TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

A. PURPOSE

The purpose of this report was to establish a 20-year master plan for the development of wastewater facilities for the City of Raymore, Missouri. The Master Plan will be used to coordinate the efforts of the City towards scheduling, budgeting, planning, designing and constructing improvements to the existing wastewater infrastructure to accommodate the projected growth in the City of Raymore.

B. SCOPE

The scope of the Master Plan included the following:

1. Conducting a limited review of the existing facilities including the collection system, pumping stations and interceptors connecting to the Little Blue Valley Sewer District.

2. Performing an evaluation of existing population and land development data, and the development of population growth projections for the existing service area as well as three additional areas (Expansion areas A, B, & C) of projected growth for the next 5, 10, 15 and 20-year periods.

3. Developing a hydraulic model of selected sewer interceptors to estimate flows for the growth projections listed above under both dry weather and wet weather flow conditions. The following sewer interceptors were selected for modeling:

    - Good Ranch West (GRW) Relief Sewer from Brookside subdivision south to the Owen-Good Pump Station
    - West Interceptor from Stonegate of the Good Ranch subdivision south to the Owen-Good Pump Station.
    - Good Ranch East (GRE) Interceptor and relief line from the south side of Canter Ridge and Evan-Brook subdivisions south to the Owen-Good Pump Station.
    - GRW Relief Interceptor from south of the Cedar Ridge subdivision southwest to the Owen-Good Pump Station
    - Alexander Creek Interceptor from the east side of the Raymore Development Park east to the Winnebago Pump Station.

- Lampkins Fork Interceptor from the north side of Wedgewood Meadows north to the northern city limit.

Figure IN-1 shows the location of the interceptors that were modeled.

4. Using the results from the model, the improvements and new construction needed to continue to operate the collection system for the next 20-years, under dry and wet weather conditions.

5. Using the results from the model, the future requirements for the existing Owen-Good Pump Station for the conditions and scenarios listed previously were determined.

6. Conducting an economic evaluation of providing treatment for customers outside of the Little Blue Valley Sewer District.

7. Conducting an evaluation of the effects of infiltration & inflow (I&I) reduction on the proposed future infrastructure needs.

8. Developing an opinion of probable cost for the recommended improvements as well as a phased schedule and prioritization for the improvements.

9. In addition to the Master Plan, the City has requested a review of their design standards and specifications. A set of reviewed and updated documents is included as a separate section in Appendix A.

C. OPINIONS OF COST

The opinions of probable cost provided in this report are based primarily on our experience and judgement as a professional consulting firm combined with information from vendors and published sources. For pipeline costs an average bury dept of 15-feet with the last 5-feet being in rock has been assumed. Geotechnical validation of this assumption is beyond the scope of this report. Rock characteristics and excavation techniques can vary significantly and therefore can have a significant impact on final cost. Burns & McDonnell has no control over weather, cost, and availability of labor, material and equipment, labor productivity, construction contractor's procedures and methods, unavoidable delays, construction contractor's methods of determining prices, economic conditions, government regulations and laws (including the interpretation thereof), competitive bidding or market conditions and other factors affecting such opinions or projections; consequently, the final project costs will vary from the opinion of costs provided in this report, and funding needs must be carefully reviewed prior to making specific financial decisions or establishing final budgets.

The opinion of costs for all items of equipment and hardware included in the improvements and upgrades typically include equipment, installation, electrical and mechanical systems, and instrumentation. For the purpose of this report it has been assumed that three phase power will be available at MO-291 HWY and relocated MO-58 HWY and at US-71 HWY.   Cost opinions do not include, sales taxes, wastewater characterization, treatability testing, pilot testing, special construction requirements, acquisition of easements, removal and disposal of hazardous materials and waste, extraordinary legal expenses, or other non-construction costs. An allowance of 20% has been made for ordinary legal, administrative and engineering costs.

* * * * *

PART I – EXISTING FACILITIES

A. GENERAL

This section of the report provides an overview of the existing wastewater facilities utilized by the City of Raymore. The City of Raymore lies within Cass County and is bounded by Kansas City on the north, The City of Belton on the west, The City of Peculiar to the South and unincorporated areas of Cass County to the east. All of the wastewater generated by the City of Raymore is currently conveyed to the Little Blue Valley Sewer District's collection system for treatment and disposal.

B. BACKGROUND

Wastewater collection and pumping facilities for the City of Raymore, Missouri include gravity sewers, force mains, a primary pump station (Owen-Good Pump Station), and two smaller pump stations. The service area contains three major watersheds (Lampkins Fork, Alexander Creek and Good Ranch). The collection system facilities, along with the watershed boundaries, city limits and limits of the existing sewer service area are shown on Figure I-1. The Lampkins Fork watershed (Service Area 4), which exists roughly north of Highway 58, gravity drains to the Lampkins Fork Interceptor, which conveys flow north to the Little Blue Valley Sewer District. The Alexander Creek watershed (Service Areas 1, 2, and 3) drains flow east to the Alexander Creek Interceptor. This interceptor also connects to the Little Blue Valley Sewer District at the Raintree Lake Pump Station. The Good Ranch watershed (Service Areas 5 through 11), generally south of Highway 58, gravity drains to the Owen-Good Pump Station, where wastewater is pumped north across the watershed divide to the Lampkins Fork interceptor.

These three watersheds contain the Existing Service Area (ESA), which is subdivided into eleven Service Areas (SAs). The ESA consists of the large interceptor sewers, collector sewers and lateral sewers that collect wastewater. These pipes carry the wastewater generated by the residential and commercial customers to treatment facilities operated by the Little Blue Valley Sewer District. Not all of the Service Areas within the ESA can be serviced entirely by gravity sewers, due to the local topography of the service area. Pump stations are used to transfer the wastewater from these areas to adjacent watersheds or interceptor sewers that convey wastewater by gravity to treatment facilities.

All wastewater generated by the City of Raymore ultimately flows to the Atherton Wastewater Treatment Facility operated by the Little Blue Valley Sewer District. The District is operating under an agreement to accept wastewater from the City of Raymore for treatment and disposal. The City is billed quarterly by the District by multiplying the percent of wastewater treated that originated from Raymore by the total quarterly operating costs and debt service of the District. Due to the relatively low wastewater flows in the City's collection system and the available capacity at the District's facilities, the construction and operation of a City-owned wastewater treatment plant historically has not been considered financially desirable. This issue will be evaluated as a component of this master plan.

1. COLLECTION SYSTEM

The existing service area for the City currently includes approximately 14,600 acres of developed and undeveloped land including areas inside the City limits and some areas outside and generally east of the City. The western boundary of the service area generally runs north-south, beginning at Kentucky Road and 155th Street and running south to Foxwood Drive / MO-58 HWY. At that point the western boundary follows US-71 HWY from Foxwood Drive / MO-58 HWY to the southern boundary just south of 195th Street. The southern boundary begins at US-71 HWY just south of 195th Street and runs generally northeast until it reaches the southern end of Lake Winnebago. The area boundary then follows the western edge of the lake and then runs southwest to approximately the City limits at Prairie Lane. The boundary then turns northwest to approximately the intersection of Gore Road and Lincoln Road, runs north to 155th Street, and continues west to the intersection of 155th Street and Kentucky Road.

The collection system within the existing service area is composed of 88 miles of pipe, mainly gravity sewer lines made of polyvinyl chloride (PVC), reinforced concrete (RCP) or vitrified clay pipe (VCP). The southern end of the service area includes the Owen-Good Pump Station and the 24-inch force main that runs from the pump station (see next section) to a 30-inch interceptor sewer just south of the intersection of Foxwood Drive and Sunset Lane. Based on the most recent survey of available data, the collection system consists of the following:

- 5,700 linear feet of 4-inch pipe
- 334,000 linear feet of 8-inch pipe
- 3,900 linear feet of 10-inch pipe
- 14,500 linear feet of 12-inch pipe
- 40,700 linear feet of 15-inch pipe
- 17,500 linear feet of 18-inch pipe

- 2,900 linear feet of 21-inch pipe
- 31,500 linear feet of 24-inch pipe, including 16,300 feet of force main from the Owen-Good Pump Station
- 5,700 linear feet of 30-inch pipe
- 6,800 linear feet of 42-inch pipe
- Approximately 1,818 brick or concrete manholes

2. PUMP STATIONS

The Owen-Good Pump Station, which is Raymore's only major pump station, collects wastewater flows from service sub-areas 5,6,7,8,9,10 and 11 and discharges into service sub-area 4 near Highway 58 at the terminus of the Lampkins Fork Interceptor. From this point, it is carried by gravity to the north boundary of the service area and into the Little Blue Valley Sewer District's collection system. The pump station site includes the pump station wet well, a valve vault for each pump, an electrical control building, an emergency generator, an odor-control building, and a 2,000,000 gallon peak flow storage basin.

A cast-in-place concrete wet well contains a main wet well with a trash rack mounted at the influent end of the well. Three 20-inch slide gates allow access to individual wet well volumes for each of the three identical submersible centrifugal pumps at the station. Each pump is driven by a 175 HP, 1750 RPM motor. The firm capacity of the station is reported to be 3,650 gpm (5.25 mgd) at 191 feet of head when calculated at the wet well set point. A 24-inch overflow line in the main well allows peak flows beyond the pumping capacity of the station to flow to an earthen holding basin location adjacent to the pump station structure. Table I-1 provides basic performance information on the centrifugal pumps. Figure I-2 shows the system and pump curves for the Owen-Good Pump Station, while Figure I-3 shows the existing configuration of the Owen-Good Pump Station.

Table I-1

Owen-Good Pump Station Data

Pump Combination	Design Flow (GPM)	Design Head (FT)
1 Pump	2,200	178
2 Pumps in Parallel	3,650	191
3 Pumps in Parallel	4,500	201

Each 12-inch pump discharge line passes through a 6-foot diameter concrete manhole section, which acts as a valve vault. A check valve and a gate valve are installed on each line, within the vault. The three 12-inch lines tie into a 24-inch force main that conveys wastewater north into the Lampkins Fork Interceptor.

The electrical control building contains the motor starters, variable frequency drives, control logic, motor overload protection and auxiliary electrical equipment for the proper operation of the pump station. The pumps are operated based on the liquid level in the wells, which is measured by pressure-transducers that are monitored from the electrical control building. Normally, two pumps are run at the same time, with the third pump acting as a standby unit. However, the third pump can operate, if available, in the event of high flows at the station. The control logic alternates the use of the pumps so that each pump is utilized in an equal fashion. An emergency generator is located at the site. In case of a power outage, the generator will automatically provide temporary power to the pump station.

If the peak wastewater flow rate into the pump station exceeds its pumping capacity and the wet well becomes completely filled, excess flow discharges through a 24-inch DIP overflow line to an open storage basin. This basin can provide up to 2 million gallons of storage, which would drain back into the wet well of the pump station after the peak flow event had subsided.

Because of the long distance that the force main travels from the pump station, sulfide odors are a concern. A chemical feed building provides chemical storage and feed capacity for odor control.

Two chemical diaphragm pumps are used for feeding Bioxide into the manhole immediately upstream of the wet well. The Bioxide helps to prevent the formation of hydrogen sulfide gas. A 5,500 gallon polyolefin tank is used to store the chemical within the odor control building. The chemical feed system has a maximum feed capacity of 0.72 gallons per hour for each pump, and operated on a 120-volt single-phase power supply.

* * * * *

PART II – POPULATION AND FLOW PROJECTIONS

A. GENERAL

This section of the report discusses the historical population growth of the City of Raymore, probable population projections, and area population distributions for the next 20-year period under two different population growth distribution scenarios.

B. HISTORIC POPULATION TRENDS

The City of Raymore developed a Growth Management Plan (GMP) in 2002, which projected the City's growth through the year 2015. GMP growth projections were based, in part, on population trends recorded by the U.S. Bureau of the Census. In addition, undeveloped areas to the south of the City limits (Expansion Areas A, B and C) were identified by the GMP as likely areas for expansion during this period and beyond.

The City has experienced steady growth since 1970. This can be attributed in part to it's location just south of the Kansas City metro area as well as it's proximity to a number of other rapidly growing cities and towns in both Kansas and Missouri. All of these areas are well within commuting distance. Raymore offers a more affordable housing market than many other nearby areas. Table II-1 provides the historic population trends for the City since 1940.

Table II-1

Population Trends 1940 - 2000

1940	207
1950	238
1960	268
1970	587
1980	3,154
1990	5,592
2000	11,146

All of the population growth observed since 1970 has been within the City limits. Current growth is now occurring both within the existing City limits and in some areas south of the City limits. Within the City, more than 5,000 acres of undeveloped land still exists, and current residential growth is occurring as of the writing of this report.

C. GMP GROWTH PROJECTION

The 2002 Growth Management Plan projected that the population of Raymore would be 37,563 by year 2015. This projection assumed a relatively constant growth rate compared to past observed trends. Table II-2 provides the population projection from the 2002 GMP.

Table II-2

Population Projection from 2002 Growth Management Plan

	Past Trends	Projection
1990	5,592	-
1995	7,851	-
2000	-	11,146
2005	-	19,951
2010	-	28,757
2015	-	37,563

D. MASTER PLANNING GROWTH PROJECTION

A model for population growth was developed for the Master Plan through discussions between Burns & McDonnell and the City Staff. Based on the available area as well as recent residential construction activity and projected construction in the residential market, it was determined that a growth rate of 500 new homes per year until at least 2030 was a reasonable value to assume. An occupancy of 2.76 persons per home was determined from 2000 census data. Added to this was a large residential development underway on the north side of the City, known as Creekmore. This project will add 150 homes per year over the next 10 years to the City's population. Based on these numbers, the population of Raymore is projected to reach approximately 55,000 by 2030. Table II-3 and Figure II-1 provide population projections based on these calculations.

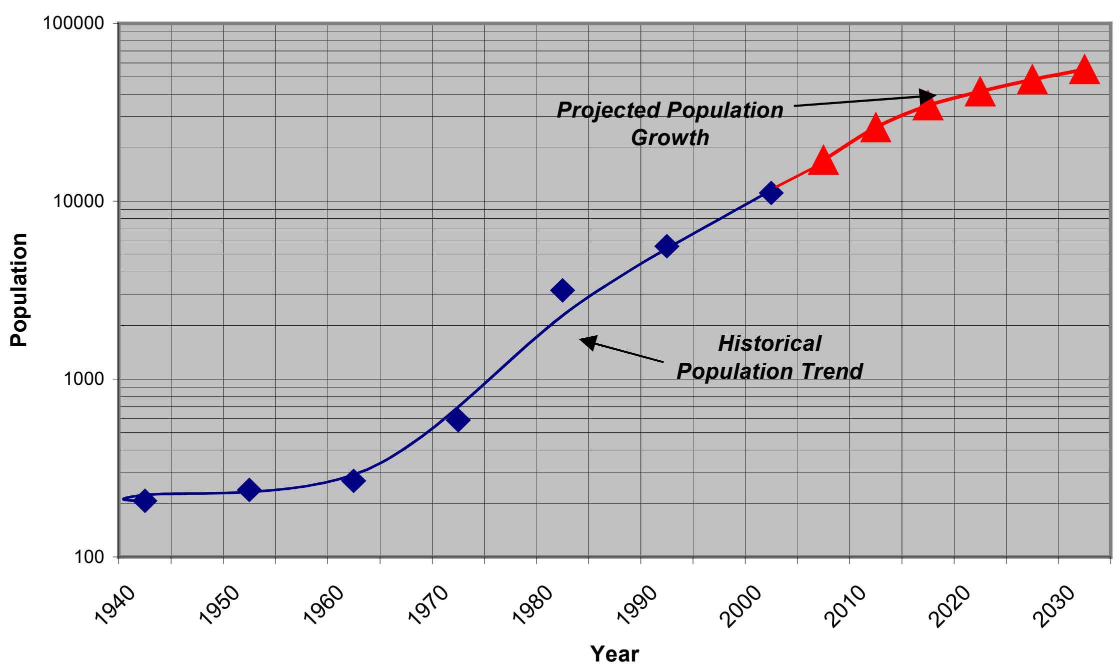
Table II-3

Population Projections for Raymore, Missouri

2000	11,146
2005	18,500
2010	27,400
2015	36,000
2020	42,900
2025	48,300
2030	55,000

It can be seen that there is a close agreement between the projections in the GMP and the projections developed in this Master Plan. Furthermore, an identical growth projection was used for the Water System Master Plan

Figure II-1
Historic and Future Population Trends
1940 – 2030



E. POPULATION DISTRIBUTION SCENARIOS

Two population distribution scenarios were considered for the Master Plan. Each scenario has its own specific patterns of growth and therefore effect the existing sewer system in a different way. Population growth in the City could take place within existing City limits or in the areas identified as Growth Areas A,B and C in the Growth Management Plan.

