 9. NEW EFFLUENT L.S. AT EXISTING W.W.T.P.
- 10. ABANDON COTTONWOOD CREEK L.S.
- 11. F.M. FROM MAIN L.S. TO EXIST. W.W.T.P.
- 12. SH21 L.S.
- 14. F.M. FROM SH21 L.S. TO OLD BASTROP HWY/ FM 621 WWL
- 15. F.M. FROM RIVER RD. L.S. TO BY PASS INTERCEPTOR
- 16. IMPROVEMENTS TO RIVER RD. L.S.
- 17. CAPE RD. L.S.
- 18. F.M. FROM CAPE RD. TO S.M. RIVER L.S.
- 19. F.M. FROM S.M. RIVER L.S. TO SMITH TRACT L.S.
- 20. INCREASE CAPACITY AT S.M. RIVER L.S.
- 21. SMITH TRACT L.S.
- 23. F.M. FROM SMITH TRACT L.S. TO OLD BASTROP HWY/ FM 621 WWL
- 25. BY PASS CREEK INTERCEPTOR
- 26. UHLAND ROAD INTERCEPTOR UPSIZE
- 27. OLD PLANT WASTEWATER LINE
- FUTURE WASTEWATER SYSTEM (F.M.) WITH CIP FROM SECTION 5
- FUTURE WASTEWATER SYSTEM (WWL) WITH CIP FROM SECTION 5
- COUNTY LINE
- HIGHWAYS
- CITY STREETS
- WATERWAYS
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FIGURE 6.2
WW TREATMENT DIVERSION ALTERNATIVE 2

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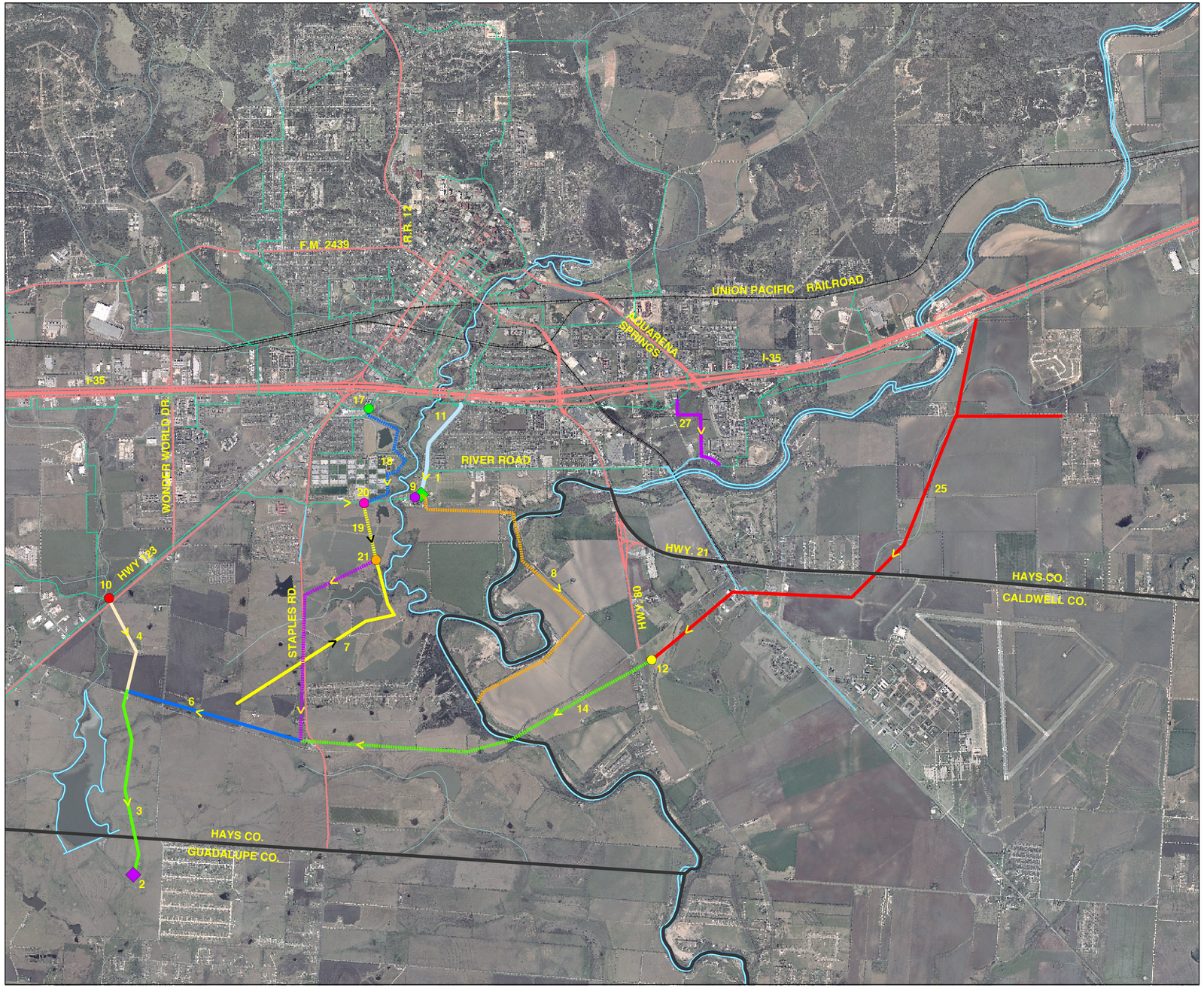

