City of Galt Wastewater Collection System Capacity Analysis Phase I

City of Galt

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SC-G21-110-10

October 2005

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Chapter 1
INTRODUCTION

The objective of this capacity analysis project is to assess the capacity of the City of Galt's (City) existing wastewater collection (sanitary sewer) system (the System) and provide planning for future developments. In the process, initial work will identify short-term urgent or critical nature improvements while also assessing the improvements needed to accommodate build-out within the existing City boundary. The work sequence followed in this sewer system capacity analysis includes:

- Information collection
- Analysis criteria adaptation and determination of loading factors
- Network setup
- Computer model analysis
- Analysis results evaluation, and
- System improvement recommendation

Information collection – The basic information required for the System's capacity analysis includes: the City's General Plan, the City's Wastewater Collection System map (WCS Map), and residential housing development as-built drawings. From these collected sources, the operational characteristics of the System, which is the primary existing conditions of the sewer pipes in the ground, are identified. These operational characteristics can be obtained from the development as-built drawings and they include the respective pipe diameters, the lay length between manholes, the pipe invert elevations at the manholes, pipe material and their approximate years of service.

Network set up – For creating a sewer system analyzing model, a network of primary and secondary sewer lines flowing from the defined tributary areas will need to be identified to form the model system network. The City's WCS Map has a complete layout of its constructed and to be constructed sewer lines in the City. It covers all System components ranging from the 6-inch line from a cul-de-sac to the primary collector that discharge into a pump station. Boyle will review the sewer system master plan obtained from the City, and identify those sewer lines that are primary and secondary collection sewer mains to form a network for input into the computer model for analysis. For this study, pipes 8-inch and larger will be identified as either a primary or secondary collector and included in the model. However, 6-inch sewer lines that are considered as secondary collectors or as having existing problems will also be included when information is available.

Loading factor determination – The flow collected into the sewer lines of a network will be from a tributary area assigned to a manhole in the network. The amount of flow from the tributary areas is quantified using the criteria usually set by a city or its associated county. The City utilizes its own development standards (Galt Code, Table 18.20-1) and also the Sacramento County Improvement Standards for estimating sewage flow from its new developments. The criteria set forth in the Sacramento County Improvement Standards, 1999, Section 7 -Sanitary Sewer Design (SCIS) have been used by developers to design collection sewers for new housing or commercial developments. For the system analysis of this project, based on the SCIS flow generation standards, flow estimation loading factors will be generated for the computer model. These loading factors will be used in the computer

model to determine the quantity of sewage flow to be collected from the various tributary areas as designated in the City's General Plan. Other key design/analysis criteria including, depth to Diameter (d/D) ratio, pipe friction coefficients, peaking factor, and rainfall dependant Inflow and Infiltration (I/I) will also be adopted in the model analysis. The d/D ratio criterion will be input into the computer model as an evaluation factor in determining the deficiency of each sewer segment. It will not be used in flow generation calculation.

Computer model analysis – The computer model selected for the System analysis is InfoSewer-Pro by MWH Soft® (the Model). The identified sewer network, sewer pipe characteristics, flow generation loading factors, and other necessary criteria will be input in the model for capacity analysis. The development of the model will be discussed in a following chapter.

Analysis results evaluation – The results of the model analysis will be presented in spreadsheet tables and deficient sewer segments will be highlighted on a network drawing. Based on the results, Boyle will evaluate the urgency of providing improvement to the identified deficient sewer segments. The most urgent improvement could certainly be for the segments within known problem areas. Detailed evaluations of the deficient segments will be presented in a following chapter.

System improvement recommendations – The final chapter of this report will be devoted to improvement recommendations. The segments requiring improvement will be tabulated according to their priority along with a preliminary budget estimate for their construction. According to the City, the work of this report is considered as Phase 1 of a citywide wastewater collection improvement/upgrade plan. The follow-up Phase 2 Project will incorporate the City's future updated General Plan along with their revised Zoning Map which may include an expanded city limit. It will address the future expansion of the City and will identify the necessary sewer mitigation projects needed to support the planned growth. The Phase 2 project should include cost estimates for the identified sewer mitigation projects.