1. AREAS OF DEVELOPMENT

The development of the two scenarios took into consideration the probable expansion of the City into currently undeveloped areas. The following paragraphs describe these areas as a precursor to the description of the two main growth scenarios.

a. Existing Service Area

The area defined as the Existing Service Area (ESA) for the City has been previously described in Part I of this report. This area contains land that is outside of the City's current corporate limits. In addition, some parcels of land currently within the corporate limits are not included within the ESA. Figure II-2 shows the current ESA, city limits, and annexation limits as of the date of this report.

b. Expansion Areas

As a part of the Growth Management Plan, three areas to the south and east of the City Limits were identified as areas where future growth would most likely take place. The extent of these areas was determined mainly by the boundary of the existing service area as well as the current limits to annexation by the City. The timing of this growth is largely dictated by plans to construct a Highway 58 bypass away from the center of Raymore and to route this highway to the south of the existing service area. The construction is expected to provide new growth opportunities in these areas. The following paragraphs provide a description of the expansion areas, Figure II-3 shows the boundaries of Expansion Areas A, B and C relative to the Existing Service Area for the City of Raymore:

- Expansion Area A

Expansion Area A is located south of the existing service area, and has Highway 71 as it's west boundary, the City's limits of annexation as its southern boundary and the existing service area as its northern boundary. The eastern boundary is formed by the limits of the local watershed. The estimated area of land within that area is approximately 1,636 acres. This area will likely experience development before Areas B or C due to its location. The relocated Highway 58 will pass through this area and this fact plus nearby Highway 71 will likely cause commercial and residential development to begin in the near future. For the purposes of this Master Plan, it is assumed that Expansion Area A will be developed to a population of 5,700 from years 2010 to 2015 and would then hold steady at 5,700 through year 2025.

- Expansion Area B

  Expansion Area B is located south of the existing service area between Expansion Areas A and C, with the City's limits of annexation as its southern boundary and the existing service area as it's northern boundary. The eastern and western boundaries are formed by the limits of the local watersheds. The estimated area of land within the area shown in Figure II-3 is approximately 2,585 acres. An additional 970 acres immediately south of the area identified as Sub-Expansion Area B-2 is the continuation of that sub-watershed, and it is recommended that the City plan to develop infrastructure to serve that area as well. Expansion Area B will likely experience development after Expansion Areas A and C due to its location away from Highways 71 and 291. For planning purposes, it is assumed in this Master Plan that Expansion Area B will begin to experience significant development around year 2015, and will completely develop to a population of 11,700 (including Sub-Expansion Area B-2) by year 2025.

- Expansion Area C

  Expansion Area C is located southeast of the existing service area, and has Expansion Area B as its west boundary, the City's limits of annexation as its southern and eastern boundaries and the existing service area as its northern boundary. The estimated area of land within the area that is likely to be developed is approximately 5,354 acres. This area will likely experience development after Expansion Area A but before Expansion Area B due to its location. The Highway 58 bypass will pass through this area, and the improvement of Highway 291 will likely become a catalyst for commercial and residential development. For planning purposes, it is assumed in this report that once

Expansion Area C begins to grow in year 2010, it will develop to a population of 17,600 by year 2025.

2. POPULATION DISTRIBUTION SCENARIO NUMBER 1

a. Area development

The first modeling scenario that was considered in this report assumed that essentially all of the population growth between 2005 and 2025 would occur in Expansion Areas A, B and C. This was considered the worst-case scenario for new infrastructure required in the Expansion Areas. Expansion Area A would be developed first, in years 2010 through 2015. Expansion Area C would be developed in years 2010 through 2025. Expansion Area B would be developed in years 2015 though 2025.

b. Population Projection

The population estimates for each of the Expansion Areas was based on the 2025 Raymore population projection of approximately 48,300 people. This scenario assumed that the projected 2005 population of the Existing Service Area remains generally constant at 16,600, with the exception of the Creekmore development area north of Highway 58, which will add approximately 4,100 people to the ESA's population over the next 10 years. While the ESA experiences very little growth, the expansion areas were assumed to accommodate the remaining 27,600 new citizens of Raymore plus an additional 15% to account for sewer capacity needed due to commerical development such as restaurants, hotels, etc. Based on the populations of each area described previously, a population density in the Expansion Areas of 3.28 persons per acre was established. Each Expansion Area was assumed to populate at a steady rate throughout the growth period described for each Area in the previous paragraph. Table II- 4 provides the projected population for each area from 2005 to 2025. The table also provides the ultimate, or build-out population for each area, based on Census data parameters of 3 homes per acre and 2.76 people per household. The date of this build-out population cannot be accurately projected, as it is determined by a number of factors including the ability and willingness of Raymore to expand it's infrastructure , as well as the capacity of the Expansion Areas to be further developed into more densely populated neighborhoods. Based on the 3 homes per acre, 2.76 persons per home figures, the ultimate population of all three Expansion Areas will be 85,710. This figure includes the 970 acres immediately south of Sub-Expansion Area B-2, which is a component of that watershed.

Table II-4

Population Projections – Scenario Number 1

Year	Expansion Area A	Expansion Area B	Expansion Area C	Yearly Total
2005	0	0	0	0
2010	0	0	0	0
2015	5,687	0	8,785	14,472
2020	5,687	4,247	17,561	27,495
2025	5,687	11,660*	17,561	34,908*
Ultimate	14,349	28,476**	42,885	85,710**

Notes: * Includes approx 3,200 persons out of Expansion Area B, but within watershed

** Includes approx. 8,000 persons out of Expansion Area B, but within watershed

3. POPULATION DISTRIBUTION SCENARIO NUMBER 2

a. Area Development

The second modeling scenario that was considered in this report was considered the worst-case scenario for the Existing Service Area and the infrastructure within this area. In this scenario, the Service Areas south of Highway 58 within the Existing Service Area, along with Expansion Area A experience population growth that results in a 2025 City-wide population of approximately 48,300 people. Expansion Areas B and C do not experience growth in this scenario. This scenario takes the Creekmore Development area into consideration, but does not consider population growth beyond this for areas north of Highway 58. The rate of growth in this scenario would be constant from year 2005 through year 2025.

b. Population Projection

The population project for this scenario began with the same assumptions as Scenario Number 1, mainly that the 2005 population of Raymore will be approximately 16,600, that the Creekmore Development area will provide 4,100 new persons by 2015, and that the additional population added to Raymore by year 2025 will be approximately 27,600 plus a 15% allowance for commercial sewer users. The previously calculated 2025 population for Expansion Area A, was assumed to be valid for this scenario as well. The remaining population growth in this scenario was assumed to take place at a constant rate between 2005 and 2025 within the southern areas of the Existing Service Area. Table II-5 provides the projected population for these two areas.

Table II-5

Population Projections – Scenario Number 2

Year	Expansion Area A	ESA – south	Yearly Total
2005	0	9752	9752
2010	5,687	16,265	22,312
2015	5,687	22,778	28,465
2020	5,687	29,292	34,979
2025	5,687	35,805	41,492
Ultimate	15,964	38,560	54,524

F. FLOW PROJECTIONS

Based on the population projections made for both Scenarios, flow projections for each Service Area along with each of the Expansion Areas were developed utilizing flow monitoring data and the developed and undeveloped area measurements for each Service and Expansion Area. Several extended periods of dry weather allowed for the collection of dry weather flow data in the collection system. Wade & Associates, who provided the flow monitoring services during the data collection phase (see Part III of this report), generated hydrographs to represent both weekday and weekend dry weather flows. These hydrographs were generated for each flow monitor in the system, and therefore each Service Area has a unique dry weather flow hydrograph. An average daily flow value for each Service Area was then calculated by averaging the total flow measured per day over a number of dry days.

Projecting future wastewater flow rates to each Service Area requires a population projection for each Service Area. Year 2005 populations were allocated to each Service Area on a percentage developed acreage basis. For example, the Year 2005 population of Raymore is projected to be 16,660 people with a total developed acreage of 3030 acres. Service Area 7 has a total developed acreage of 503 acres. Thus, the population of Service Area 7 was calculated as follows:

$$Area\_7\_Population \approx \frac{503}{3030} \times 16600 \approx 2{,}756$$

Year 2025 populations were then developed for each of Service Areas 5 - 10 based on the ESA population calculated for Population Distribution Scenario No. 2, and distributed into each Service Area according to percentage total undeveloped acreage. Year 2025 populations were developed for Service Area Nos. 1, 2, and 10 by calculating the ultimate population of three homes per acre and 2.76 people per home. After calculating the populations for Year 2005 and Year 2025 for Service Areas 1-2 and 5-11, the growth rate between 2005 and 2025 was assumed to be constant. Service Area 3 was assumed to experience zero growth through the Year 2025. Populations for Service Area 4 were developed using the ultimate population plus the data provided in the *Creekmore Sanitary Sewer Feasibility Study by Archer Engineers dated December 31, 2003*. The methods chosen to project population for each Service Area were selected to represent the "worst case" with respect to the wastewater infrastructure. It should not be presumed that any of these worst-case projections for individual Service Areas or Expansion Areas would exist simultaneously with another worst-case projection in another Area. Each was chosen to provide a means to analyze each Area independently, since the exact distribution of future population cannot be known at this time.

Table II-6 provides a summary of the population growth strategy for each individual Service Area.

Table II-6

Population Growth Strategy Summary

Service Area	Population Basis
5, 6, 7, 8, 9, 10	Population Distribution Scenario 2
1, 2, 11	Ultimate Development
4	Ultimate Development + Creekmore Population
3	Zero Growth

After developing population projections for each Service Area, individual Year 2003 dry weather hydrographs were scaled up for years 2010, 2015, 2020, and 2025. For example, the population factor was applied to the hydrographs as follows:

$$\text{Year 2015 Hydrograph} = \frac{Year\_2015\_Population}{Year\_2005\_Population} \times Year\_2003\_Hydrograph$$

The sole exception to this procedure was Service Area 11, where complications with high base infiltration complicated the evaluation of the hydrograph. For this area, the upstream hydrographs form Service Areas 5&6 were used to develop a composite hydrograph that was based on an average flow rate of 100 gal/cap/day.

For the Expansion Areas, flow calculations were based on a flow contribution of 100 gallons per capita per day for an average flow plus a peaking factor of 3.0 to accommodate for future wet weather. This value was considered the flowrate at ultimate conditions. The flow rates at 2005, 2010, 2015, 2020 and 2025 could then established using the population projections for each of the Expansion Areas as provided in Table II-4 of this report.

Table II-7 provides summary information on the projected population and flowrates for the Expansion Areas investigated in Growth Scenario Number 1. Table II-8 and II-9 provide summary information regarding the results of wastewater flow projections for the Existing Service Areas, as well as for Expansion Area A, which is a component of Growth Scenario Number 2.

*****

PART III – DATA COLLECTION

A. GENERAL

Prior to the development of the computer model or analysis of the collection system, field data was collected to provide baseline data on existing conditions within the interceptors and how they respond to dry and wet weather events. This section of the report provides a summary of the flow data collection performed in preparation for the analysis.

The data collection phase for the development of this master plan involved flow monitoring in strategic locations throughout the Existing Service Area from May 1st through July 8th 2003. Rain gauges were also placed in a number of locations throughout the City for the same time period.

B. FLOW MONITORING PROGRAM

Flow monitoring devices were placed in thirteen (13) locations, including locations within each of the six sewer interceptors that are addressed in this Master Plan. The flow meters continuously recorded data at 15-minute increments for velocity and depth of flow. The meters were operated from May 1, 2003 through July 8, 2003, and captured data during dry weather conditions as well as several wet weather events. The information gathered was used evaluate infiltration and inflow impacts on the collection system and to calibrate a hydraulic model of critical interceptor sewers. Table III-1 presents information on the flowmeters used in this collection system.

Flow meters and rain gauges were installed, calibrated and maintained by Wade and Associates of Lawrence, KS. Copies of the flow monitoring reports are provided in Appendix B.

TABLE III-1

FLOWMETER LOCATIONS

Upstream Service Area No.	Flowmeter No.	Manhole No.	Interceptor	Approximate Location
1	M1	AC174	Alexander Creek	Highway 58 and Kurzweil Road
2	M2	AC81	Alexander Creek	Highway 58 and Kurzweil Road
3	M3	AC2	Alexander Creek	Raintree Parkway near Highway 291
4	M4	LF001	Lampkins Fork	155th Street east of Kentucky Road
5	M5	WGE6	GRE	Magnolia Street west of S. Madison
5	M5-1	GWRE6	GRE	Magnolia Street west of S. Madison
6	M6-1	GE13	GRE	S. Madison south of Sunny Lane
7	M7	GRW17	West	Horseshoe Dr and Foxridge Rd.
8	M8	GRW2	West	195th St. east of Highway 71
9	M9	GWR24	GRE Relief	South of Cedar Ridge Circle
9	M9-1	GW14	GRE Relief	South of Cedar Ridge Circle
10	M10	GWR8	GRW Relief	In Owengood Ranch
11	M11	GRE1	GRW	In Owengood Ranch

The flow meters use a solid-state electromagnetic sensor that correlates measured liquid velocities to the mean velocity in a sewer pipe of a specified diameter as well as a pressure transducer that measures water depth to compute flow area. The flow rate is then calculated from this data using the continuity equation:

$$Q = V \times A$$

where: Q = flowrate, cfs

V = mean velocity, ft/second

A = flow area, ft^2

Rainfall measurements were recorded at five (5) different locations throughout the City, generally distributed throughout the various Service Areas that make up the Existing Service Area. The location of the flow meters is shown on Figure III-1. Table III-2 identifies the five rain gauges and their approximate locations.

TABLE III-2

RAIN GAUGE LOCATIONS

Rain Gauge No.	**Description / Location**
RRG1	Owen-Good Pump Station
RRG2	Public Works Office Roof
RRG3	155th and Kentucky Road
RRG4	Sierra Drive and Ward Road
RRG5	City Hall Building Roof

The gauges recorded rainfall depth and intensity at 15-minute intervals, which led to the development of rainfall hyetographs. Figure III-1 also shows the locations of the rain gauges. During a period from April 2 to July 29, 2003 a number of rainfall events were recorded, ranging from 0.4 inches to 1.8 inches of total rainfall per event. All of the rainfall events were less than 2-year storm events as referenced from MoDOT Intensity Duration Frequency Curves for the Kansas City region. This data was used for wet weather flow calibration by comparing model results with the actual rainfall events that occurred during the monitoring periods. Also, this data was used to develop a correlation between rainfall intensity and collection system response. The correlation was used to develop a theoretical design storm event and extrapolate the impact of the theoretical design storm on the collection system.

*****

PART IV – FLOW DATA ANALYSIS AND RESULTS

A. GENERAL

This section of the report provides additional information regarding the flow and rainfall monitoring that was performed in 2003. The results of the monitoring were used to calibrate the hydraulic model as well as to develop priorities for the Phase II evaluation which will consist of a focused and more detailed investigation of Infiltration and Inflow (I/I) sources. Field activities for Phase II will consist of smoke testing, manhole inspections and closed circuit television inspections (CCTV). In addition specific defects will be identified along with estimates of I/I contribution and opinions of probable cost for repair of defects. The Phase II investigation began in the Summer of 2004.

B. RESULTS

During the flow and rainfall monitoring period described in Part III, wastewater flows were measured at 15-minute intervals. Utilizing rain gages, the depth of rainfall was recorded and rainfall intensities were calculated. Information obtained by both the flow meters and rain gages were compared to identify dry and wet weather flow patterns for each service area. The difference between the dry and wet weather flow constitutes infiltrat ion and inflow contribution.