Legend

- 1. EXISTING W.W.T.P. EXPANSION (FOR PEAK FLOW)
- 2. COTTONWOOD CREEK W.W.T.P.
- 3. COTTONWOOD CREEK INTERCEPTOR - SOUTH OF OLD BASTROP HWY.
- 4: COTTONWOOD CK MAIN BETWEEN OBH & CCLS
- 12. SH21 L.S.
- 6. OLD BASTROP HWY/ FM 621 WWL
- 7. SMITH TRACT WWL
- 8. NEW W.W.T.P. OUTFALL LINE
- 9. NEW EFFLUENT L.S. AT EXISTING W.W.T.P.
- 10. ABANDON COTTONWOOD CREEK L.S.
- 11. F.M. FROM MAIN L.S. TO EXIST. W.W.T.P.
- 14. F.M. FROM SH21 L.S. TO OLD BASTROP HWY/ FM 621 OVERSIZE
- 17. CAPE RD. L.S.
- 18. F.M. FROM CAPE RD. TO S.M. RIVER L.S.
- 19. F.M. FROM S.M. RIVER L.S. TO SMITH TRACT L.S.
- 20. INCREASE CAPACITY AT S.M. RIVER L.S.
- 21. SMITH TRACT L.S.
- 23. F.M. FROM SMITH TRACT L.S. TO OLD BASTROP HWY/ FM 621 WWL
- 25. BY PASS CREEK INTERCEPTOR WITH DIVERSION FROM BLANCO RIVER L.S.
- 27. OLD PLANT WASTEWATER LINE
- FUTURE WASTEWATER SYSTEM (F.M.) WITH CIP FROM SECTION 5
- FUTURE WASTEWATER SYSTEM (WWL) WITH CIP FROM SECTION 5
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FIGURE 6.3
WW TREATMENT DIVERSION ALTERNATIVE 2A

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relief main can be avoided with Alternative 2 by constructing a gravity main from Old Plant lift station to River Road lift station (project #27), where the flow would be pumped to the By Pass Creek interceptor. The same flow diversion with Alternative 2A would also avoid the Linda Drive relief main, except in this case the flow would be pumped to the 24-inch River Road main. An increase in pumping capacity at River Road lift station would not be needed with Alternative 2A for the added Old Plant flow since the Blanco River lift station flow would be diverted from the River Road lift station to the By Pass Creek interceptor. A variation of the Old Plant lift station diversion for either Alternative 2 or 2A would be a force main to the 24-inch River Road main, instead of a gravity main to the River Road lift station, as shown in Figures 6.2 and 6.3. The Old Plant diversion possible with Alternative 2 offers considerable cost savings compared to the Linda Drive relief main required with Alternative 1, as shown in Table 6.2 below (and also included in Table 6.1).

TABLE 6.2. SUMMARY OF CIP AND COST DIFFERENCES OF THE CURRENT AND PROPOSED OPTIONS FOR THE OLD PLANT LIFT STATION IMPROVEMENTS

Old Plant CIP Improvements Currently Proposed for Alternative 2:			
Linda Drive			
Line Replacement Size	*LF*	*Cost per LF*	*Total Cost*
21 inch*	4,010	$ 166.00	$ 665,660.00
24 inch	1,390	$ 194.00	$ 269,660.00
	Mobilization:		$ 100,000.00
	Contingency & Design Fees (35% of Construction Costs):		$ 362,380.00
		TOTAL COST:	**$1,397,700.00**
Old Plant CIP Improvements Necessary with Diversion to River Road LS*:			
Gravity Line from Old Plant LS to River Road LS			
Line Size	*LF*	*Cost per LF*	*Total Cost*
12 inch	3,580	$ 41.00	$ 146,780.00
	Mobilization:		$ 100,000.00
	Contingency & Design Fees (35% of Construction Costs):		$ 86,320.00
		TOTAL COST:	**$ 333,100.00**

* Note that additional costs are included in Table 6.1 for the required pumping capacity increase at the River Road lift station and the construction of the southeast facilities to divert the River Road/Old Plant flows to the new treatment plant.

With these diversions, as well as the flow from the Cottonwood Creek Interceptor and the San Marcos River Lift Station, the new southeast treatment facility would receive 7.4 MGD of average dry weather flow and 19.5 MGD of peak wet weather flow based on a 5-year 6-hour design storm.

In Alternative 2, the existing River Road treatment plant would receive 7.5 MGD of average dry weather flow and 36.9 MGD of peak wet weather flow per the 2027 model. Since the existing facility is currently permitted for a peak flow of 18 MGD, this alternative would require expanding the capacity for peak flow by about 19 MGD. The permitted plant capacity would not be increased for average flow, but improvements such as final clarifiers, piping, and disinfection would be needed to increase hydraulic capacity for peak flows.

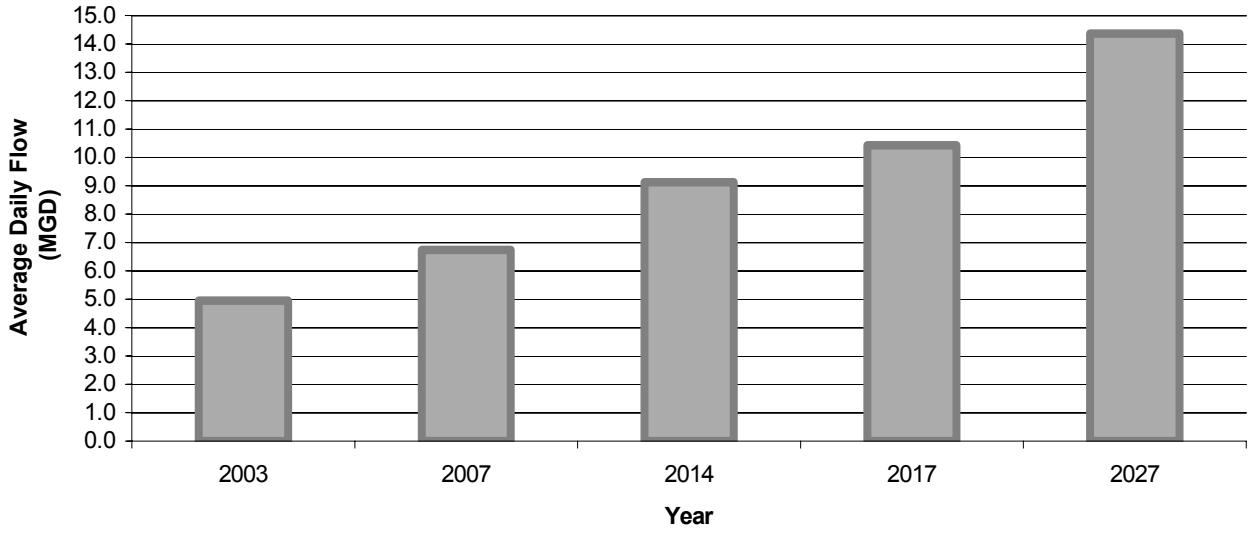
It might be noted that the ratio of projected wet weather flow to dry weather flow is higher for the River Road plant than for the Cottonwood Creek plant. This occurs because more of the 2027 flows to the new southeast plant comes from future development of new areas, which has modeling criteria of 750 gallon/day/acre for peak I/I. More of the flow to the River Road plant comes from existing development, which has higher I/I rates that were determined in the flow monitoring program.