This Phase I draft report is organized into the following Chapters:

- Chapter 1 – Introduction
- Chapter 2 – Background Conditions
- Chapter 3 – Analysis Criteria
- Chapter 4 – Initial Flow Generation
- Chapter 5 – Sewer Model Development
- Chapter 6 – System Recommendations

Chapter 2
BACKGROUND CONDITIONS

LAND USE

The City of Galt is located in California approximately 25 miles south of Sacramento (**Figure 1**). As of 2004, the City has a population of approximately 22,231. According to the current city limit boundaries, the City is approximately 90 percent developed with 67 percent of the land being utilized for residential housing and 33 percent for public use, open space, commercial, office professional, light manufacturing, and industrial use. A large portion of developed residential dwellings in the City consists of maximum-density and single-family dwellings. The City's General Plan (1999-2005, **Figure 2**) includes zoning maps indicating how areas of the City are developed within the current City limits boundary. The City is currently in the process of revising their General Plan. The revision will most likely address the planned development in the underdeveloped areas. The impact from the planned future development to the System will be limited to the primary and secondary collection sewers that are expecting to receive flows from the revised undeveloped areas. The system improvements needed to accommodate the proposed developments per the revised General Plan should be addressed in Phase 2.

THE SYSTEM

The entire population of the City of Galt is served by the System of gravity sewer lines with intermediate lift stations. Boyle is not aware of any development within the City that is presently using septic system to dispose its wastewater water. The System network of sewer lines extends from boundary to boundary of the current city limits. The sewer line diameters range from the smallest at 6 inches to the largest at 24 inches. Their years of services date back to a time before the initial construction of the McFarland Lift Station in 1965, according to record drawings of the City. Prior to 1998, the primary material used to construct sewer mains in the City was Vitrified Clay Pipe (VCP). After 1999, polyvinyl chloride (PVC) pipes were the preferred material for sewer lines. Newer subdivisions being constructed in the City are using SDR 35 PVC pipes for the sewer mains due to their relative cost effectiveness, low roughness, and ease of installation. Force mains in the City have historically been constructed using asbestos concrete (AC) pipe. Because of the hazardous nature of asbestos, all new force mains constructed in the City are required to utilize non-asbestos based pipe. Presently, there are two major sewage force main in the City. The first major sewer force main is the 14" AC pipe sewer force main used to deliver pumped wastewater from the A Street and McFarland lift stations to the Live Oak Lift Station. The second major sewer force main is the 16-inch force main that conveys the pump flow from the Live Oak Lift Station to the City's wastewater treatment plant, where the incoming flow is recorded and treated before disposal at the City's reclamation fields.

Over the years the City has identified specific problem areas where wastewater has spilled out of the system. The five major problem areas involve sewer segments in:

1) Quail Hollow Drive, Lorna Way, and Trudy Way to McFarland Lift Station;
2) Meadowview Developments to First Street Lift Station;

3) Galt Market area to Chabolla Lift Station;
4) Fourth Street, Spruce Avenue, and Oak Avenue all North of A Street; and
5) F Street between First Street and Third Street.

These major problem area sewer segments will be included in the network for model analysis. The short segment in Meladee between Lincoln and Glendale Avenue was not included in the model due to lack of invert information and it will be addressed separately.

SANITARY LIFT STATIONS

The City of Galt is relatively flat topographically. Eleven (11) pump stations have been constructed to support the gravity sewer system and lift the collected sewage to avoid excessively deep sewer construction. The locations of the pump stations are indicated on **Figure 2**, The City of Galt Zoning Map. Collectively, there are three tiers of pumping. The first tier of pumping includes seven (7) smaller pump stations:

- Crystal Lift Station with two 300 gpm, 3 hp pumps.
- Chabolla Lift Station with two 300 gpm, 3 hp pumps.
- Kost Lift Station with two 380 gpm, 5 hp pumps.
- First Street Lift Station with one 5 hp and one 7.5 hp pumps. Each pumps about 250 gpm.
- E Street Lift Station with two 500 gpm, 3 hp pumps.
- Sparrow Lift Station with two 700 gpm, 15 hp pumps.
- Elm & Amador Lift Station with one 3 hp and one 6 hp pump. Each has a pumping rate of about 500 gpm.

These first-tier pumps in general lift and dump their respective flow into nearby gravity sewer manholes ranging from 17 to 400 feet away. The first tier pump flows along with the additional flow collected on the way by the primary sewers eventually are discharged into one of the second tier pump stations that include:

- "A" Street Lift Station, which has two 40 hp pumps and receives flows from four first-tier lift stations - Chabolla, Kost, First Street and "E" Street. Each pump can pump 1,050 gpm at 94.5 TDH and 1,800 gpm at 66 ft TDH. Pumped flow is delivered to the Live Oak Lift Station by a 14-inch force main.
- McFarland Lift Station receives flows from three first-tier lift stations – Crystal, Sparrow, and Elm & Amador. A new submersible pump station is being constructed near the existing station by the Schmidt Ranch developer to receiving additional flow from a new residential development. The new rated capacity of the McFarland Lift Station will be 885 gpm. There will be two submersible pumps of 885 gpm at about 80" of TDH. The pumps will discharge into the same force main that "A" Street Lift Station uses to deliver the pumped flow directly to the Live Oak Lift Station
- Vintage Lift Station has two 18 hp pumps. This pump station does not receive any first-tier pump flows. It is considered as a second tier lift station because it discharges directly into the Live Oak Lift Station. The pump station has two submersible pumps, and each pump can pump approximately 1,500 gpm at 33 ft TDH.