Flow data from four storm events were used to determine wet weather flow patterns. Dry weather flows were determined from flow meters during the week of May 21-May 27, 2003. During that time period, no rainfall was recorded.

To provide a method for ranking service areas according to the severity of I/I, two analyses were conducted. The first analysis consisted of comparing the average I/I flows per unit length of pipe (gpd/inch diameter per foot of pipe). The second analysis compared the peak I/I flows per unit length of pipe (gpd/inch diameter per foot of pipe). A ranking of "1" for "Worst" to "11" for "Best" was given to each service area for each analyses. To determine the final service area ranking, an average ranking was determined from both individual rankings.

Table IV-1 gives a summary of the average and peak I/I for each service area and the individual and composite rankings.

Table IV-1

Average and Peak I/I amounts per Service Area

Service area	Avg. I/I Flow gpd/in*ft	Peak I/I Flow gpd/in*ft	Composite Ranking
1	0.50 (5)	6.55 (7)	**6**
2	0.61 (4)	6.00 (8)	**5**
3	0.24 (9)	9.27 (4)	**7**
4	0.64 (3)	10.35 (3)	**2**
5	0.32 (7)	4.19 (9)	**9**
6	0.92 (2)	7.20 (6)	**4**
7	0.30 (8)	3.73 (10)	**10**
8	0.41 (6)	15.18 (1)	**3**
9	0.06 (11)	3.33 (11)	**11**
10	1.51 (1)	14.12 (2)	**1**
11	0.17 (10)	7.95 (5)	**8**

Individual Rankings are in ( ).

Table IV-2 gives a distribution of the worst to the best service areas.

Table IV-2

Service Area Ranking Distribution.

Ranking	Service Area
1 (Worst)	10
2	4
3	8
4	6
5 (Medium)	2
6	1
7	3
8	11
9	5
10	7
11 (Best)	9

C. RECOMMENDATIONS

Based on the monitoring program, rankings and interviews with City Staff, it is recommended that a combination of investigative actions including smoke testing, manhole inspections and televising of identified problem sewer lines, be accomplished for each service area. These efforts would provide a basis for an I&I reduction program.

Infiltration is typically generated by groundwater penetrating the collection system through defective joints or holes in manholes and pipes. Inflow can come from many sources, including roof drains, foundation drains and sump pump discharges tied into the collection system. Inflow from surface runoff can drain into manholes with open or ventilated lids. Large defects in manholes, pump station wells, or collection system piping can also be a route for surface runoff to the collection system.

I&I reduction consists of structural improvements such as repair and replacement of damaged and defective infrastructure components, as well as administrative improvements such as City codes that eliminate foundation and roof drains from connecting to the collection system and prevent them in future housing developments.

Meetings were held with City Engineering and Operations Staff to verify areas of the collection system that have historically experienced problems such as surcharged manholes, basement backups and frequent trouble calls. Interviews confirmed that portions of the following service areas were historically problematic: 10, 4, 8, 6, 2, and 1. In addition, it was noted that service area 9 exhibited very high per capita flows during dry weather. This is likely due to infiltration induced by high groundwater. Based on analysis of flow data and interviews with City Staff, field investigations of portions of service areas 10, 4, 8, 6, 2, 1, 9, and a small portion of 7 are recommended. Figure IV-1 shows the areas to be investigated.

A cursory review of I&I improvement projects applied to cities the size of Raymore suggest that an aggressive effort to reduce I&I could result in reductions of between 35% to 45% of the measured peak wet weather I&I flowrates. Phase I of this effort was the monitoring program. The Phase II effort will more clearly define the extent of rehabilitation needs within the collection system through a variety of field investigations. Table IV-3 gives approximate quantities of the recommended Phase II field investigations. The City has recently purchased a CCTV apparatus and will provide the CCTV inspections.

Table IV-3

Recommended Phase 2 Activities for Each Service Area.

Service Area	Manhole Inspections (each)	Smoke Testing (LF)	CCTV* (LF)
1	171	42,740	42,740
2	24	13,185	N/A
4	78	16,554	21,187
6	50	12,956	12,956
7	18	4,400	N/A
8	51	15,300	N/A
9	4	48,050	12,681
10	45	18,240	18,240

*: CCTV inspections to be performed by City Staff.

* * * * *

PART V – MODEL DEVELOPMENT AND RESULTS

A. GENERAL

The interceptor systems identified in the Introduction were modeled using a computer simulation to project the likely response of those parts of the collection system under the projected future wastewater flow rates. Both dry weather and wet weather conditions were simulated, using the projected flowrates as described in Part II of this report. The modeling simulations were run under two different conditions. The first condition was that the relative I&I contribution of the interceptors remained unchanged in the future. The second condition considered the potential effects of I&I reduction on the collection system. For the purposes of modeling the effects of the City's I/I reduction program, the wet weather input hydrographs were reduced in magnitude by 40%, representing a 40% reduction in I&I throughout the system. The results of the Phase II field investigation of the sewer collection system will provide a more accurate estimate of the potential for I&I reduction. If it is determined that a 40% reduction is not possible, the results and conclusions of this report will need to be modified accordingly. Part V of this report provides a description of the data used, a limited description of the modeling process, a summary of the modeling results, and the criteria used in developing the hydraulic model.

B. MODEL DEVELOPMENT

The Existing Service Area, which includes the City of Raymore as well as areas outside of the City limits, contains all of the sewer interceptors that were modeled as part of this Master Plan. Wastewater flow is collected and conveyed through one of these six sewer interceptor systems. The following is a brief summary of the physical characteristics of each interceptor system that was modeled:

- Alexander Creek Interceptor
  This interceptor, located within Service Area 3, collects wastewater from an area approximately 5,000 acres in size. The interceptor is 18 inches in diameter and constructed mainly of PVC. The interceptor varies in slope from 0.1% to 2.5%.

- GRW Interceptor

This interceptor passes through Service Areas 9 & 10 and serves a total area of over 800 acres. The main GRW Interceptor and parallel relief lines collect wastewater from these Service Areas as well as flow from the West Interceptor. Flows are then conveyed to the Owen-Good Pump Station. The majority of the interceptor is 15-30 inches in diameter and constructed of PVC in the upstream sections, and reinforced concrete pipe further downstream. The interceptor varies in slope from 0.2% to 1.5%.

- GRW Relief Sewer
  This relief sewer, located within Service Area 10, collects wastewater in parallel with the GRW Interceptor from an area approximately 800 acres in size. The relief sewer is constructed predominantly of 10-inch PVC pipe. The relief sewer varies in slope from 0.2% to 0.9%.

- West Interceptor
  This interceptor, located within Service Area 8, collects wastewater from an area approximately 500 acres in size. The diameter of the interceptor ranges from 18 to 21 inches and is constructed of PVC. The interceptor varies in slope from 0.2% to 1.5%.

- GRE Interceptor
  This interceptor, located within Service Areas 5, 6 & 11, collects wastewater from a combined area of approximately 2,300 acres in size. The interceptor varies in size from 8 to 30 inches in diameter and is constructed mainly of reinforced concrete pipe. The interceptor varies in slope from 0.2% to 3.4%.

- GRE Relief Interceptor
  This relief interceptor conveys excess flow from Service Areas 5, 6 & 11. It collects wastewater, along with the main GRE Interceptor, from a combined area of approximately 2,300 acres in size. The interceptor varies in size from 10 to 12 inches in diameter and is constructed of PVC pipe. The interceptor varies in slope from 0.4% to 3.4%.

- Lampkins Fork Interceptor
  This interceptor, located within Service Area 4, collects wastewater from an area approximately 3,000 acres in size, as well as all flows pumped by the Owen-Good Pump Station. The

interceptor is 27 to 42 inches in diameter and constructed of reinforced concrete pipe. The interceptor varies in slope from 0.2% to 2.0%.

The modeled interceptors were shown on Figure IN-1.

As described in Part II of this report, three additional areas outside of the Existing Service Area are included in this Master Plan. These areas, called Expansion Areas A, B and C, were identified in the 2002 Growth Management Plan and were selected as the most likely areas for growth outside of the City limits. Part II of this report presented a detailed description of these areas.

New interceptors for those areas were sized for ultimate populations. Because there is no existing sewer infrastructure in these areas, no hydraulic computer simulation for these new interceptor designs was performed. Detailed information on the preliminary sizing of the interceptors for the Expansion Areas, as well as other wastewater infrastructure needs for those areas, is presented in Part VI of this report.

The capacities of the seven interceptors described on the preceding pages were analyzed using the HydroWorks™ hydraulic modeling software. This software is a dynamic hydraulic modeling tool that calculates the velocity, flowrate and depth of flow within each pipe segment in the model for a given input of wastewater and wet weather I&I. The hydraulic equations, which HydroWorks™ uses to describe flow propagation in both open channel and pressurized sewer pipes, are known as the 'de Saint-Venant' equations. These are non-linear partial differential equations that use simplified numerical algorithms for their solution. When calibrated with a sufficient amount of data, these simplified models are known to be relatively accurate in describing the dynamic behaviors of a sewer collection system. Results under each set of conditions are computed for a small increment of time, which are carried forward for the next set of computations representing the next small increment in time. The data provided from this type of model can describe the likely status of the collection system throughout any specified dry or wet-weather event. Recommendations can be made regarding what part or parts of these modeled interceptors require upsizing and/or rehabilitation based upon these results.

Before the model can be used to develop useful data, the physical components of the sewer network must be represented or "built" inside the computer model space. The model requires basic physical attribute information to construct the interceptors. This information typically is taken from construction drawings and sewer atlas maps as well as, in this case, from the City's GIS database files. Information includes:

- pipe diameter
- pipe length between modeled manholes
- upstream and downstream pipe inverts
- pipe condition (effecting the friction losses within the pipe)
- manhole location and numbers
- basic manhole design (internal area, depth, etc.)

The physical attribute data for the interceptors is converted into .dsd files for use by the HydroWorks™ modeling software. After inputting the data, HydroWorks™ initiates a validation process. The validation process checks to see that each interceptor is physically contiguous, and that the minimum amount of data has been provided for each physical element of the model to allow operation of the modeling engine. Figures V-1 through V-4 show plan views and pipe diameters of the seven interceptors that were modeled.

The sources and quantities of wastewater and I&I water that are to be conveyed by the modeled interceptors are converted into .qin file hydrographs. Hydrographs represented by the .qin files may be input into specific manholes at the user's discretion. Figure V-6 shows the location where flow was added to the collection system for each interceptor.

Once the physical data and the flow data are assembled the software is ready to run a simulation. In order to specify a particular simulation, a specific .qin file and a specific .dsd file must be associated with the established project. After the flow data and the physical data have been set and the model has been calibrated, HydroWorks™ can simulate a specific scenario over a user-defined time period.

Calibration of the model is required to obtain an acceptable agreement between measured and computer-generated flows for both dry and wet weather conditions. This provides some assurance that the modeling results for future flows are truly predictive for the system. Typically, a number of observed and recorded dry weather and wet weather events are compared to the model results under similar modeled circumstances. Adjustments to the .qin hydrographs in the model are performed to bring behavior of the model as close to the observed behavior of the actual collection system as possible. The following paragraphs provide a summary of the calibration method used:

- Dry Weather Calibration and Modeling

  Several extended periods of dry weather allowed for the collection of dry weather flow data in the collection system. Minimum flowrates occurring during the dry weather periods are considered to be mainly infiltration from groundwater sources not directly associated with storm events. These minimum flows typically occur between 1:00 am and 3:00 am. Wade & Associates, who provided the flow monitoring services during the data collection phase, generated hydrographs that represent both weekday and weekend dry weather flows. These hydrographs were generated for each flow monitor in the system. When the model was developed, simplified hydrographs were developed to mimic the dry weather flow hydrographs observed at each of the upstream flow meters. In addition, smaller flow hydrographs were developed which were assigned to multiple locations along the interceptors downstream of these meters. These hydrographs represent wastewater flow added to the interceptor downstream of the upstream flow meter as well as various dry weather I&I contributions that could not be directly observed by the flow meters. All of these hydrographs were written into the model's .qin file and were routed through each interceptor during a simulation. The results of the modeling simulation were then compared to the flow monitoring results at the downstream flow meters. Modifications were then made to the smaller, downstream hydrogaphs in the .qin file. These hydrograph modifications were performed to improve the match between the model output and the downstream meter readings for dry weather events.

  Future dry weather flow hydrographs were then developed for each Service Area as detailed in Part II of this report. Those flows were similarly written into .qin files and were used to model the future scenarios for each of the seven interceptors.

- Wet Weather Modeling

  The relationship between the Service Area and I&I within the system will dictate the amount of water that is added to the raw wastewater in the collection system during a wet weather event. Each Service Area exhibits a unique pattern of wet weather I&I that is partially dictated by the total area and geography that contributes to the I&I, and partially dictated by the physical condition of the collection system. Systems with high I&I typically are older collection systems with damage or deterioration to pipe joints, pipe walls and manholes. The segments of the collection system modeled in this master plan are interceptors only, thus the majority of the wastewater that is collected occurs at the upstream reach of the pipe. Additional contributions are limited mainly to the confluence of another interceptor. Because of this, the only other significant source of water into the interceptor is I&I. Strategic locating of flow monitoring devices helped determine the extent of I&I for each interceptor. Flow meters were placed at the far upstream and the far downstream end of each interceptor. Thus, I&I flows and contributing wastewater flows from a given storm event were roughly determined by subtracting the dry weather hydrograph from the wet weather hydrograph from a specific flowmeter. The incremental flow between two meters could be determined by subtracting the upstream values from the downstream values.

The development of wet weather input hydrographs was performed using wet weather data collected by the flow meters along with rainfall data collected by the rain gauges. For the purposes of this report, the wet weather conditions projected in the model were established to represent a 5-year, 1-hour storm. The 5-year return interval is one that is very commonly used by municipalities in hydraulic evaluations such as this one. The one-hour duration was chosen since the time of concentrations for the eleven Service Areas are all less than one hour. In terms of storm intensity, the shorter the duration of the storm, the more intense the rainfall and the larger the observed peak I&I rates are likely to be. This storm therefore represents an intense rainfall with a large peak I&I. In this case, the peak storm intensity was a rainfall rate of 2 in/hour.

For wet weather flows, the data collection period included four wet weather events that were recorded by both the flow meters within the collection system and the rain gauges located throughout the existing service area. The data from the flowmeters were used to generate wet weather hydrographs for each of the rain events at the various meter locations. The data from the rain gauges was used to develop rainfall hyetographs for the same events. A correlation was then developed for each Service Area between the maximum individual storm intensity and the observed peak flowrate of I&I within the interceptor at the location of the flow meter. Not all data that was collected was used in the correlations, as some of the observed rainfall events were either too brief or too light in intensity to result in a robust response from the collection system. From measured values of flow, rainfall intensity and land service area, an inflow coefficient was calculated for each Service Area utilizing the Rational Formula:

$$Q = K \times I \times A$$

where: Q = peak flow observed in the interceptor (cfs)
I = Peak rainfall intensity (in/hr)
A = Contributing area of the Service Area(s) (acre)
K = Inflow Coefficient

The calculated inflow coefficient was specific to each Service Area and was used to generate a graph relating peak rainfall intensity for a given storm (I) to the peak I&I that would be observed within a given interceptor in each Service Area (Q). The peak flowrate predicted by the graph for a particular storm could then be applied to a normalized area-specific wet weather hydrograph to generate a wet weather hydrograph specific for that storm in that Service Area. This graph could then be added to the dry weather hydrograph to create a combined hydrograph that was routed through the system.