Table 6.1 provides descriptions of the CIP projects necessary for the flow diversions associated with treatment Alternatives 1, 2, and 2A. Also provided in the table are the probable cost estimates associated with each CIP project based on the alternative. These estimates are provided as a means of comparison between the alternatives. The costs associated with expanding the existing plant in Alternative 1 to treat the average dry and peak wet weather flows and expanding the plant to treat higher peak wet weather flows and constructing a new facility in Alternatives 2 and 2A were not included, as it was assumed that cost to treat the flows would be about the same.

6.3 SYSTEM AVERAGE DAILY FLOW

The treatment alternatives outlined in the previous sections discuss the expansion of existing treatment facilities and the construction of new facilities. While the need for expansion to treat higher peak flows at the existing facilities is more prominent in these alternatives, average daily flow is also important in the design of components of a wastewater treatment plant. The current permitted average daily flow (maximum month) for the existing River Road treatment facility is 9.0 MGD and the facilities are rated to treat an average daily flow of 7.6 MGD. Figure 6.4 illustrates the increase in average daily flow over the study period. Judging by the projected steady increase in flow represented in Figure 6.4, the need for additional treatment capacity is imminent. TCEQ permit provisions require that planning of additional facilities should commence when average flow received at the plant reaches 75% of the permitted average flow, and construction of facilities should begin at 90%. A recent study on the River Road treatment plant was completed by CH2M Hill. The City will proceed with a decision on timing for future treatment capacity, at either the existing plant or a new Southeast plant, based on the findings of the CH2M Hill evaluation.

Figure 6.4
San Marcos Wastewater Collection System:
2003-2027 Average Daily Flow (MGD)



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7.0 WASTEWATER IMPACT FEE

The major scope of work for this wastewater master plan study is developing an updated CIP. A current CIP is a requirement for updating the impact fee (capital recovery fee) per service unit that is used to recoup costs for the capacity of CIP improvements that is required for new development. This section presents the data to support the wastewater impact fee update.

7.1 BACKGROUND

According to Chapter 395 of the Texas Statutes Local Government Code, *Financing Capital Improvements Required By New Development in Municipalities, Counties, and Certain Other Local Governments,* "impact fee means a charge or assessment imposed by a political subdivision against new development in order to generate revenue for funding or recouping the costs of capital improvements or facility expansions necessitated by and attributable to the new development." Preparation of the CIP and calculation of the impact fee must be based on (a) an analysis of the total capacity and level of usage of the existing system, (b) the costs of improvements to upgrade, expand or replace the existing system to meet existing needs, and (c) identification of the expansions and improvements and their costs attributable to providing capacity in planned improvements for new development in the service area.

Chapter 395 requires "the projected demand for capital improvements or facility expansions required by new service units projected over a reasonable period of time, not to exceed 10 years." The total capacity for proposed capital improvements herein is based on the projected flows in 25 years (for economy of scale in "$ per gpm capacity per inch-diameter" of new pipe and to avoid the disruption from construction of additional relief/replacement mains in only 10 years), but the capacity utilization for purposes of impact fee calculation is based on the difference between the projected flows in only 10 years versus existing flows.

Impact fees are charged to new development on a "per service unit" basis. By definition in Chapter 395, "service unit means a standardized measure of consumption, use, generation, or discharge attributable to an individual unit of development calculated in accordance with generally accepted engineering or planning standards and based on historical data and trends applicable to the political subdivision in which the individual unit of development is located during the previous 10 years." Since impact fees can apply to capital improvements for a range of functions (water supply, treatment, and distribution facilities; wastewater collection and treatment facilities; stormwater, drainage, and flood control facilities; and roadway facilities), the City is interested in developing a definition for a wastewater service unit that may be consistent with the service unit for other utilities, such as for water service, where a service unit has been defined for the water master plan by water meter size, which is related to water usage (and wastewater generation). Consequently, PBS&J is developing the proportionate cost of proposed capacity improvements that are attributable to new development in the next 10 years, and the City is investigating

the quantification for a wastewater service unit equivalent and wastewater impact fee per service unit equivalent.

7.2 COLLECTION SYSTEM CIP IMPROVEMENTS REQUIRED REGARDLESS OF TREATMENT EXPANSION ALTERNATIVES

The CIP schedule during the time period between 2003 and 2014 was examined and appropriate collection system projects were identified by the City for the impact fee development. A listing of the selected projects is provided in Table 7.1. The listing includes some projects that were in the previous CIP and which have been under construction or completed during the timeframe of this master plan update (i.e., new capacity required for new development in the next 10 years). Not included are CIP projects which primarily upgrade, expand or replace existing facilities serving existing development in order to meet safety, environmental or regulatory standards or provide better service (e.g., Rio Vista area rehab/replacement).