Live Oak Lift Station is the only third tier lift station in the City. It has two 3,080 gpm (at 70 ft TDH) 77 hp pumps located at the intersection of Live Oak Ave and McFarland. It receives all the sewage flow generated in the City and pumps it to the Wastewater Treatment Plant for treatment.

For the capacity analysis of this project, it is assumed that each pump station has or will have a pumping capacity that equals the Peak Wet Weather Flow (PWWF) rate estimated by the model for the pump station. This assumption will provide continuation of flow in the model analysis. It is also assumed that the firm maximum pumping capacity of the pump station will maintain the water level inside the wet well sufficiently low such that no backwater effect will be exerted on the influent sewer.

The computer model will calculate the total flow to be discharged into each of the existing lift stations at current and future built-out conditions as shown in the current General Plan. The existing pump stations that do not have the pumping capacity to handle the calculated flow will require future upgrade improvements to cope with the new flows. They will be identified as one of the City's future capital improvement projects.

Chapter 3
ANALYSIS CRITERIA

SACRAMENTO COUNTY IMPROVEMENT STANDARDS

Again, the City of Galt has adopted the SCIS as the sewer design guidelines for new developments in the City. Developers are required to design the sewer system of their development based on the SCIS criteria. The SCIS specifies a method to calculate the average daily flow (Average Dry Weather Flow, ADWF) from service areas. Based on the development density, each service area SCIS assigns a certain number of equivalent single family dwelling units (ESD) and classify them as follows: Low Density Residential, Medium Density Residential, Commercial/Industrial, and Transit Oriented Development. The corresponding flow (ADWF) generation from each classification is determined by multiplying 310 gpd/ESD with 6 ESDs per acre for Low Density and Commercial/Industrial, 15 ESDs per acre for Medium Density, and 11 ESDs per acre for Transit Oriented Development. Galt's General Plan has established a more diversified land use classification/zone. Based on the same SCIS method and Galt Code, Table 18.20-1, the diversified classifications/zones will be equated to the SCIS zones for design flow estimate. Overall, the design parameters will be established based on site-specific conditions accompanied by the guidelines set forth in the Improvement Standards.

The SCIS also specifies that the sewer system design flow will be the Peak Wet Weather Flow (PWWF), which is equal to the Peak Dry Weather Flow (PDWF) plus Infiltration and Inflow (I/I). The PDWF is the product of the ADWF multiplied by a Peaking Factor (PF). To determine flow generation, the City has provided zoning maps and base maps for counting residences and determining acreages. With the information, for residential housing developments, the Average Dry Weather Flow (ADWF) generation has been determined by counting the actual number of residential units for existing developments.

For schools in the system, the ADWF is based on the student population of the school including administrative, teaching, and operating personnel. Boyle has contacted the schools for this information.

I/I is dependent upon area rainfall. In general, the City has recommended using an I/I rate of 1,600 gpd/acre in earlier developed areas and 1,200 gpd/acre in new areas that have been constructed in the last 15 years (also SCIS recommendation).

The ADWF is a total quantity of flow will be discharged from an ESD in one day averaged on a yearly basis. To determine the days that might have higher flows, the SCIS has included an equation, $PF = 3.5 - 1.8Q_a^{0.05}$, where Q_a = ADWF mgd, for determining a peaking factor (PF) to multiply the ADWF and estimate the higher daily flows called the PDWF. When I/I is added to the PDWF, it becomes the highest daily sustainable flow, the PWWF, the system should be able to convey. This PWWF can also be considered as the diurnal peak-hourly sewage flow rate collected from a community. The PF is a minimum of 1.2 for service areas consisting primarily of commercial and/or industrial zones.

Friction factor "n" is a key element that will affect the rate of flow the sewer pipe can convey. It is one of the factors needs to be input into the computer model for capacity analysis. The SCIS standards recommends an n = 0.013 be used for all pipes regardless of its material.

The SCIS standards have also recommended that the maximum depth of flow at design condition (which is PWWF) in any collector of 12" diameter or less to be 0.7 of the pipe diameter (d/D ratio). Lines that are larger than 12" in diameter can be considered for flowing full when no direct service sewer connections from households are identified. The computer model will compare the calculated d/D ratio with the input default d/D ratio to determine if the existing sewer line has the capacity to carry the estimated flow. Any sewer line having a calculated flow d/D ratio higher than the default will be "red flagged" as deficient by the computer model and indicated on the display map.