The storm intensity and I&I peak flow correlations were also used later to predict peak I&I within the system for the desired 5-year, 1-hour storm event. Figures V-7 through V-14 show the Q vs. I graph for Meters M1 through M11, with the exception of Meters M3, M4, M5-1 and M11. These four Meters did not yield a positive coefficient for a variety of reasons. Meter M3 was located at the downstream end of the Alexander Creek Interceptor, and the results suffered from significant attenuation due to the long travel time, as well as the fact that the data collected indicated that the majority of the Interceptor experienced very little I&I. Meter M4 suffered from the contributing flow of the Owen-Good Pump Station, rendering accurate calculation of the wet weather I&I unreliable. Meter M5-1 monitored a relief line that conveyed flow for only 4 hours during the entire monitoring period, therefore not providing enough data to reliably calculate the coefficient while also demonstrating that little to no I&I reached that line. Meter M11 was located at the downstream end of the GRE and GRE Relief Interceptors, and conveyed flow from Service Areas 5, 6 and 11. The data from this interceptor suffered from a high baseline I&I that was masked within the dry weather data, resulting in calculated wet weather flow values in this interceptor that were artificially low.

Figure V-7

Storm Intensity vs. Inflow – Meter No. 1

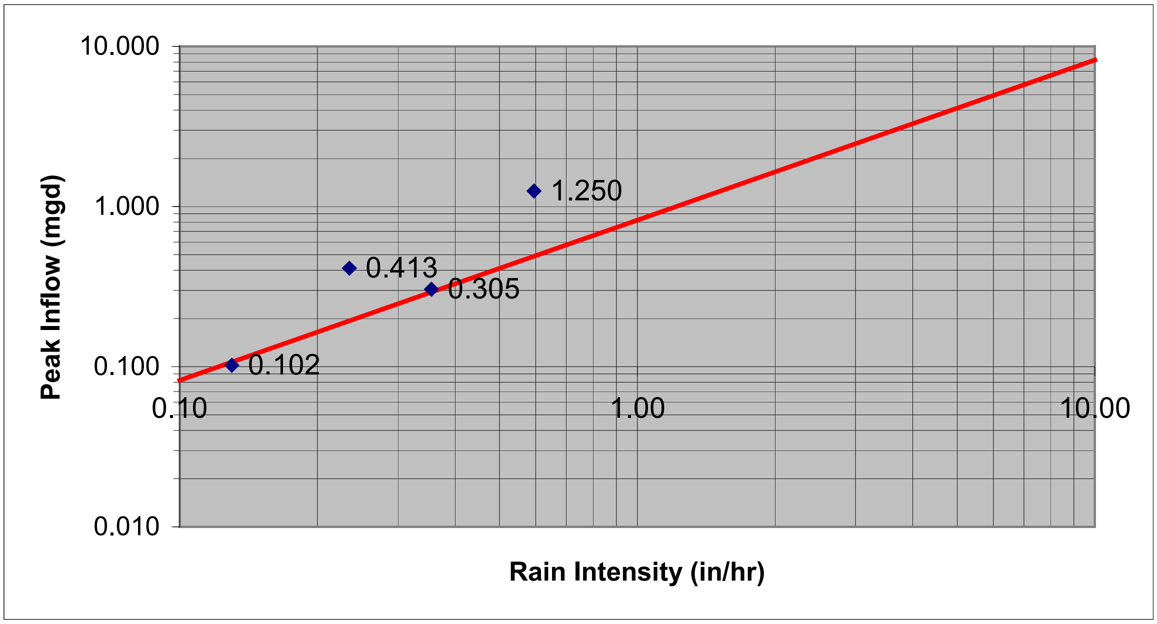


Figure V-8

Storm Intensity vs. Inflow – Meter No. 2

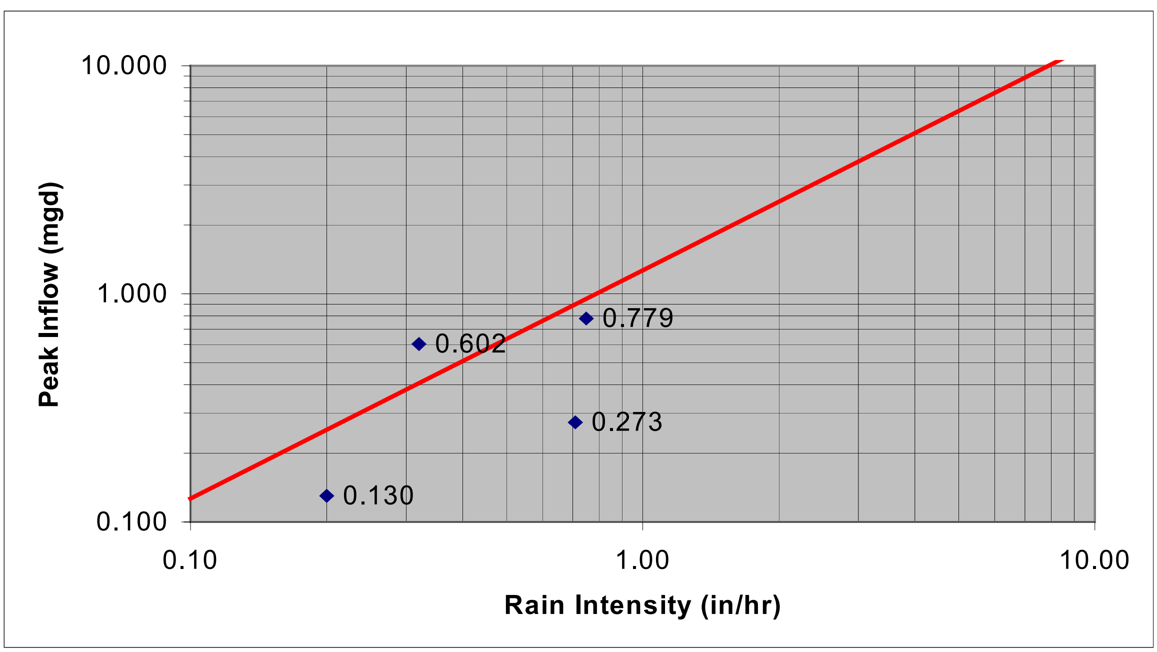


Figure V-9

Storm Intensity vs. Inflow – Meter No. 5

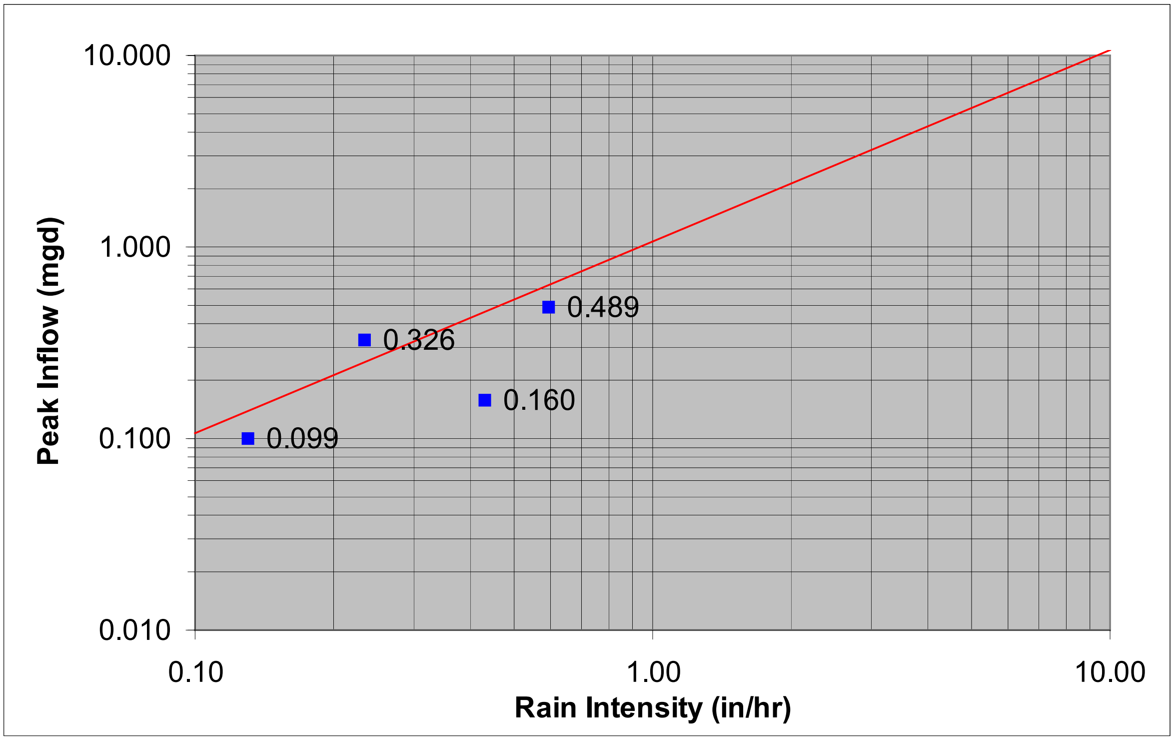


Figure V-10

Storm Intensity vs. Inflow – Meter No. 6a

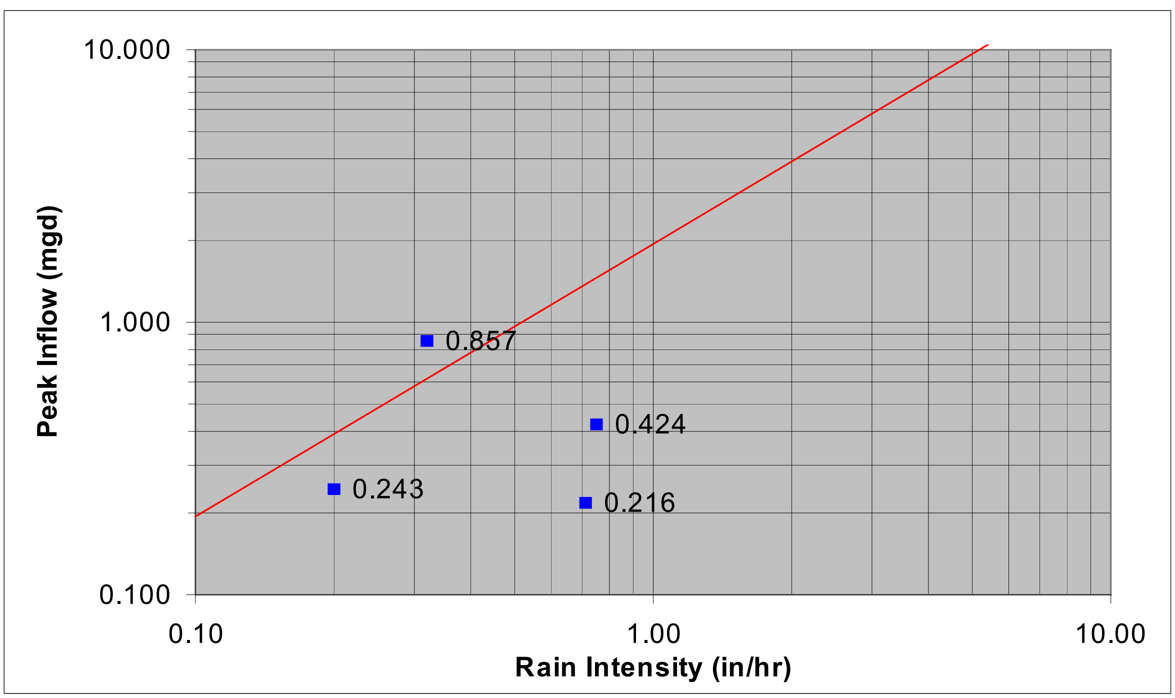


Figure V-11

Storm Intensity vs. Inflow – Meter No. 7

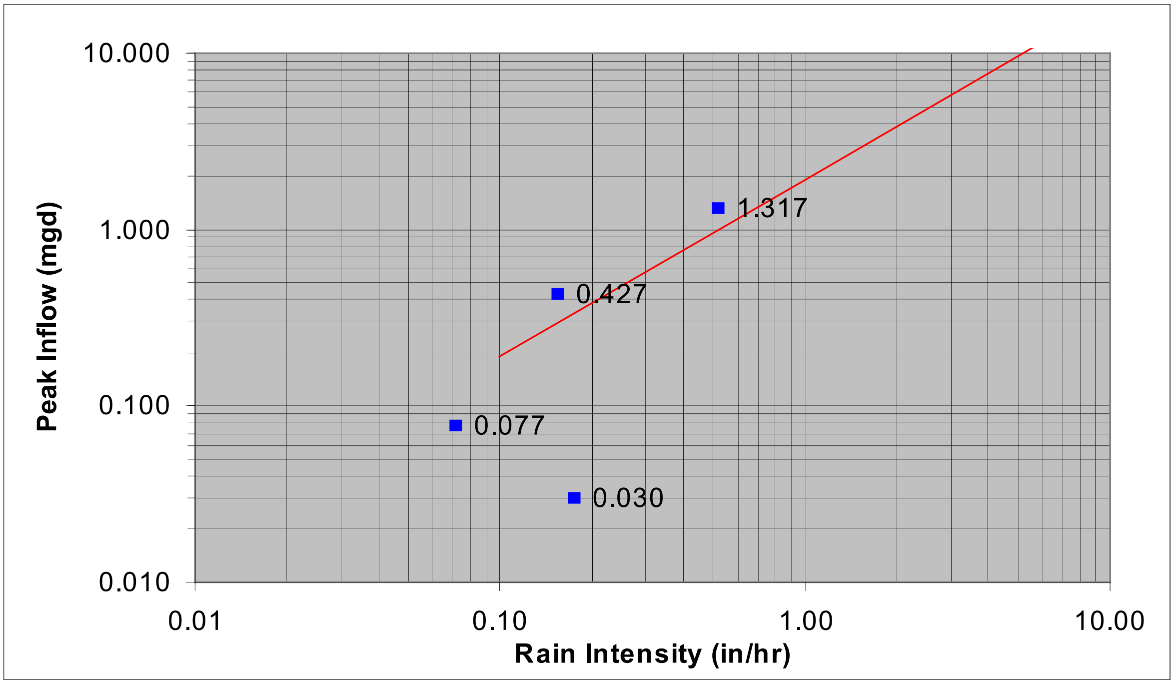


Figure V-12

Storm Intensity vs. Inflow – Meter No. 8

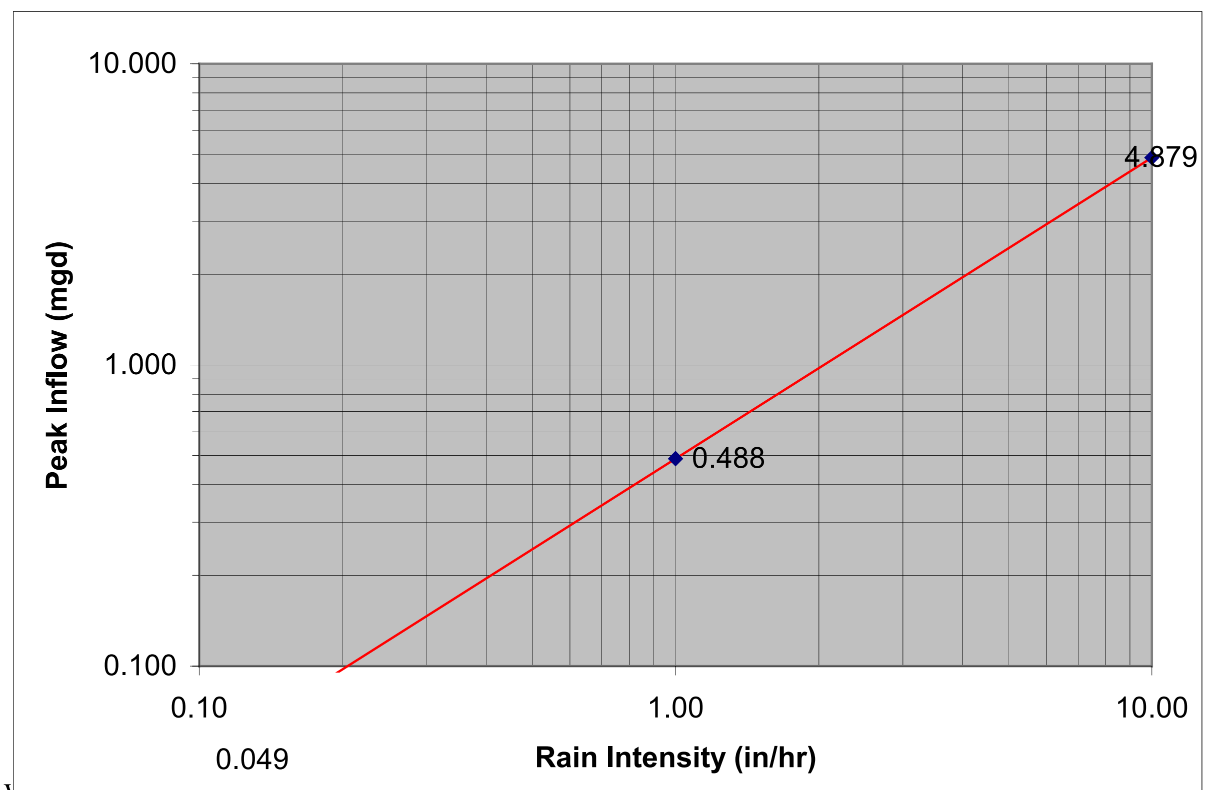


Figure V-13

Storm Intensity vs. Inflow – Meter No. 9

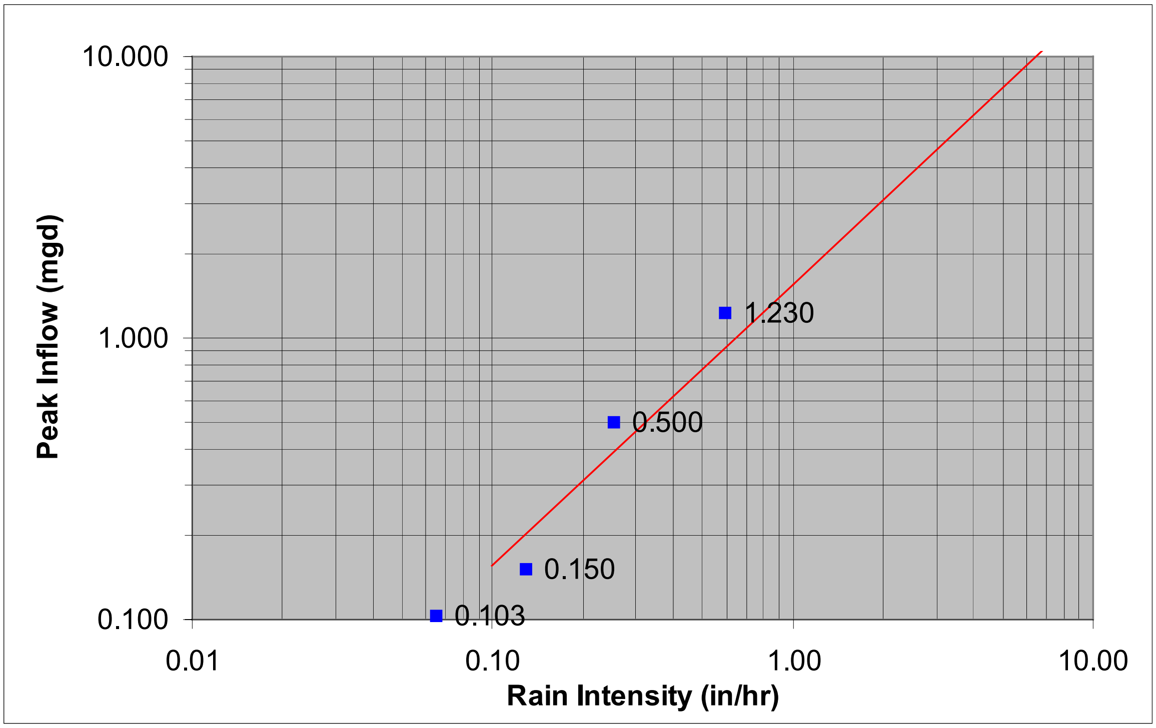


Figure V-13a

Storm Intensity vs. Inflow – Meter No. 9a

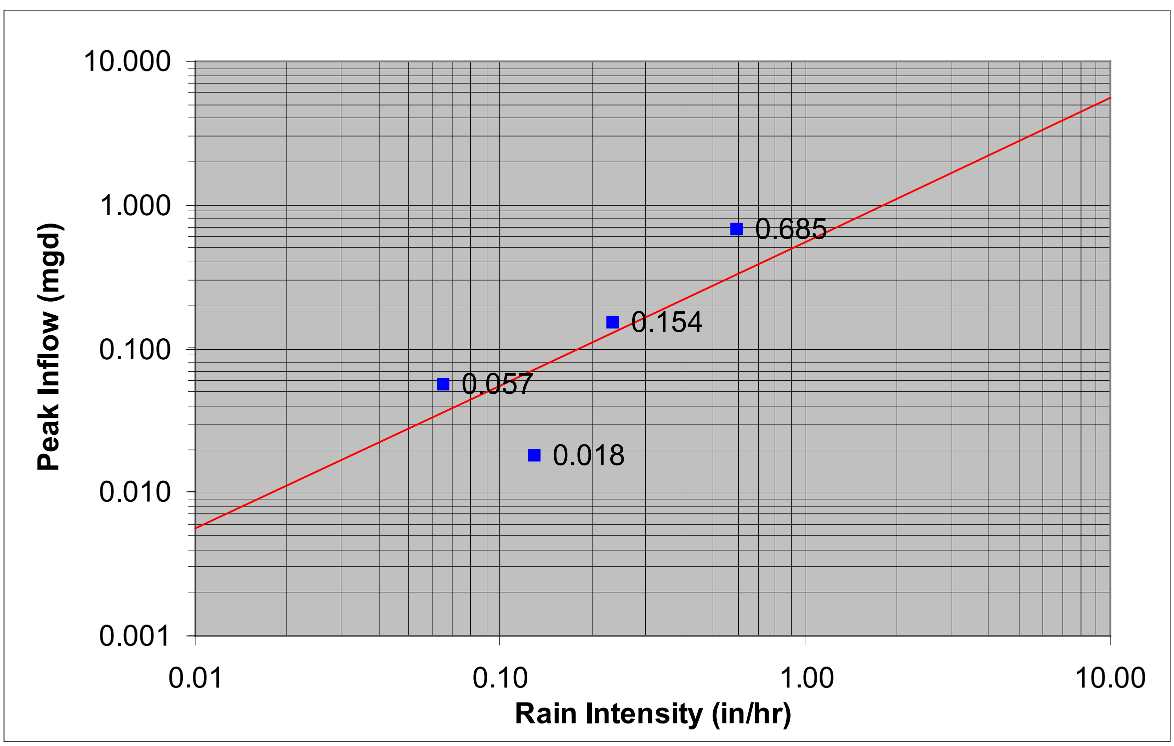
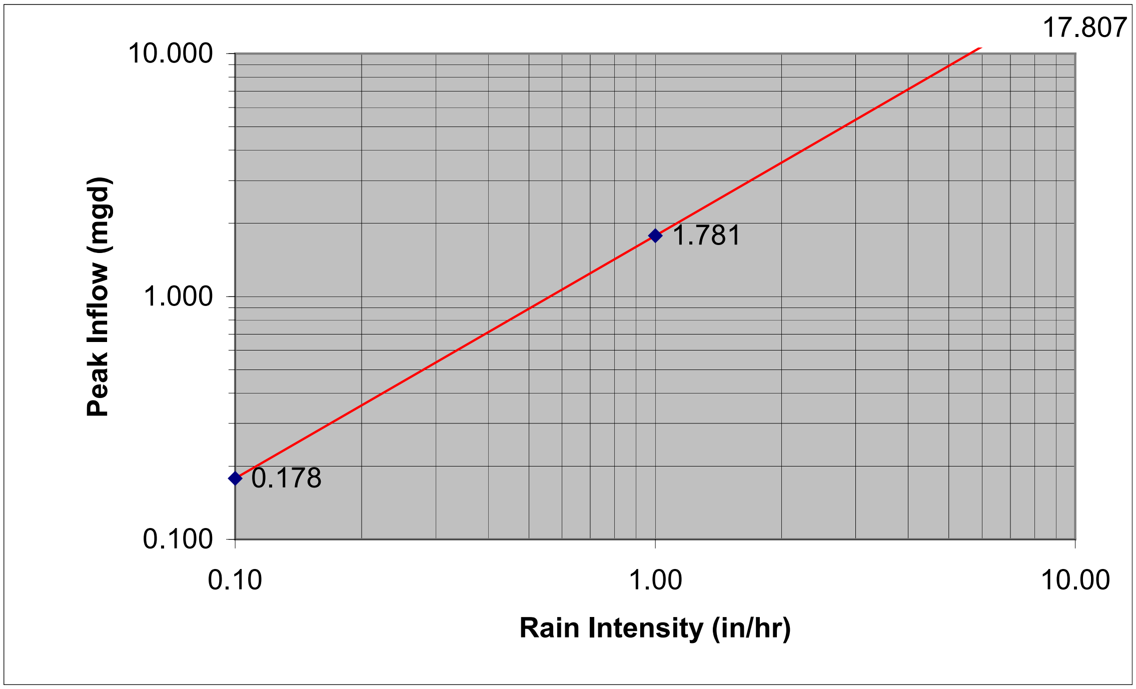


Figure V-14

Storm Intensity vs. Inflow – Meter No. 10



These various complicating factors resulted in the calculation of negative coefficient values for these locations. This problem is not uncommon in this type of analysis. In the case of Meters 3, 4, 5-1 and 11, observation and evaluation of the wet weather data resulted in the development of wet weather approximations that were unique to each of the four Service Areas:

- Service Area 3 – The Alexander Creek Interceptor was determined to be relatively unaffected by wet weather. However, wet weather flow from Service Areas 1 and 2 are substantial and is conveyed through the Interceptor, but no additional wet weather I&I was added within Service Area 3.

- Service Area 4 – *The Creekmore Study by Archer Engineers dated December 31, 2003* provided peaking factors that derive wet weather flows in this Service Area. These factors were applied, and peak wet weather flow rates from Owen-Good Pump Station were added to the flow as well to define peak flows through the Lampkins Fork Interceptor.

- Service Area 5 – Meter 5-1 was inserted into a relief line that did not exhibit I&I under most conditions. Therefore no I&I was added to this line and it was hydraulically connected to the main GRE Interceptor to carry excess flow. Results indicate it operates as a relief sewer and conveys only rainfall dependent inflow and infiltration flow.

- Service Area 11 – Wet weather flow from Service Areas 5&6 were routed through the system. Dry weather flows for Service Area 11 were calculated by averaging the shape of the dry weather hydrographs from Service Areas 5 and 6 and then adjusting this to represent a 100 gallons/capita/day contribution from the population of Service Area 11. Since the population projection for this area was based on the conservative estimate of 2.67 persons/home and 3 homes/acre for the 2025 simulation, the corresponding flow rates were deemed adequate for the purposes of our evaluation without additional wet weather contributions.

The combined wet and dry-weather hydrographs represent the flow within the collection system interceptors without any improvements or repairs to defects in the system. An I&I reduction program could reduce the amount of water entering the system by 35% to 45% during wet weather events. Due to uncertainties inherent in the flow monitoring process, and the difficulty of completely separating infiltration from base flow, it was presumed for the purpose of computer model inputs that I&I reduction efforts in Raymore would result in a 40% reduction in measured inflow during wet weather events. The effect of this 40% reduction would be to reduce the 5-year, 1-hour storm into the equivalent of a:

- 2-year, 2-hour storm;
- 5-year, 3-hour storm;
- 10-year, 4-hour storm; or a
- 25-year, 6-hour storm.

The results of the Phase 2 field investigation will be used to validate this assumption. If the Phase 2 results indicate that a 40% reduction is not possible, the modeling results will need to be adjusted accordingly.