TABLE 7.1. LISTING OF CIP PROJECTS APPLICABLE IN THE IMPACT FEE DEVELOPMENT INDEPENDENT OF TREATMENT EXPANSION ALTERNATIVE

Impact Fee Development: CIP Projects
Craddock Avenue Wastewater Line Improvements
IH 35 Fairlawn/Blanco River Lift Station
North IH 35 Interceptor
Cottonwood Creek Interceptor/Cottonwood Creek Lift Station and Force Main/San Marcos River Lift Station
Cottonwood Creek Subdivision Offsite Water and Wastewater Extension Improvements
McCarty Lane Wastewater Line
Modified Riverside MKT Interceptor
Sink Creek Phase I
Eastside Interceptor Phase III
Purgatory Creek Interceptor
River Ridge Lift Station Improvements
Modified Sink Creek Phase II
Uhland Road Interceptor Upsize
Galisteo Ranch Oversize
Posey Road-Old Bastrop Highway Wastewater
Centerpoint Wastewater Improvements East of Outlet Malls

The costs of the identified CIP projects were quantified, and capacity utilization of the projects was evaluated with the model for peak wet weather flows under 2003 and 2014 conditions. The percent capacity utilization was developed based on the increase in utilization of capacity over the 10 year time

period, and the associated costs accrued over the impact fee period were totaled. Note that the capacity utilization and costs associated with the CIP projects correlating to each of the future treatment alternatives were evaluated separately for the impact fee period.

Table 7.2 summarizes the capacity utilized over the impact fee period and the total and utilized capacity costs associated with the CIP projects independent of the future treatment alternative chosen. The percentages of utilized capacity were developed based on the peak wet weather flow conditions of 2003 and 2014 occurring in response to the 5-year 6-hour design storm. Queries were performed in the hydraulic model to obtain the peak wet weather flows in the pipes furthest downstream of the CIP project, assuming that the pipe furthest downstream would be most representative of the total flow carried by the line under investigation. The percentage of capacity consumed over the impact fee period was calculated by comparing the peak wet weather flow in 2003 versus the peak wet weather flow in 2014.

With the exception of the River Ridge Lift Station CIP and the Modified Sink Creek I CIP projects, the cost estimates for the CIP projects in Table 7.2 were gathered from reports provided by the City for consistency with the previous impact fee study – Water and Wastewater Impact Fee Analysis 2002–2012 (City of San Marcos, 2002). The calculated percentages of capacity utilization were applied to these cost estimates, including an additional 110% for financing. Per Chapter 395, impact fees may recoup construction contract costs plus finance costs for payment of principal and interest on bonds issued to finance the CIP improvements, plus surveying and engineering fees and land acquisition costs. The CIP projects that are currently under construction or have been recently completed are listed last in Table 7.2. The utilized capacities of these projects under existing and 2014 conditions are provided in the table and with internal resources, the City will provide construction costs to approximate the costs associated with the capacity utilized over the impact fee period. Upon the addition of the construction costs of these projects and the necessary meter records and population data for calculation per SUE, the City will be able to turn to Table 7.2 to calculate impact fees per service unit (for the CIP projects independent of the treatment alternative).

7.3 CIP IMPROVEMENTS FOR TREATMENT EXPANSION ALTERNATIVES

The CIP projects required for flow diversion to each of the future treatment alternatives were evaluated separately for impact fee assessment. In general, average flows were the basis for allocation of treatment capacity costs to new development, and peak flows for allocation of costs for conveyance of flows to the treatment facilities.

Tables 7.3, 7.4, and 7.5 outline the capacity utilization of CIP projects and costs associated with diverting flow for future treatment alternatives 1, 2, and 2A, respectively. The utilized capacities of the CIP projects, with the exception of the treatment facility construction/expansion projects and related construction of outfall lines and lift stations, were determined by comparing the future peak wet weather

TABLE 7.2 BASIS FOR IMPACT FEE CALCULATION FOR ELIGIBLE CIP PROJECTS INDEPENDENT OF FUTURE TREATMENT ALTERNATIVE

Capacity Utilization and Probable Cost Estimates for CIP Projects Independent of Treatment Alternative										
CIP Project	Source of Cost Estimate	Estimated Cost of Construction[A]	Cost of Financing	Total Project Cost	Utilized Capacity (%)		During Impact Fee Period	Utilized Capacity ($)		During Impact Fee Period
					2003	2014		2003	2014	
Riverside MKT	COSM Water and Wastewater Impact Fee Analysis 2002-2012	$ 2,300,000	$ 2,530,000	$ 4,830,000	26%	30%	4%	$ 1,247,251	$ 1,436,356	$ 189,110
Sink Creek Phase I (Upsize 15" US of LS 6 to 27") & Modified Sink Creek Phase II	COSM Water and Wastewater Impact Fee Analysis 2002-2012 & Probable Cost Estimate Developed by PBS&J for the Modified Sink Creek I Project	$ 6,022,000	$ 6,624,200	$ 12,646,200	26%	85%	59%	$ 3,288,012	$ 10,749,270	$ 7,461,260
Eastside Interceptor Phase III	COSM Water and Wastewater Impact Fee Analysis 2002-2012	$ 1,000,000	$ 1,100,000	$ 2,100,000	10%	16%	6%	$ 210,000	$ 344,065	$ 134,060
Purgatory Creek Interceptor	COSM Water and Wastewater Impact Fee Analysis 2002-2012	$ 3,600,000	$ 3,960,000	$ 7,560,000	13%	19%	6%	$ 982,800	$ 1,412,598	$ 429,800
River Ridge Lift Station	Probable Cost Estimate Developed by PBS&J	$ 350,000	$ 385,000	$ 735,000	86%	94%	8%	$ 632,100	$ 688,123	$ 56,020
Uhland Road Interceptor Upsize	COSM Fiscal Year 2004-2013 CIP Projects	$ 2,500,000	$ 2,750,000	$ 5,250,000	37%	51%	14%	$ 1,960,571	$ 2,699,019	$ 738,450
Cottonwood Creek Interceptor Phase II: Cottonwood Creek LS Abandonment/ 18" Force Main Turnaround to Gravity	COSM Fiscal Year 2004-2013 CIP Projects	$ 6,600,000	$ 7,260,000	$ 13,860,000	0%	1%	1%	$ -	$ 138,600	$ 138,600
Galisteo Ranch Oversize	COSM Fiscal Year 2004-2013 CIP Projects	$ 100,000	$ 110,000	$ 210,000	0%	44%	44%	$ -	$ 92,833	$ 92,830
Posey Road- Old Bastrop Highway Wastewater	COSM Fiscal Year 2004-2013 CIP Projects	$ 625,000	$ 687,500	$ 1,312,500	0%	15%	15%	$ -	$ 201,486	$ 201,490
Centerpoint WW East of Outlet Malls	COSM Fiscal Year 2004-2013 CIP Projects	$ 225,000	$ 247,500	$ 472,500	0%	5%	5%	$ -	$ 24,607	$ 24,610
Craddock Avenue Improvements	COSM Project Records	***$ 422,490.20***	$ 464,739	$ 887,229	43%	55%	12%	$ 383,096	$ 491,178	$ 108,082
I-35 Fairlawn/ Blanco River LS	COSM Project Records	***$ 2,724,900.64***	$ 2,997,391	$ 5,722,291	2%	10%	8%	$ 117,782	$ 590,216	$ 472,434
North I-35 Interceptor	COSM Project Records	$ 32,518.61	$ 35,770	$ 68,289	3%	15%	12%	$ 1,967	$ 10,404	$ 8,437
Cottonwood Creek Interceptor (Lines B, C, and E)	COSM Project Records	$ 2,108,872.00	$ 2,319,759	$ 4,428,631	6%	17%	11%	$ 279,677	$ 761,076	$ 481,398
McCarty Lane WW Line	COSM Project Records	***$ 396,951.50***	$ 436,647	$ 833,598	0%	7%	7%	$ -	$ 61,198	$ 61,198
10-year CIP Total Independent of Future Treatment Alternative:										**$ 10,597,779**

[A]: Bold italicized costs reflect construction costs provided by COSM plus an additional 10% to account for engineering and right-of-way fees.

TABLE 7.3. CAPACITY UTILIZATION AND PROBABLE COST OF CIP PROJECTS ASSOCIATED WITH TREATMENT ALTERNATIVE 1

DESCRIPTIONS AND ESTIMATED PROBABLE COSTS: PROJECTS REQUIRED FOR THE ADDITIONAL WASTEWATER TREATMENT ALTERNATIVE 1

Alternative 1:
Take all flow to existing WWTP, incl Cottonwood Ck; expand plant for 14.9/56.4 yr2027 flows; pump effluent to new outfall d/s of confluence & dam.

Project ID	LOCATION	Estimated Cost of Construction	Cost of Financing	Total Project Cost	% Utilized 2003	% Utilized 2014	% Utilized During Impact Fee Period	Utilized Capacity ($) 2003	Utilized Capacity ($) 2014	During Impact Fee Period