Once calibration was complete, the model could then be used simulate various flow conditions for both dry and wet weather through the collection system interceptors. The results are normally generated as .txt files, which can be exported to other software packages for analysis. Other types of model output include graphic pipe profiles with hydraulic profile overlay. Summary data from the model output files are provided in Appendix C. These tables provide the peak anticipated flowrate that would need to be conveyed through the interceptors under year 2005, 2010, 2015, 2020 and 2025 wet weather conditions under both the mitigated and unmitigated I&I conditions. Evaluation of these simulation results is provided in Part VI of this report.

As with any software program, there are some limitations associated with the use of HydroWorks™, as well as limitations that are made in connection with the modeling process. Some of these limitations are:

- The program does not calculate or consider critical depth or hydraulic jumps within the collection system.

- The distribution of design rainfall is assumed to be uniform over all the Service Areas modeled. This assumption may be more accurate for storms of low intensity and long duration, and less so for storms of shorter duration and higher intensity. This assumption is also generally more valid for small watersheds such as those associated with the City of Raymore. Larger municipalities would experience more varied storm intensities over their collection systems.

- Assumptions have to be made regarding the condition of the collection system. These assumptions are often based on the results of sewer inspections or on the experience of the City's sewer maintenance crews. Individual sections of pipe may vary in condition, which would effect the friction losses experienced during both dry and wet weather flows.

- The HydroWorks™ software was developed in the United Kingdom, and as such, it utilizes flow units of m^3/s. The software uses a maximum of 3 significant digits, so 0.001 m^3/s is the minimum unit of flow available to the software to use. This translates into approximately 16 gallons per minute. Any difference less than this value is not considered in the simulation. For the current modeling effort involving the interceptors, this limitation should not pose a problem, as the larger diameter of gravity interceptors would not be sensitive to a flow increment of that size.

The results of the modeling are highly dependent on the presence of deficiencies within the collection system. There are several types of deficiencies typically found in gravity sewer systems. These deficiencies are common to a number of the interceptors that were modeled as part of this master plan are summarized as follows:

- Sewers with zero or negative slope as identified by the computer program or by measured inverts.

- Sewers with very flat slopes (less than 0.10 percent)

- Manholes or sewer segments where the incoming sewer or sewers discharge into a single sewer of equal or smaller diameter. This type of deficiency causes a hydraulic restriction in the interceptor's capacity.

- Sewers overloaded (pipe running completely full or surcharged) at peak dry-weather flow conditions.

- Manholes and/or piping with excessive infiltration due to structural defects.

C. MODEL RESULTS

The HydroWorks™ software simulated Scenario No. 2 for hydraulic conditions that were predicted for 2010, 2015, 2020 and 2025. Wet weather conditions for both the 5-year 1-hour storm as well as an assumed 40% reduction in the rainfall induced I&I resulting from the 5-year 1 hour storm were included in the model. In general, the following can be said regarding the results of the modeling:

- All of the Service Areas have adequate sewer interceptor capacity during dry weather events through year 2025.

- Lampkins Fork Interceptor has adequate capacity through year 2025 with a 40% reduction in rainfall induced I&I resulting from a 5-year 1-hour storm, but is under capacity by 2005 if I&I levels are not reduced .

- Alexander Creek Interceptor has adequate capacity through year 2025 with a 40% reduction in rainfall induced I&I resulting from a 5-year 1-hour storm, but is under capacity by 2005 if I&I levels are not reduced.

- The remaining interceptors and Service Areas will require varying levels of improvement to posses adequate capacity through 2025 under a 5-year 1-hour storm condition, even with a 40% reduction in rainfall induced I/I resulting from a 5-year 1-hour storm.

Details regarding each of the Interceptors are provided in the following paragraphs:

The Lampkins Fork Interceptor is undersized to adequately convey flow in all of the conditions that were modeled resulting from wet weather induced I/I. By 2020, the equivalent of an additional 8-inch sewer will be required to convey all of the unmitigated flow. However, with the presumed 40% reduction in inflow, the current interceptor has adequate capacity for all flows through 2025. See Figure V-15 for the layout of this interceptor. Figure V-16 provides a hydraulic profile of the Lampkins Fork interceptor under the 2025 60% wet weather I&I remaining condition.