1*	**Existing WWTP:** Expand to 14.9 avg/56.4 peak	$ 26,800,000	$ 29,480,000	$ 56,280,000	34%	61%	27%	$ 19,135,200	$ 34,330,800	$ 15,195,600
2*	**Future Cottonwood Ck WWTP:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
3	**Cottonwood Ck Interceptor south of Old Bastrop Hwy:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
4	**Cottonwood Ck main between OBH and CCLS:** 8" Force Main convey flow from OBH to CCLS	$ 126,000	$ 138,600	$ 264,600	0%	20%	20%	$ -	$ 52,920	$ 52,920
NA	**Cottonwood Ck Lines A & D:** Use existing 18" FM ; Use existing 24" WWL	$ 887,458	$ 976,204	$ 1,863,662	0%	70%	70%	$ -	$ 1,304,563	$ 1,304,563
5	**Linda Drive Improvements:** Upsize Existing WWL from 18" to 21" and 24"	NA	NA	NA	NA	NA	NA	NA	NA	NA
6	**CIP #21** - Old Bastrop Hwy:** 8" WWL and 0.5 MGD Lift Station to Pump Flow from OBH to the CCLS	$ 1,020,500	$ 1,122,550	$ 2,143,050	0%	1%	1%	$ -	$ 26,788	$ 26,790
7	**CIP #22** - Smith Tract WW line:** 12" WWL and 4" FM to SMRLS	$ 441,900	$ 486,090	$ 927,990	0%	7%	7%	$ -	$ 60,521	$ 60,520
8*	**New WWTP outfall line:** 48" FM	$ 3,150,000	$ 3,465,000	$ 6,615,000	34%	61%	27%	$ 2,249,100	$ 4,035,150	$ 1,786,050
9*	**New lift station at existing WWTP:** Effluent Lift Station with Capacity to Pump 56.38 MGD	$ 3,500,000	$ 3,850,000	$ 7,350,000	34%	61%	27%	$ 2,499,000	$ 4,483,500	$ 1,984,500
10	**Abandon Cottonwood Ck LS:** Do Not Abandon	NA	NA	NA	NA	NA	NA	NA	NA	NA
11*	**Force Mains from Main LS to Existing WWTP:** Add'tl 48" FM needed for capacity of 48.49 MGD	$ 891,000	$ 980,100	$ 1,871,100	34%	61%	27%	$ 636,174	$ 1,141,371	$ 505,200
12	**SH 21 Lift Station:** Peak Flow = 0.5 MGD	$ 400,000	$ 440,000	$ 840,000	0%	90%	90%	$ -	$ 756,000	$ 756,000
13	**Force Main from SH 21 LS to Existing WWTP:** 4" FM	$ 270,000	$ 297,000	$ 567,000	0%	90%	90%	$ -	$ 510,300	$ 510,300
14	**Force Main from SH 21 LS to CIP #21-OBH:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
15	**Force Main from River Road LS to By Pass Interceptor:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
16	**Improvements to River Road LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
17	**Cape Road LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
18	**Force Main from Cape Road LS to SMRLS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
19	**Force Main from SMRLS to Smith Tract LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
20	**Increase Capacity at San Marcos River LS:** Improve to meet capacity of Peak Flow = 7.9 MGD	$ 225,000	$ 247,500	$ 472,500	0%	66%	66%	$ -	$ 309,836	$ 309,840
21	**Smith Tract LS (CIP #22):** Lift Station to handle Peak Flow = 0.2 MGD	$ 500,000	$ 550,000	$ 1,050,000	0%	71%	71%	$ -	$ 750,000	$ 750,000
22	**Force Mains from SMRLS to WWTP:** Add'tl 24" FM needed for capacity of 7.9 MGD	$ 232,800	$ 256,080	$ 488,880	0%	66%	66%	$ -	$ 320,577	$ 320,580
23	**Force Main from Smith Tract LS to #21 WWL:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
24	**Increase Capacity at Cottonwood Creek LS:** Add'tl Capacity needed to handle Peak Flow = 7.1 MGD	$ 200,000	$ 220,000	$ 420,000	0%	71%	71%	$ -	$ 297,549	$ 297,550
25	**By-Pass Creek Interceptor:** 8" and 12" WWL	$ 834,700	$ 918,170	$ 1,752,870	0%	7%	7%	$ -	$ 122,701	$ 122,701
26	**Uhland Road Interceptor:** Upsize existing 24" with 30"	$ 583,200	$ 641,520	$ 1,224,720	23%	31%	8%	$ 281,686	$ 379,663	$ 97,978
27	**Old Plant LS Diversion - WWL to River Road LS**	NA	NA	NA	NA	NA	NA	NA	NA	NA
	TOTALS:	$ 40,062,558	$ 44,068,814	$ 84,131,372				$ 24,801,160	$ 48,882,240	**$ 23,860,413**

*: Capacity utilization of treatment is based on average daily flow.
**: COSM CIP for 2002-2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines

TABLE 7.4. CAPACITY UTILIZATION AND PROBABLE COST OF CIP PROJECTS ASSOCIATED WITH TREATMENT ALTERNATIVE 2

DESCRIPTIONS AND ESTIMATED PROBABLE COSTS: PROJECTS REQUIRED FOR THE ADDITIONAL WASTEWATER TREATMENT ALTERNATIVE 2

Alternative 2:
Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek. North flow diverted via River Road LS.

Project	LOCATION	Estimated Cost of Construction	Cost of Financing	Total Project Cost	% Utilized 2003	% Utilized 2014	% Utilized During Impact Fee Period	Utilized Capacity ($) 2003	Utilized Capacity ($) 2014	During Impact Fee Period
1*	**Existing WWTP:** Maintain 7.6 avg/ increase capacity for peak 36.8 MGD	$ 18,800,000	$ 20,680,000	$ 39,480,000	54%	68%	14%	$ 10,152,000	$ 26,846,400	$ 5,527,200
2*	**Future Cottonwood Ck WWTP:** Build 7.4 avg/19.5 peak WWTP	$ 27,121,300	$ 29,833,430	$ 56,954,730	12%	53%	41%	$ 3,254,556	$ 30,186,007	$ 23,351,440
3	**Cottonwood Ck Interceptor south of Old Bastrop Hwy:**42" WWL Gravity Line to C'wood Plant	$ 1,289,250	$ 1,418,175	$ 2,707,425	0%	67%	67%	$ -	$ 1,804,950	$ 1,804,950
4	**Cottonwood Ck main between OBH and CCLS:** 30" WWL Gravity line flow from CCLS to OBH	$ 461,000	$ 507,100	$ 968,100	0%	60%	60%	$ -	$ 580,860	$ 580,860
NA	**Cottonwood Ck Lines A & D:** Reverse Flow and utilize existing 18" FM as gravity; Use existing 24" WWL	$ 887,458	$ 976,204	$ 1,863,662	0%	5%	5%	$ -	$ 93,183	$ 93,183
5	**Linda Drive Improvements:** Upsize Existing WWL from 18" to 21" and 24"	NA	NA	NA	NA	NA	NA	NA	NA	NA
6	**CIP #21** - Old Bastrop Hwy:** 27" WWL	$ 706,000	$ 776,600	$ 1,482,600	0%	48%	48%	$ -	$ 714,344	$ 714,340
7	**CIP #22** - Smith Tract WW line:** 12" WWL and 4" FM to SMRLS	$ 344,400	$ 378,840	$ 723,240	0%	7%	7%	$ -	$ 47,168	$ 47,170
8*	**New WWTP outfall line:** 42" FM	$ 2,674,000	$ 2,941,400	$ 5,615,400	54%	68%	14%	$ 3,032,316	$ 3,818,472	$ 786,160
9*	**New lift station at existing WWTP:** Effluent Lift Station with Capacity to Pump 36.84 MGD	$ 2,800,000	$ 3,080,000	$ 5,880,000	54%	68%	14%	$ 3,175,200	$ 3,998,400	$ 823,200
10	**Abandon Cottonwood Ck LS:** Abandon Lift Station	$ 20,000	$ 22,000	$ 42,000	-	-	-	-	-	-
11*	**Force Mains from Main LS to Existing WWTP:** Add'tl 42" FM needed for capacity of 36.84 MGD	$ 756,360	$ 831,996	$ 1,588,356	54%	68%	14%	$ 857,712	$ 1,080,082	$ 222,370
12	**SH 21 Lift Station:** Peak Flow 5.1 MGD	$ 900,000	$ 990,000	$ 1,890,000	0%	84%	84%			
13	**Force Main from SH 21 LS to Existing WWTP:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
14	**Force Main from SH 21 LS to CIP #21-OBH:** 18" FM	$ 489,600	$ 538,560	$ 1,028,160	0%	75%	75%	$ -	$ 773,355	$ 773,360
15	**Force Main from River Road LS to By Pass Interceptor:** 15" FM	$ 716,100	$ 787,710	$ 1,503,810	0%	84%	84%	$ -	$ 1,263,200	$ 1,263,200
16	**Improvements to River Road LS:** Increase firm pumping capacity to 4.6 MGD	$ 250,000	$ 275,000	$ 525,000	0%	75%	75%	$ -	$ 393,750	$ 393,750
17	**Cape Road LS:** Corner of Cape Road and Luciano - LS w/ capacity to divert Eastside II and III flows to new facility (Peak Flow = 6.5 MGD)	$ 900,000	$ 990,000	$ 1,890,000	0%	86%	86%	$ -	$ 1,627,904	$ 1,627,900
18	**Force Main from Cape Road LS to SMRLS:** 18" FM	$ 277,440	$ 305,184	$ 582,624	0%	86%	86%	$ -	$ 501,057	$ 501,060
19	**Force Main from SMRLS to Smith Tract LS:** 18" FM	$ 265,200	$ 291,720	$ 556,920	0%	84%	84%	$ -	$ 467,482	$ 467,480
20	**Increase Capacity at San Marcos River LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
21	**Smith Tract LS (CIP #22):** Lift Station to handle Peak Flow = 7.4 MGD	$ 900,000	$ 990,000	$ 1,890,000	0%	84%	84%	$ -	$ 1,586,478	$ 1,586,480
22	**Force Mains from SMRLS to WWTP:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
23	**Force Main from Smith Tract LS to #21 WWL:** 24" FM	$ 408,000	$ 448,800	$ 856,800	0%	84%	84%	$ -	$ 719,203	$ 719,200
24	**Increase Capacity at Cottonwood Creek LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
25	**By-Pass Creek Interceptor:** 8" and 12" WWL	$ 834,700	$ 918,170	$ 1,752,870	0%	7%	7%	$ -	$ 122,701	$ 122,701
26	**Uhland Road Interceptor:** 24" and 30" WWL	$ 583,200	$ 641,520	$ 1,224,720	23%	31%	8%	$ 281,686	$ 379,663	$ 97,978
27	**Old Plant LS Diversion - WWL to River Road LS**	NA	NA	NA	NA	NA	NA	NA	NA	NA
	TOTAL:	$ 62,384,008	$ 68,622,409	$ 131,006,417				$ 20,753,470	$ 77,004,659	**$ 41,283,303**

*: Capacity utilization of treatment is based on average daily flow.

**: COSM CIP for 2002-2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines

TABLE 7.5. CAPACITY UTILIZATION AND PROBABLE COST OF CIP PROJECTS ASSOCIATED WITH TREATMENT ALTERNATIVE 2A

DESCRIPTIONS AND ESTIMATED PROBABLE COSTS: PROJECTS REQUIRED FOR THE ADDITIONAL WASTEWATER TREATMENT ALTERNATIVE 2A

Alternative 2A:
Maintain existing 7.6/increase capacity for peak 36.8 MGD WWTP; Divert all other flow (7.4/19.5 MGD) to new facility on Cottonwood Creek. North flow diverted via Blanco River LS.