The Alexander Creek Interceptor is undersized to adequately convey flow for all of the conditions resulting from wet weather induced I/I from a 5-year 1-hour storm that were modeled. The equivalent of a parallel 15-inch interceptor is currently required to convey all of this flow. This additional required capacity is the same through year 2025. This is mainly due to the fact that the wet weather contribution to the system is much larger than the dry weather flow. This condition was observed in all segments of the interceptor. By contrast, the 40% reduction in wet weather I&I results in effectively eliminating the need for additional capacity in this interceptor. 5-year, 1-hour storm wet weather conditions for all years can be conveyed by the existing interceptor if the reduction in I&I is achieved. Figure V-17 provides the required relief sewer diameters of this interceptor under the all 100% wet weather I&I conditions, while Figure V-18 provides the pipe size requirements for the 2025 60% wet weather I&I remaining condition. Figure V-19 provides the hydraulic profile of the

Alexander Creek Interceptor under the 2025 60% wet weather I&I remaining condition, showing the minor surcharging that was projected to occur within the system under those conditions.

The Good Ranch East (GRE) and GRE Relief Interceptors generally required some degree of improvement to convey any of the future wet weather flows for the 5-year, 1-hour storm event. The amount of additional capacity required by these two interceptors determined for the year 2005 is the same additional capacity that was projected to be required in the year 2025. The condition where 40% of the rainfall induced I/I resulting from a 5-year 1-hour storm is removed from the model resulted in a significant reduction in the required interceptor capacity. Although additional capacity was still required, it was significantly less and therefore significantly less costly to implement. Figures V-20 and V-20Aprovide the required relief sewer diameters for the GRE interceptor under the 5-year 1-hour storm, while Figures V-21 and V-21A provide the required relief sewer diameters under the 60% of the rainfall induced I&I resulting from the 5 year 1-hour storm.

The modeling results of the Good Ranch West Interceptor indicate that the GRW Interceptor will require increases in capacity to accommodate the 5-year, 1-hour storm under both the 100% and 60% wet weather I&I remaining conditions. The capacity needs for the 60% of the 5-year 1-hour storm are significantly less that under the 100% condition.

The West Interceptor required an immediate increase in capacity for the 5-year 1-hour storm event , but no increase in capacity was needed when the I&I was reduced to 60% of the 5-year 1-hour storm.

The required pipe diameters for the GRW and West Interceptors for the 2005-2010 model period under the 5-year 1-hour storm event are shown in Figure V-22. The required relief sewer diameters for the 2010-2015, 2015-2020 and 2020-2025 periods are shown in Figures V-23, 24 and 25, respectively. Figure V-26 provides the required relief sewer diameters for these interceptors under the 60% of the 5-year 1-hour storm. In this case, the additional pipe capacity needed does not change from 2005 to 2025.

*****

PART VI – EVALUATION OF ALTERNATIVES

A. GENERAL

Analysises of Scenario No. 1 and Scenario No. 2 are provided in this section of the report. Scenario No. 1 was evaluated to determine the most cost-effective means of conveying wastewater flow from the Expansion Areas to the Owen-Good Pump Station. Scenario No. 2 was evaluated to determine to what extent improvements must be made to each interceptor to accommodate future wastewater flows. Results from conveying both 100% and 60% of the rainfall induced I/I resulting from a 5-year 1-hour storm were developed for Scenario No. 2. Treatment and pumping alternatives for final disposal of the wastewater were evaluated as well.

B. POPULATION GROWTH DISTRIBUTION SCENARIO NO. 1

The general concept of Scenario No. 1 is for additional growth in the City to occur in areas south of the City limits, but mainly within the annexation limits (Expansion Areas A, B and C). Population and flow projections were based on all projected growth occurring in the Expansion Areas. Table II-7 provides the projected population and flows for this scenario. This scenario represents a worst-case condition for this proposed infrastructure.

Because there are no existing sewer interceptors within Expansion Areas A, B or C, development of a dynamic hydraulic computer model was not necessary. Sizing new interceptor sewers for these areas was accomplished as follows:

1. The general routes of interceptors within the drainage basins that make up each Expansion Area were determined through investigation of the surface topography, the location of existing pump stations operated by the City, and pumps stations operated by the Little Blue Valley Sewer District.

2. Each new interceptor was assigned a service area and ultimate population that it would serve. Each interceptor alignment was then divided into pipe segments between 1,000 ft and 2,000 feet in length, with each segment providing conveyance for a proportional amount of the total population to be served by that interceptor. This population was assigned using an assumed density of 3 homes per acre and 2.67 persons per home. This results in the ultimate population as described in Part II of this report.

3. Based on a flow contribution of 100 gallons per capita per day and a peaking factor of 3.0, the interceptors were sized using Manning's Equation. For the purposes of conservative design, a value of 0.013 was used for Manning's number, a value more in line with reinforced concrete or vitrified clay pipe rather than the smoother PVC or polyethylene. Use of a higher number allows for the future degradation of the pipe surface over time. The calculation began with the most upstream section of pipe. The calculated flow through that pipe was then added to the next section, in addition to it's own specific quantity of flow. Continuing this process downstream resulted in a calculated diameter for each interceptor, as the flow quantities grew cumulatively downstream.

4. Once the diameter for a full pipe was determined, this diameter was increased to allow for an 80% full condition, and then that diameter was increased if necessary to match one of the standard plastic pipe diameters.

This procedure was performed to calculate flows, size interceptor diameters, and ultimately size pump stations for drainage basins within all three Expansion Areas. Gravity interceptors were sized to accommodate flows for the projected ultimate or "build-out" population of each area. Pump stations and wet wells were sized for year 2025 projections. Three different collection system alternatives were evaluated using this procedure.

In addition, an Opinion of Probable Cost was provided for the total infrastructure needs of each alternative. Opinions of Probable Cost are based on the following:

1. Depth of bury for the interceptors averages 15 feet with the lowest 5 feet in rock. Blasting of rock is acceptable and debris disposal is no more than 5 miles from the construction site.

2. 3-phase power is available at sites near the relocated Highway 58 as well as at sites near Highway 291.

3. 3-phase power will be routed to Pump Station B-2 from relocated Highway 58 and Highway 291.

4. 3-phase power will be routed from Pump Station B-2 to Pump Station B-1.

5. Stand-by power will be required for all Pump Stations.

Each Expansion Area Alternative is described in the following paragraphs:

- Alternative 1: Figure VI-1 shows proposed improvements for Expansion Areas Service Alternative 1. In this Alternative, all of the flow generated in Expansion Area C would be pumped by a series of pump stations into the downstream end of Alexander Creek Interceptor and into the Raintree Pump Station. Flow from Expansion Area B would be pumped into the upstream end of the interceptor serving Expansion Area A. Flow collected in Expansion Areas A and B would be pumped to the Owen-Good Pump Station, where it would be pumped, along with wastewater from the Existing Service Area, to the Lampkins Fork Interceptor. The interceptor ultimately discharges into the Little Blue Valley Sewer District's collection system. Figure VI-1 shows the general layout of this Alternative, including potential locations of pump stations and interceptors. Table VI-1 provides a summary of recommendations for Alternative 1, including an opinion of probable cost. Table VI-2 provides more detailed force main information for Alternative 1, and Table VI-3 provides additional details regarding pump station design criteria for Alternative 1. Note that the infrastructure sizing provided in Table VI-1 is calculated for the ultimate population expected at build-out.

Table VI-2
Scenario No. 1
Alternative 1 Force Main Sizing

Force Main Name	Diameter (in)	Peak Flow (mgd)	Downstream Pump Station
C1-A	20	4.64	Raintree
C1	18	3.81	C1-A
C2	12	1.50	C1
B2	16	3.12	A1
B1	6	0.30	A1
A1	18	5.14	Owen Good

The force main diameters and the peak flow rates correspond to the 2025 condition. Ultimate population flow rates were not used, as this size of force main would result in unacceptably low velocities in the lines in the early years of operation. When the population exceeds the capacity of these pipelines, improvements will be required in the form of either additional force mains or the upsizing of the existing mains, potentially using trenchless techniques. Table VI-3 also utilizes the year 2025 flowrates for pump station size requirements.

Table VI-3

Scenario No. 1

Alternative 1 Pump Station Design Criteria

Pump Station Name	Average Flow (mgd)	Peak Flow (mgd)
C1-A	0.28	0.83
C1	0.77	2.31
C2	0.50	1.50
B2	1.04	3.12
B1	0.10	0.30
A1	1.71	5.14

Table VI-4 provides a proposed Capital Improvement Schedule and probable cost for Alternative 1.

- Alternative 2: Figure VI-2 shows proposed improvements for Expansion Area Alternative 2. Flows from Expansion Area C and Expansion Area B are both pumped directly into the upstream end of Interceptor A-1 in Expansion Area A. The wastewater flow from all three Expansion Areas is then pumped into the Owen Good Pump Station. The pump station then pumps this wastewater, along with wastewater from the Existing Service Area, into the Lampkins Fork Interceptor, which ultimately flows by gravity into the Little Blue Valley Sewer District's collection system. This Alternative results in the same interceptor sizing and pump station sizing for Expansion Areas B and C. Figure VI-2 shows the general layout of this alternative. Table VI-5 shows recommended improvements for Alternative 2, including an opinion of costs. Table VI-6 provides force main sizing for Alternative 2. Table VI-7 provides pump station design criteria for Alternative 2.

Table VI-6

Scenario No. 1

Alternative 2 Force Main Sizing

Force Main Name	Diameter (in)	Peak Flow (mgd)	Downstream Pump Station
A1	20	9.77	A1
B1	6	0.30	A1
B2	16	3.12	A1
C1-A	12	0.83	A1
C2	12	1.50	A1
C1	20	4.64	A1

Table VI-7

Scenario No. 1

Alternative 2 Pump Station Design Criteria

Pump Station Name	Average Flow (mgd)	Peak Flow (mgd)
A1	3.26	9.77
B1	0.1	0.30
B2	1.04	3.12
C1	0.77	2.31
C1-A	0.28	0.83
C2	0.50	1.50

Table VI-8 shows a recommended Capital Improvement Schedule and probable cost for Alternative 2.

- Alternative 3: Figure VI-3 shows proposed improvements for Expansion Area Alternative 3. In this Alternative, Expansion Areas B and C would be pumped into a combined force main that discharges directly to Owen Good Pump Station, while Expansion Area A would be pumped separately to Owen Good Pump Station. The force main sizes for Expansion Areas B and C remain unchanged from Alternative 2. Figure VI-3 provides a general layout for Alternative 3. Table VI-9 shows recommended improvements for Alternative 3, including an Opinion of Probable Cost. Tables VI-10 and VI-11 show the force main sizes for all the Expansion Areas, and pump station design criteria for all the Expansion Areas, respectively.

Table VI-10

Scenario No. 1

Alternative 3 Force Main Sizing

Force Main Name	Diameter (in)	Peak Flow (mgd)
A1	12	2.01
B1	6	0.30
B2	16	3.12
C1-A	12	0.83
C2	12	1.50
C1	20/24	4.64/7.76

Table VI-11

Scenario No. 1

Alternative 3 Pump Station Design Criteria

Pump Station Name	Average Flow (mgd)	Peak Flow (mgd)	Pumps into Force Main
A1	0.67	2.01	A1
B1	0.10	0.30	B1
B2	1.04	3.12	B2 / C1
C1-A	0.28	0.83	C1-A / C1
C2	0.50	1.50	C2 / C1
C1	0.77	2.31	C1

Table VI-12 shows a recommended Capital Improvements Schedule and probable cost for Alternative 3.

C. POPULATION GROWTH DISTRIBUTIONS SCENARIO NO. 2

Scenario No. 2 evaluated the concept that additional growth in the City would occur in two areas. The majority of the growth would occur within the Existing Service Areas that drain to the south, specifically Service Areas 5, 6, 7, 8, 9, 10 and 11. The remainder of the growth was assumed to occur in Expansion Area A, which would exhibit that same population and growth schedule as in Scenario No. 1 (population of 5,687 developing between years 2010 and 2015). Scenario No. 2 was developed to represent the worst-case scenario for improvements within the Existing Service Areas south of Highway 58, while still taking into consideration the likelihood of expansion and growth due to the relocation of Highway 58 in Expansion Area A. Part II of this report provided a detailed explanation of this Scenario. Detailed requirements for relief sewers are shown in Part V of the report. Cost evaluations are included in Tables VI-4, VI-8 and VI-12.

D. TREATMENT SYSTEM ALTERNATIVES

The City of Raymore is somewhat unique in that the city is divided geographically. Generally areas north of HWY 58 lie within the Little Blue Valley Sewer District (LBVSD). As such, any wastewater flows that are generated within the LBVSD must, by statute, continue to be served by the LBVSD. Areas generally south of HWY 58 lie outside the LBVSD service area and may be served by their own treatment facility. One of the fundamental issues that must be evaluated is the economics of continuing treatment service with the LBVSD versus developing a new treatment facility to serve the portion of Raymore that lies outside LBVSD service area. For the purpose of conducting this evaluation the anticipated cost of remaining in the LBVSD during the planing period was compared to the cost of either developing a new WWTF near the site of the Owen-Good Pump Station or developing a regional facility with the City of Belton, Missouri.

1. DEVELOPMENT COST FOR A NEW WWTF AT THE OWEN –GOOD PUMP STATION SITE.

This alternative requires the development of a wastewater treatment facility for Raymore, which would treat all of the wastewater generated from Service Areas 5 through 11, as well as future flows from Expansion Area A. This represents the modeled results from population distribution Scenario 2 which yielded flow rates based on all planning period development occurring in the existing service area south of HWY 58 plus Expansion Area A. The plant would be located near the south of the Owen Good Pump Station site and would consist of the following unit processes:

- Influent Structure and Screening
- Grit and Scum Removal
- Influent Flow Measurement and Monitoring
- Activated Sludge Facilities
- Effluent Disinfection
- Effluent Flow Measurement and Monitoring
- Sludge Thickening and Digestion
- Sludge Dewatering, Storage and Disposal

The treated and disinfected effluent would be discharged to East Creek. Based on the model results for Population Distribution Scenario 2, the following wastewater plant loadings were projected:

Table VI-13

Projected 2025 WWTP Loadings

Parameter	Average Monthly	Maximum Monthly	Peak Hourly
Flow, mgd	4.5	6.3	13.5
BOD_5, lbs/day	9,240	12,940	23,100
TSS, lbs/day	10,920	15,290	27,300
TKN, lbs/day	1,130	1,590	2,840

In order to construct and operate the conceptual WWTP, the City of Raymore would be required to obtain an NPDES permit from the Missouri Department of Natural Resources (MDNR). The permit would contain discharge limitations that would define the required level of wastewater treatment. Table VI-14 provides the NPDES Permit discharge limits applied to the City of Belton, which would likely be very similar to the limits placed on a facility to serve Raymore. The fecal coliform limit in the Table is the only item not taken from Belton's permit. This item is the water quality standard established by the EPA.

Table VI-14
Projected NPDES Permit Discharge Limitations

Parameter	Daily Maximum	Monthly Average
BOD_5, mg/l	20	20
TSS, mg/l	45	30
Ammonia-N, mg/l	4	4
Fecal Coliform, mpn/100 ml	200	-
PH, su	6.0 to 9.0	-

The plant would be designed as an activated sludge treatment plant, with disinfection most likely provided by ultraviolet (UV) radiation exposure. Using UV treatment eliminates the need for storage of chlorine compounds and simplifies the operation and maintenance of the disinfection system. The activated sludge design would include both aerobic and anoxic zones of treatment, which would allow for the removal of ammonia nitrogen. Table VI-15 provides general design parameters for the activated sludge processes, including basin volumes, clarifier diameters, and sludge pumping capacities.

Table VI-15

Activated Sludge Facility Parameters

Anoxic Basin Volume, gallons	1,370,000
Anoxic Average Hydraulic Retention Time, hrs.	7.3
Aeration Basin Volume, gallons	4,610,000
Aeration Average Hydraulic Retention Time, hrs.	24.6
Final Clarifiers' (2) Diameter, ft.	94
Final Clarifiers' Average Surface Overflow Rate, gpd/ft^2	320
RAS Pumping Capacity, gpm	4,700
WAS/Scum Pumping Capacity, gpm	320

Table VI-16 provides a preliminary opinion of cost for the construction, operation and maintenance of the proposed treatment plant through year 2025.