Project	LOCATION	Estimated Cost of Construction	Cost of Financing	Total Project Cost	% Utilized 2003	% Utilized 2014	% Utilized During Impact Fee Period	Utilized Capacity ($) 2003	Utilized Capacity ($) 2014	During Impact Fee Period
1*	**Existing WWTP:** Maintain 7.6 avg/ increase capacity for peak 36.84 MGD	$ 18,800,000.00	$ 20,680,000.00	$ 39,480,000.00	54%	68%	14%	$ 10,152,000.00	$ 26,846,400.00	$ 5,527,200
2*	**Future Cottonwood Ck WWTP:** Build 7.4 avg/19.5 peak WWTP	$ 27,121,300.00	$ 29,833,430.00	$ 56,954,730.00	12%	53%	41%	$ 3,254,556.00	$ 30,186,006.90	$ 23,351,439
3	**Cottonwood Ck Interceptor south of Old Bastrop Hwy:** 42" WWL Gravity Line to C'wood Plant	$ 1,289,250.00	$ 1,418,175.00	$ 2,707,425.00	0%	67%	67%	$ -	$ 1,804,950.00	$ 1,804,950
4	**Cottonwood Ck main between OBH and CCLS:** 30" WWL Gravity line flow from CCLS to OBH	$ 461,000.00	$ 507,100.00	$ 968,100.00	0%	60%	60%	$ -	$ 580,860.00	$ 580,860
NA	**Cottonwood Ck Lines A & D:** Reverse Flow and utilize existing 18" FM as gravity; Use existing 24" WWL	$ 887,458.00	$ 976,203.80	$ 1,863,661.80	0%	5%	5%	$ -	$ 93,183.09	$ 93,183
5	**Linda Drive Improvements:** Upsize Existing WWL from 18" to 21" and 24"	NA	NA	NA	NA	NA	NA	NA	NA	NA
6	**CIP #21** - Old Bastrop Hwy:** 27" WWL	$ 706,000.00	$ 776,600.00	$ 1,482,600.00	0%	48%	48%	$ -	$ 714,343.64	$ 714,344
7	**CIP #22** - Smith Tract WW line:** 12" WWL and 4" FM to SMRLS	$ 344,400.00	$ 378,840.00	$ 723,240.00	0%	7%	7%	$ -	$ 47,167.83	$ 47,168
8*	**New WWTP outfall line:** 42" FM	$ 2,674,000.00	$ 2,941,400.00	$ 5,615,400.00	54%	68%	14%	$ 3,032,316.00	$ 3,818,472.00	$ 786,156
9*	**New lift station at existing WWTP:** Effluent Lift Station with Capacity to Pump 36.8 MGD	$ 2,800,000.00	$ 3,080,000.00	$ 5,880,000.00	54%	68%	14%	$ 3,175,200.00	$ 3,998,400.00	$ 823,200
10	**Abandon Cottonwood Ck LS:** Abandon Lift Station	$ 20,000.00	$ 22,000.00	$ 42,000.00	-	-	-	-	-	-
11*	**Force Mains from Main LS to Existing WWTP:** Add'tl 42" FM needed for capacity of 36.8 MGD	$ 756,360.00	$ 831,996.00	$ 1,588,356.00	54%	68%	14%	$ 857,712.24	$ 1,080,082.08	$ 222,370
12	**SH 21 Lift Station:** Peak Flow 5.1 MGD	$ 900,000.00	$ 990,000.00	$ 1,890,000.00	0%	84%	84%	$ -	$ 1,587,600.00	$ 1,587,600
13	**Force Main from SH 21 LS to Existing WWTP:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
14	**Force Main from SH 21 LS to CIP #21 OBH:** 18" FM	$ 489,600.00	$ 538,560.00	$ 1,028,160.00	0%	75%	75%	$ -	$ 773,355.13	$ 773,355
15	**Force Main from River Road LS to By Pass Interceptor:** 15" FM	$ 716,100.00	$ 787,710.00	$ 1,503,810.00	0%	84%	84%	$ -	$ 1,263,200.40	$ 1,263,200
16	**Improvements to River Road LS:** Increase firm pumping capacity to 4.6 MGD	$ 250.00	$ 275.00	$ 525.00	0%	75%	75%	$ -	$ 393.75	$ 394
17	**Cape Road LS:** Corner of Cape Road and Luciano - LS w/ capacity to divert Eastside II and III flows to new facility (Peak Flow = 6.5 MGD)	$ 900,000.00	$ 990,000.00	$ 1,890,000.00	0%	86%	86%	$ -	$ 1,627,904.47	$ 1,627,904
18	**Force Main from Cape Road LS to SMRLS:** 18" FM	$ 277,440.00	$ 305,184.00	$ 582,624.00	0%	86%	86%	$ -	$ 501,056.64	$ 501,057
19	**Force Main from SMRLS to Smith Tract LS:** 18" FM	$ 265,200.00	$ 291,720.00	$ 556,920.00	0%	84%	84%	$ -	$ 467,482.11	$ 467,482
20	**Increase Capacity at San Marcos River LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
21	**Smith Tract LS (CIP #22):** Lift Station to handle Peak Flow = 7.41 MGD	$ 900,000.00	$ 990,000.00	$ 1,890,000.00	0%	84%	84%	$ -	$ 1,586,477.73	$ 1,586,478
22	**Force Mains from SMRLS to WWTP:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
23	**Force Main from Smith Tract LS to #21 WWL:** 24" FM	$ 408,000.00	$ 448,800.00	$ 856,800.00	0%	84%	84%	$ -	$ 719,203.24	$ 719,203
24	**Increase Capacity at Cottonwood Creek LS:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
25	**By-Pass Creek Interceptor:** 8", 18", 21" and 24" WWL	$ 834,700.00	$ 918,170.00	$ 1,752,870.00	0%	31%	31%	$ -	$ 543,389.70	$ 543,390
26	**Uhland Road Interceptor:** NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
27	**Old Plant LS Diversion - WWL to River Road LS**	NA	NA	NA	NA	NA	NA	NA	NA	NA
	TOTAL:	$ 61,551,058.00	$ 67,706,163.80	$ 129,257,221.80				$ 20,471,784.24	$ 78,239,928.70	**$ 42,477,543**

*: Capacity utilization of treatment is based on average daily flow.

**: COSM CIP for 2002-2012 budgeted for the cost of upsizing these projects, but the estimations seen here are for initial construction of lines

flows of the year 2014 to the peak wet weather flows occurring in 2027. The CIP projects for each treatment alternative were sized to convey their associated peak wet weather flows generated in 2027. The capacity utilization for the treatment facility and related projects was assessed using the 2014 average dry weather flow to the facilities and the ultimate average dry weather flow capacities. For the line work associated with the treatment alternative CIP, the appropriate pipes were evaluated for peak flow occurring under 2014 conditions and the flows were assessed for capacity utilization using the future maximum capacity of the projects.

The probable cost estimates developed for the CIP projects were based on recent bid tabs provided by the City of San Marcos and gathered from comparable municipalities for similar projects. The cost of expanding the existing facility in treatment alternative 1 and the cost of constructing a new facility in Alternatives 2 and 2A were based on a costs developed from the estimates included in the City's 2002–2012 Impact Fee. The $3.67 treatment cost per gallon developed from the report is based on dry weather flow. In Alternatives 2 and 2A, the cost required to expand the existing facilities to treat a higher peak flow was calculated using an estimate of $1 per additional gallon of capacity required to treat a peak weather flow of 36.84 MGD. Based on information provided from the City, it was assumed that the current peak wet weather capacity at the existing facility is 18 MGD.

Based on the treatment alternative chosen, Table 7.3, Table 7.4, and/or Table 7.5 can be used to determine additional costs eligible for calculating impact fees to be assessed to new customers of the San Marcos system.