Table VI-16

Proposed Raymore WWTP Costs

	Unit	Quantity	Unit Cost	Total
Treatment Plant Construction	Gal	4,500,000	$3.75	$16,875,000
Land Acquisition	Acre	25	$9,000	$225,000
Site Power	LS			$125,000
5' Manhole	Each	17	$9,644	$163,948
Subtotal				$17,388,948
Contingency 20%				$3,477,790
Legal, Admin & Engineering Costs 20%				$3,477,790
Subtotal				$24,344,527
Annual O&M Cost				$550,000
Present Worth of Annual O&M				$6,308,445
Total 2005 Present Worth Basis				$30,652,972

2. REGIONALIZATION WITH THE CITY OF BELTON MISSOURI

Another Alternative is to convey a portion of the City's wastewater flow to the City of Belton's wastewater plant. Flow rates and service projections are assumed to be the same as for developing a new treatment facility at the Owen Good Pump Station site. This Alternative would require several new pieces of infrastructure as well as improvements to some existing facilities. These items include:

- A gravity interceptor measuring 48 inches in diameter and approximately 10,800 feet in length,
- A 24-inch force main, approximately 3,300 feet in length between the nearest pump stationwithin Belton's city limits and it's WWTP
- Improvements to increase the Belton pump station's pumping capacity.
- Improvements to increase the Belton WWTPs treatment capacity by 4.5 mgd.

In addition, costs for contributing to the operation and maintenance of the facility are included.

Table VI-17 shows the present worth of developing a regional WWTF with the City of Belton.

Table VI-17 Belton Expansion Option
Regionalization Alternative With the City of Belton
2005 Present Worth

	Item	Unit	Quantity	Unit Cost	Cost	
Owen Good Interceptor						
	36" Sewer	LF	3900	$333	$1,298,700	
	5' Manhole	Each	10	$9,644	$96,440	
Expansion Area "A" Interceptor	36" Sewer	LF	3900	$333	$1,298,700	
	5' Manhole	Each	10	$9,644	$96,440	
Belton Interceptor	48' Sewer	LF	10,800	$455	$4,914,000	
	6' Manhole	Each	27	$1,222	$32,994	
Belton Force Main	24" Force Main	LF	3250	$105	$341,250	
	Air Release	Each	3	$8,000	$24,000	
Upgrade Belton Pump Station		LS			$1,500,000	
Upgrade WWTP phase 1 year 2005 2.25 MGD		Per Gal	2250000	$3	$6,750,000	
Subtotal Phase I					$16,352,524	
Contingency @ 20%					$3,270,505	
Legal Engineering and Administrative @ 20%					$3,270,505	
Establishment of Regional Sewer District 7%					$1,144,677	
Subtotal Phase I					$24,038,210	
Upgrade WWTP Phase 2 Year 2015 2.25 MGD		Per Gal	2250000	$3	$6,750,000	
Contingency @ 20%					$1,350,000	
Legal Engineering and Administrative @ 20%					$1,350,000	
Establishment of Regional Sewer District 7%					$472,500	
Subtotal Phase 2					$9,922,500	
*Year 2005 PW of Phase 2 Improvements					$5,540,724	
Additional Annual O&M Cost						$650,000
**Year 2005 PW of Additional O&M Cost					$7,455,435	
Total Project 2005 Present worth Basis					$37,034,369	
* I = 6% n = 10 years						
** I = 6% n = 20 years						

3. CONTINUED SERVICE PROVIDED BY LBVSD

The option of continuing service provided by the LBVSD would require upgrading the Owen – Good Pump Station to pump the projected flows from Population Growth Distribution Scenario 2 to the Lampkins Fork interceptor. Additionally, per gallon service charges would apply to flows accepted by the LBVSD. The most significant cost of this alternative is the payment toLBVSD for conveyance and treatment of Raymore’s wastewater. Currently, the District charges the City a rate equal to the total operating cost of its facilities, divided by the proportion of total flow that each client contributes. For Raymore, the current cost is approximately $1015 per million gallons. Rate increases of 6% per year for each of the next 5 years are planned in order to pay for an expansion of LBVSD’s Atherton Wastewater Treatment Plant. Based on these rates, and the projected population, future billings for collection and treatment were projected for the planning period.

LBVSD service charges were projected by multiplying the yearly population projection for the City of Raymore by an average per capita flow contribution of 102 gallons per capita per day, then multiplying the projected service charge per gallon to the flow projection. Service charges were determined based on current billings and applying a 6% rate increase for each of the next five years beginning in year 2005. Costs were then reduced to present worth over the planning period. Table VI-18 shows the present worth calculation for the areas within the LBVSD and the total for the entire City of Raymore.

Continued Pumping into the LBVSD Collection System would require the upgrade and expansion of the Owen-Good pump station. Modeling shows that an average flow capacity of 5.0 mgd and a peak flow capacity of 11.3 mgd is required for the year 2005. This assumes that the   2-million gallon storage tank at the pump station remains in service. The existing 24-inch force main from Owen Good Pump Station to Lampkins Fork does not have the capacity to carry the entire 11.3 mgd peak flow. A parallel 24 inch forcemain would be required to carry the additional flow in this alternative. Table VI-19 shows the opinion of probable cost for expansion of the Owen- Good pump station

Table VI-19

Opinion of Probable Cost for OG Pump Station and Forcemain

2005 Present Worth

Capacity Upgrade From 3650 (5.25 mgd) to 7850 (11.3 mgd) firm Capacity

Item	Unit	Quantity	Unit Cost	Cost	
Parallel 24 Inch Force Main	LF	17000	$105	$1,785,000	
Air Release Station	Each	8	$8,000	$64,000	
Pump Station Upgrade	LS			$1,879,000	
Odor Control	LS			$150,000	
Site Electrical	LS			$80,000	
Subtotal				$3,958,000	
Contingency @ 15%				$593,700	
Legal Engineering & Admin @ 20%				$791,600	
Subtotal				**$5,343,300**	
Annual O&M Cost					$102,041
PW of Annual O&M Cost				**$1,170,400**	
Total project PW 2005 Basis				**$6,513,700**	

Table VI-20 shows the difference in cost between developing and operating a new WWTP at the Owen-Good pump station site versus continuing service with the LBVSD. Costs are shown in 2005 present worth.

Table VI-20 Treatment Alternatives
Present Worth of Continued Service to LBVSD vs City Owned WWTF

Alternative			
Service From LBVSD		**4.5 MGD WWTP**	
PW Annual Billings Total Service Area	$25,028,830	PW Annual Billings Total Service Area	$25,028,830
PW Capital & O&M for Owen-Good PS	$6,513,700	PW Annual Billings for Area South of HWY 58	($17,240,827)
		PW Capital & O&M for 4.5MGD WWTP	$30,652,972
	$31,542,530		**$38,440,975**

Based on the present worth cost projections provided in this section, the preferred wastewater treatment alternative is to continue pumping wastewater from all areas of Raymore to the LBVSD collection system. A present worth savings of $5,148,808would be gained by continuing to utilize LBVSD for service as opposed to developing a City-owned WWTP. In addition, it would prevent the additional administrative needs of a City-owned wastewater treatment plant that cannot easily be accounted for in financial terms. Compared to conveying some of the flow to Belton’s WWTP, there is a savings of $5,491,839 when the wastewater is conveyed to LBVSD.

E. OWEN-GOOD PUMP STATION EVALUATION

Based on the above-recommended treatment alternative, the Owen Good Pump Station will require modifications in order to continue serving the City of Raymore through Year 2025. Scenario No. 2, as previously described, presents the worst case for projected flow to Owen Good Pump Station, thus it was used to model future conditions. Flows to Owen Good Pump Station were evaluated through Year 2025 by combining outfall hydgrographs from the Good Ranch East and Good Ranch West drainage areas for dry, wet (5-year 1-hour storm), and mitigated I&I conditions (40% reduction in the wet weather induced I/I from a 5-year 1-hour storm). Additionally, flows from Expansion Area A were added for projected average dry weather and peak wet weather conditions. Projected flows to Owen Good Pump Station are summarized in Table VI-21.

Table VI-21

Projected Flows Into Owen Good Pump Station

Year	Average DWF (mgd)	Peak DWF (mgd)	Peak WWF (mgd)	60% Peak WWF (mgd)	OG PS Cap. Peak WWF 2 MG Storage
2005	1.500	2.488	14.859		8.736
2005-2010	2.810	3.537	17.504	12.871	11.254
2010-2015	3.550	4.564	18.532	13.898	-
2015-2020	4.290	5.546	19.490	14.880	-
2020-2025	5.030	6.573	20.449	15.907	-

The City of Raymore has committed to reduce, or mitigate, I&I. Thus, the Owen Good Pump Station will be exposed to the 60% Peak WWF as shown in Table VI-21. If there were no storage available, the pump station would be required to pump a peak flow of 15.907 million gallons per day in the Year 2025. Due to the two million gallons of storage presently available, the pump station size may be adjusted to account for stored peaks. A five-year, one-hour storm event was modeled under mitigated I&I conditions (60% Peak WWF) to evaluate the required future pump station capacity. If the total two million gallons of storage are utilized, the pump station would not likely have time to drain the basin if a storm of similar magnitude were to follow within a 24 hour period. If one million gallons of storage is utilized, the overflow basin would eventually overtop after several successive storm events.

Under a "balanced condition", the pump station could theoretically handle an infinite number of successive five-year, one-hour events. In order to determine the capacity adjustment resulting from storage use, the combined Year 2025 hydrographs were integrated over the pump station capacity. Figure VI-4 visually depicts the calculation required to determine the balanced condition for Year 2025 60% Peak WWF. The area, which equals total volume of flow, above horizontal pump station capacity line must be equal to the area below the horizontal pump station capacity line in order to have a "balanced" condition.

Based on the results of the analysis, a 11.3 million gallon per day capacity pump station is recommended. The 11.3 million gallon capacity pump station will be capable of handling peak unmitigated flows through the Year 2010 (11.254 mgd) using two million gallons of storage, and the peak mitigated flows through Year 2025 (11.289 mgd) for one million gallon of storage. This would provide the City of Raymore five years to mitigate the current I&I problem and provide enough capacity for projected Year 2025 peak mitigated flows.

The current pump station has a firm capacity of 5.25 million gallons per day. In order to accommodate future flows, an additional 6.05 million gallons per day of capacity will be required. Modifications to the existing pump station may include, but not be limited to, the addition of three pumps, an expansion of wet and dry well capacity, and the placement of a parallel 24-inch force main. Table VI-19 shows an opinion of probable construction and lifecycle cost for the expansion of the Owen-Good Pump Station.

The preliminary plan and section layout of the expanded Owen-Good Pump Station is shown on Figures VI-5 through VI-7. The schematic of the valves and piping shown in Figure VI-7 provides operational flexibility so that either one or both 24-inch force mains may be in use depending upon flow. The system head curve is different for either a single force main or both force mains operating. Friction losses for two force mains operating are less than the friction losses developed for a single force main operating. If only 1, 2, or 3 pumps are operating, only one force main will be required to convey the flow. If 4, 5, or 6 pumps are operating, then both force mains will be required to convey the flow. The preliminary operating curve for up to three pumps “ON” and one force main operating is shown in Figure VI-8. The preliminary operating curve for 1, 2, 3, 4, 5, and 6 pumps “ON” and both force mains operating is shown in Figure VI-9.

* * * * *

PART VII – CONCLUSIONS AND RECOMMENDATIONS

A. GENERAL

Part VII of the report reviews the alternatives evaluated in Part VI of this report and provides further discussion and recommendations regarding improvements, costs, scheduling and technical considerations for the proposed capital improvements.

B. CONCLUSIONS AND RECOMMENDATIONS

In response to the projected future growth of the City of Raymore, several alternative wastewater infrastructure expansion plans were considered. The population of Raymore, estimated to be 16,600 in 2005, is expected to increase to 48,340 by 2025. This population growth could occur in a number of areas both inside the City limits or in the expansion areas. Analyses were performed to determine the improvements to City's main sewer interceptors that would be required to convey wastewater generated by the year 2025 population regardless of where that population is centered. The analysis considered the worst case for each service area rather than conducting the analysis assuming the population was distributed uniformly across the expansion areas in the existing service areas.

The approach taken to size new interceptors considered the ultimate build out condition. However the build out condition for expansion areas was based on a lower population density (3 homes per acre and 2.76 persons per home). The requirements for the Owen-Good Pump Station to respond to this growth were also considered in the analysis.

The analyses considered both cost and technical components of each alternative. Similarly, an economic and technical analysis was performed for three different wastewater treatment alternatives for the City. Based on these analyses the following recommendations are made:

1. EXPANSION AREAS A, B, AND C

Alternative 3 is recommended for expansion areas A, B & C. In addition to being the lowest cost alternative, Alternative 3 has several other advantages.

- Alternative 3 minimizes the size of pump station A-1. Alternatives 1 and 2 require peak flow pump station capacities for pump station A-1 of 5.14 mgd and 9.77 mgd, respectively, as compared to 2.01 mgd for Alternative 3.
- Force main A-1 size is minimized thus reducing construction cost and reducing the potential for odors during the initial development of the expansion areas.
- Alternative 1 requires pumping of flows from Expansion Area C to the LBVSD pumpstation at Raintree Lake. The capacity of the Raintree Pump Station is reported to be 6.07 mgd. Flows generated from Expansion area C are estimated to be 4.64 mgd., leaving a capacity of 1.43 mgd for the Raintree and Middle Big Creek service areas of the LBVSD. Although detailed study of the LBVSD facilities is beyond the scope of this study, it is believed that the City of Raymore would incur substantial additional costs to upgrade the Raintree Lake Pump Station for Alternative 1 that are not included on Alternative 1 costs. Expansion of the Raintree Lake pump station is not required for Alternatives 2 or 3.
- Alternative 3 provides for collection of all flows generated outside of the LBVSD at the Owen-Good Pump Station. This provides the City of Raymore the flexibility to either provide their own treatment facility or to consolidate wastewater treatment service with the City of Belton in the future.

Figure VII-1 summarizes the recommend improvements for Expansion Area Service Alternative 3. Table VII-1 shows the Opinion of Probable cost for Expansion Area Service Alternatives 1 through 3 and the estimated per connection cost of improvements.

Table VII-1 Comparison of Expansion Service Area Alternatives

Expansion Area Service Alternative	Opinion of Probable cost	Per Service Connection Cost
1	$28,231,000	$2,257
2	$30,261,000	$2,419
3	$27,459,000	$2,195

2. WASTE WATER TREATMENT:

Several alternatives for waste water treatment were evaluated for the City of Raymore, they included:

- Remaining in the LBVSD and Pumping all flows generated in the service areas outside the LBVSD service area to the Lampkins Fork Interceptor.
- Developing and maintaining a new City owned WWTF at the Owen-Good Pump Station site.
- Consolidating Service with the City of Belton for wastewater treatment.

The analysis developed opinions of probable cost for each of these alternatives and compared them on a present worth basis. Table VII-2 illustrates the comparison.

Table VII-2 Comparison of Present Worth of Treatment Alternatives

Alternative	Present Worth
Continued Service by LBVSD	$31,542,530
City Owned WWTF	$38,440,975
Consolidation With the City of Belton	$37,034,369

Based on the analysis described is Part VI and a comparison of the treatment alternatives it is recommended that The City of Raymore continue to purchase treatment service from the LBVSD. It is also recommended that the City of Raymore reevaluate this issue on a regular basis as increasing rates may occur. A change in the service rate from LBVSD, depending on its magnitude, may affect the cost balance and make one of the other treatment alternatives compare more favorably.

3. OWEN-GOOD PUMP STATION AND FORCE MAIN

The model results predict that the Owen-Good Pump Station will experience average dry weather flows of 5.0 mgd by year 2025 with diurnal dry weather peaks of 6.6 mgd. During the 5-year one hour rainfall event it is anticipated that the station will receive peak flows of 20.4 mgd, if no I/I reduction program is instituted. With an aggressive I/I reduction program it was assumed that the wet weather peak can be reduced to 15.9 mgd. Given that the station has a 2-million gallon extraneous holding basin the peak flow could be reduced to 11.3 mgd with a reserve of 1 million gallons of storage. Allowing a reserve of 1 million gallons would allow flexibility in operation and provide for some margin of safety in the event of consecutive storms over a short period of time.

The existing force main does not have capacity to convey 11.3 mgd. Therefore, a parallel force main is recommended. A parallel 24-inch force main would provide the needed capacity and would provide for a reserve of approximately 2.2 mgd in the event that future expansion is necessary.

Under current conditions the model predicts that the Owen-Good pump station and storage systems firm capacity would be exceeded for a 5-year 1-hour rainfall event. The firm capacity of the station is 5.25 mgd, the model predicts that a firm capacity of 8.7 mgd is required. It is therefore recommended that planning for the upgrade of the Owen-Good Pump station begin in the near future. Table VII-3 shows the opinion of probable cost for upgrading the pump station.

Table VII-3
Opinion of Probable Cost for OG Pump Station and Forcemain
2005 Present Worth
Capacity Upgrade From 3650 (5.25 mgd) to 7850 (11.3 mgd) firm Capacity

Item	Unit	Quantity	Unit Cost	Cost	
Parallel 24 Inch Force Main	LF	17000	$105	$1,785,000	
Air Release Station	Each	8	$8,000	$64,000	
Pump Station Upgrade	LS			$1,879,000	
Odor Control	LS			$150,000	
Site Electrical	LS			$80,000	
Subtotal				$3,958,000	
Contingency @ 15%				$593,700	
Legal Engineering & Admin @ 20%				$791,600	
Subtotal				**$5,343,300**	
Annual O&M Cost					$102,041
PW of Annual Power Cost				**$1,170,400**	
Total project PW 2005 Basis				**$6,513,700**	

4. LAMPKINS FORK INTERCEPTOR

The Lampkins Fork interceptor was modeled for peak flows in Service Area 4, including the proposed Creekmore development. In addition, the capacity of the interceptor was checked for the addition of up to 18.5 mgd contributed from the Owen-Good Pump Station. The model predicted only mild surcharging of limited segments of the interceptor for that condition. Since the recommended capacity of the Owen-Good Pump Station is only 11.3 mgd the capacity of the Lampkins Fork Interceptor is deemed to be adequate.

5. ALEXANDER CREEK INTERCEPTOR

The Alexander Creek interceptor was modeled for a 5-year 1-hour rain event. The model predicted that overflows would occur for the 5-year 1-hour rain event. The model was repeated assuming that I/I reduction of 40% could be achieved by an aggressive I/I reduction campaign in service areas 1 and 2. The results of the modeling assuming 40% I/I reduction showed only minor surcharge under wet weather conditions. Hydraulic profiles for both conditions are shown in Appendix C. It is therefore recommended that the City pursue an aggressive I/I reduction campaign in Service Areas 1 & 2.

6. SERVICE AREAS 5 & 6

The Good Ranch East (GRE) interceptor system was modeled for both dry and wet weather conditions. For dry weather conditions, the interceptor system has adequate capacity to handle all anticipated conditions. However, the 5-year 1-hour storm event results in significant overloading of the interceptor system from manholes GE20 to GE6, WGE7 to GE6 and G6 to GE1. Figures VII-2 through VII-3a show the required relief line sizes for the 5-year 1-hour storm and 40% reduction of rainfall induced I/I resulting from the 5-year 1-hour storm. In some cases, the model projected adding a third parallel relief line where two lines already exist. In practice it may be more desirable to remove one or both of the existing lines and replace them with an appropriately sized line, this matter should be resolved in the final design phase. Table VII-4 shows the required relief line sizes and opinion of probable cost for the relief sewers for the 5-year 1-hour storm and 40% reduction in I/I induced by the 5year 1-hour storm for Service Areas 5 & 6. Detailed analysis can be found in Appendix C.

Table VII-4
Relief Summary for GRE and GRE Relief Interceptors

Pipe ID			5-year 1-hour Storm		60% 5-year 1-hour Storm	
			Relief sewer		Relief sewer	
Conduit	Downstream	Length	2020-2025	Projected	2020-2025	Projected
ID	Manhole	(ft)	Req'd Dia. (in	Cost ($)	Req'd Dia. (in	Cost ($)
WGE7.1	WGE6	400	10	$23,600	10	23600
WGE6.1	WGE5	400	8	$19,600	NA	$0
WGE5.1	WGE4	400	8	$19,600	NA	$0
WGE4.1	WGE3	358	8	$17,542	NA	$0
WGE3.1	WGE2	364	8	$17,836	NA	$0
WGE2.1	WGE1	230	8	$11,270	NA	$0
WGE1.1	GE6	213	8	$10,437	NA	$0
GE22.1	GE21	171	15	$12,654	12	$10,602
GE21.1	GE20	400	15	$29,600	12	$24,800
GE20.1	GE19	331	15	$24,494	12	$20,522
GE19.1	GE18	315	15	$23,310	12	$19,530
GE18.1	GE17A	400	15	$29,600	12	$24,800
GE17A.1	GE17	400	15	$29,600	12	$24,800
GE17.1	GE16	85	15	$6,290	12	$5,270
GE16.1	GE15	299	15	$22,126	12	$18,538
GE15.1	GE14	299	15	$22,126	12	$18,538
GE14.1	GE13	249	15	$18,426	12	$15,438
GE13.1	GE12	299	15	$22,126	12	$18,538
GE12.1	GE11	390	10	$23,010	8	$19,110
GE11.1	GE10	394	10	$23,246	8	$19,306
GE10.1	GE9	397	10	$23,423	8	$19,453
GE9.1	GE8	299	10	$17,641	8	$14,651
GE8.1	GE7	308	10	$18,172	8	$15,092
GE7.1	GE6	331	10	$19,529	8	$16,219
GE6.1	GE4	233	12	$14,446	8	$11,417
GE4.1	GE3	230	12	$14,260	8	$11,270
GE3.1	GE2	400	12	$24,800	8	$19,600
GE2.1	GE1	302	12	$18,724	8	$14,798
Total				**$557,488**		**$385,892**

Table VII-4 shows that the difference in relief sewer cost between a 40% reduction in I/I and no reduction in I/I is $171,600. The City is currently beginning a program to clean, televise and rehabilitate older portions of the collection system in the "Old Town" and Preakness Drive Area. It is therefore recommended that the City install relief lines based on the 40% reduction of I/I and pursue an aggressive I/I reduction program in Service Areas 5 & 6.

The model recommended that relief capacity required from GE21 to GE1 range from 15-inch diameter pipe to twelve-inch diameter pipe for the 100% I/I remaining condition. The model recommended that relief capacity required from GE21 to GE1 range from 12-inch diameter pipe to 8-inch diameter pipe for the 60% I/I remaining condition. Typical engineering practice is to either maintain a consistent or an increasing pipe diameter from upstream to downstream, but never to decrease pipe diameter.

Also, the model has suggested relief sewer where relief already exists, which would result in three lines in parallel in some places. Three lines in parallel may result in maintenance issues. A better option may be to excavate the line in the worst condition and replace it with a new line that would provide required combined capacity. One of the existing lines would remain in place to provide uninterrupted service through the construction period and additional relief upon completion of the project. Thus, only two lines would be in place rather than three.

Furthermore, relief will likely need to extend to the manhole immediately north of manhole GE7, which was not modeled due to lack of information. The above-mentioned items should be considered in the final design of the improvements to increase capacity. Due to the urgency of the I/I issue in Service Areas 5 & 6 and to insure that the relief projects for Areas 5 & 6 will be properly funded, the 100% I/I remaining relief cost will be used in the development of the CIP.

7. SERVICE AREAS 7, 8, 9 & 10

The Good Ranch West (GRW) interceptor system was modeled for both dry and wet weather conditions. For dry weather conditions, the interceptor system has adequate capacity to handle all anticipated conditions. However, the 5-year 1-hour storm event results in significant overloading of the interceptor system from manholes GW15 to GW1a, and GRW18 to GRW17. Figures VII-4 through VII-5 show the required relief line sizes for the 5-year 1-hour storm and 40% reduction of rainfall induced I/I resulting from the 5-year 1-hour storm. Table VII-5 shows the required relief line sizes and opinion of probable cost for the relief sewers for the 5-year 1-hour storm and 40% reduction in I/I induced by the 5year 1-hour storm for Service Areas 7, 8, 9, & 10. The difference in relief sewer cost between 40 % reduction in I/I and no reduction is $478,000. Detailed analysis can be found in Part V of the report.

Table VII-5
Relief Summary for GRW, GRW Relief, and West Interceptors

Pipe ID			5-year 1 hour Storm		60% 5-year 1-hour Storm	
			Relief Sewer		Relief Sewer	
Conduit	Downstream	Length	2020-2025	Projected	2020-2025	Projected
ID	Manhole	(ft)	Req'd Dia. (in)	Cost ($)	Req'd Dia. (in)	Cost ($)
GW14.1	GW13	381	15	$28,194	8	$18,669
GW13.1	GW12	138	15	$10,212	8	$6,762
GW12.1	GW11	272	15	$20,128	8	$13,328
GW11.1	GW10	285	15	$21,090	8	$13,965
GW10.1	GW9	400	15	$29,600	8	$19,600
GW9.1	GW8	463	15	$34,262	8	$22,687
GW8.1	GW7	384	15	$28,416	8	$18,816
GW7.1	GW6	118	15	$8,732	8	$5,782
GW6.1	GW5	348	15	$25,752	8	$17,052
GW5.1	GW4	404	15	$29,896	8	$19,796
GW4.1	GW3	364	15	$26,936	8	$17,836
GW3.1	GW2	299	15	$22,126	10	$17,641
GW2.1	GW1	410	15	$30,340	10	$24,190
GW1.1	GW1A	217	15	$16,058	10	$12,803
GRW17.1	GRW16	400	12	$24,800	NA	$0
GRW16.1	GRW15	400	12	$24,800	NA	$0
GRW15.1	GRW14	299	12	$18,538	NA	$0
GRW14.1	GRW13	299	12	$18,538	NA	$0
GRW13.1	GRW12	354	12	$21,948	NA	$0
GRW12.1	GRW11	299	12	$18,538	NA	$0
GRW11.1	GRW10	400	12	$24,800	NA	$0
GRW10.1	GRW9	394	12	$24,428	NA	$0
GRW9.1	GRW8	253	15	$18,722	NA	$0
GRW8.1	GRW7	262	15	$19,388	NA	$0
GRW7.1	GRW6	436	15	$32,264	NA	$0
GRW6.1	GRW5	246	15	$18,204	NA	$0
GRW5.1	GRW4	305	15	$22,570	NA	$0
GRW4.1	GRW3	194	15	$14,356	NA	$0
GRW3.1	GRW2	328	15	$24,272	NA	$0
GRW2.1	GRW1	410	15	$30,340	NA	$0
GRW1.1	GRWI7	259	15	$19,166	NA	$0
Total Cost				**$707,414**		**$228,927**

The analysis also revealed that there is substantial dry weather infiltration originating in Service Areas 9&10. Figure VII-6 is a photograph showing active infiltration during dry weather at manhole GW18. In all probability the infiltration is due to defects in pipes and manholes coupled with high groundwater induced by Silver Lake.

Figure VII-6 - Active Infiltration at Manhole GW18

It is recommended that the City install the required relief sewers based on maintaining a 40% reduction in rainfall induced I/I and begin a program of smoke testing, manhole inspections and CCTV Silver Lake area to identify sources of I/I.

The model has suggested relief sewer where relief already exists, which would result in three lines in parallel in some places. Three lines in parallel may result in maintenance issues. A better option may be to excavate the line in the worst condition and replace it with a new line that would provide required combined capacity. One of the existing lines would remain in place to provide uninterrupted service through the construction period and additional relief upon completion of the project. Thus, only two lines would be in place rather than three. This should be considered in the final design of the improvements to increase capacity. Budgetary projections for the CIP will be based upon the 40% reduction in rainfall induced I/I.

8. SERVICE AREA 11

Specific recommendations for improvements to Service Area 11 associated with wet weather conditions are discussed above in conjunction with Service Areas 5 & 6. Additional improvements include decommissioning the pump station located near Oak Drive, north of Hubach Hill Road, and installing a gravity interceptor to connect to the GRE interceptor just downstream of manhole GE1. The interceptor is designated as Interceptor SW-1. Figure VII-1 shows the approximate alignment and sizing of the recommended interceptor and Table VII-6 shows the opinion of probable cost.

Table VII-6
Shadow Wood Interceptor Opinion of Probable Cost

	Item	Unit	Quantity	Unit Cost	Cost
Interceptor SW-1	12-inch		1500	$62	$93,000
	18-inch		1500	$87	$130,500
	21-inch		5250	$105	$551,250
	48-inch manhole		23	$4,500	$103,500
Subtotal					$878,250
Contingency 15%					$131,738
Engineering Admin Legal 20%					$175,650
Total interceptor SW-1					**$1,185,638**

9. REDUCTION OF WET WEATHER FLOWS

Reduction of wet weather flows benefits the City of Raymore in several distinct ways. The most dramatic effect are to reduce the potential cost of relief sewers required to convey wet weather flows and reduced flow volumes to LBVSD with corresponding lower treatment costs. Table VII-7 shows the difference in cost between installing relief sewers to convey the 5-year 1-hour storm compared to the cost of installing relief interceptors to convey 60% of the rainfall induced I/I from the 5-year 1-hour storm.

Table VII-7 Comparison of Relief Sewer Cost

Interceptor	Cost of Relief Sewer for 5-year 1 hour Storm	Cost of Relief Sewer for 60% of The 5-year 1 hour Storm	Savings
Lambkins Fork	$0	$0	$0
Alexander Creek	$2,007,000	$0	$2,007,000
GRE	$558,000	$386,000	$172,000
GRW	$707,000	$229,000	$478,000
Totals	**$3,272,000**	**$615,000**	**$2,657,000**

The most dramatic impact, savings of $2 million, can be seen on the Alexander Creek Interceptor. Discounting the Alexander creek interceptor, a savings of $650,000 could be expected from the GRE and GRW interceptors alone. These savings would be partially offset by the costs to correct I/I problems.

In addition to reducing the cost of relief sewers, it is anticipated that the LBVSD will be upgrading metering stations on the Lampkins Fork and Alexander Creek Interceptors. The upgraded metering stations will provide the District with capacity to more accurately measure peak wet weather flows. As the District's ability to measure peak flows improves, peak flows that are not currently metered will be accounted for, and total volumes will likely increase. Over time, this should provide incentive to the City to minimize extraneous flows.

Reducing extraneous flows in the system also in effect "buys back" capacity in the existing system for future expansion that would otherwise be needed for conveyance of extraneous flows.

Part IV of the report details recommendations for proceeding with a Phase II investigation to identify extraneous sources of flow and to validate the assumption that a 40% reduction of extraneous flows can be accomplished. Table VII-8 provides a synopsis of the recommended actions. Figure VII-7 shows specific areas where smoke testing, manhole inspections and television inspection is recommended.

Table VII-8

Recommended Phase 2 Activities for Each Service Area.

Service Area	Manhole Inspections (each)	Smoke Testing (LF)	CCTV* (LF)
1	171	42,740	42,740
2	24	13,185	N/A
4	78	16,554	21,187
6	50	12,956	12,956
7	18	4,400	N/A
8	51	15,300	N/A
9	4	48,050	12,681
10	45	18,240	18,240

*: CCTV inspections to be performed by City staff.

10. SCHEDULE OF RECOMMENDED CAPITAL IMPROVEMENTS

Based on the recommendations described above and phased development of Expansion Areas A, B, and C, a schedule of recommended capital improvements was prepared. The schedule is based on pursuing Expansion Area Service Alternative 3, and assumes that a reduction of extraneous flows of 40% is achievable. The recommended capital improvements schedule may need to be adjusted pending the outcome of the Phase II study. Table VII-9 shows the capital costs for specific projects and the time frame for which the projects should be executed. Figure VII-1 shows the recommended improvements.

APPENDICES

APPENDIX A –

Updated City Specifications and Standards

APPENDIX B –

Wade & Associates Report May 2003

Wade & Associates Report August 2003

APPENDIX C –

Modeling Results