Two feet per second (2 fps) minimum flow velocity is also recommended by SCIS. When the minimum velocity of 2 feet per second cannot be achieved, SCIS recommends that the minimum slopes be noted as follows:

Diameter	**Minimum Slope**	**Design Capacity**	**ESD's Served**
Inches	*Fixed Alignment*	*MGD*	*Max units*
8	0.0035	0.38	300
10	0.0025	0.58	500
12	0.0020	0.85	800
15	0.0015	1.60	2100

CITY DEVELOPMENT STANDARDS

The City's Galt Code, Table 18.20-1, "Development Standards", presents a list of standards for the various zones indicated in the City's General Plan. It supplements the SCIS standards, which provide only two residential housing densities, one commercial/industrial and one transit oriented development classifications (zones). Specifically for residential developments, Table 18 specifies the minimum lot size/minimum net lot area per unit for R1A, low density residential at 10,000 sf per lot to R2 residential at 5,000 sf per lot. For this capacity analysis, loading factors will be generated for all classifications diversified from the SCIS classifications.

STANDARDS APPLICATION

In general, both the SCIS and the Galt Code's Development Standards are to be used as guidelines for developers to design new sewer lines or systems for new developments. Galt is 90 percent developed. Most of the developed areas are now occupied with residential housing units. The City has consolidated almost all the residential development lot maps onto a background map. As a result of this effort, the exact number of housing units can be counted from the development base map. By multiplying the counted units with a flow generation factor, the amount of sewage collected in the associated sewer system can be quantified. There is no individual flow measurement device for recording the flow collected from each development. However, because all the sewage flow from the City is directed to the City's only wastewater treatment plant for processing, the total flow generated in the City, on an annual, daily and hourly basis has been recorded, and there are many years of records to be used as a reference.

The computer model will calculate the flows generated from each tributary area. When adding the tributary flows together, it will produce a total flow that eventually discharges into the wastewater treatment plant. The availability of the flow records at the treatment plant will provide a crosscheck and verification of whether the model computed flows/results are within acceptable margins. Subsequently, it provides confirmation that the flow generation factors/multipliers are reasonable or indicates that these factors require adjustment.

For the computer model, the adaptation of standards is summarized as follows:

Criteria	**Applied Standards in Computer Model**
ESD per acre	SCIS and Galt Code recommendations used as base, adjusted to suit City zoning diversity.
Flow Generation, 310 gpd/ESD or 232 gpd/ESD*	SCIS recommendations used as a base, adjusted for existing development according to WWTP influent records.
Friction Factor	SCIS recommendation, 0.013 for all pipe material
Peaking Factor	SCIS recommended equation
Water Depth to Diameter Ratio (d/D ratio)	SCIS recommendations, 0.7
Minimum Slope	SCIS recommendations, 0.0035 for 6-inch sewer and others as listed in SCIS. These criteria will apply only to sewers in model without invert and slope information.
Inflow and Infiltration (I&I)	City recommendations.

* Under paragraph 7-2, A, SCIS standards also specifies that for "Single family planned unit developments" the flow per unit shall be 232 gallons per residential unit/day (0.75 ESD) and the actual number of units per acre shall be considered. This guideline can be implemented when the actual number of units is available, either planned or constructed. Since the number of developed units is know in each of the City's existing housing areas, the 232 gpd/ESD will be considered in the following flow generation estimates.

Chapter 4
INITIAL FLOW GENERATION

INTRODUCTION

The primary purpose of undertaking this initial flow generation estimate exercise is for developing flow generation criteria (factor/multipliers) for input into the Model to perform the capacity analysis. The Model selected for this capacity analysis project primarily calculates the tributary area flow by multiplying an area (model calculated) with a flow generation factor (manual input) in the Model. Therefore, it is essential that the flow generation factor, gpd/ESD represent closely the current flow being collected from tributary areas. This is especially true when the SCIS standards also use low density residential ESD/acre flows as the base for non-residential areas

The City is presently about 60 percent occupied by developed residential area. It is appropriate to assume that most of the current sewage flow generated by the City is from the residents when no major industries are present in the City. Under such conditions, when the selected flow generation multiplier (gpd/ESD) times an actually counted number of residential units produces a total flow that is close to the recorded flow of the WWTP, it is fair to say the selected flow generation multiplier is representative. The flows to each lift stations will be calculated as described in the above paragraphs. The lift station tributary approach has been selected to estimate flow generation because the collected sewage flow primarily goes through one or two steps of pumping before being directed to the treatment plant. This approach will also break the entire system into smaller grids that simplify the analysis.

TRIBUTARY AREAS

The construction of the pump stations has basically divided the existing wastewater collection system of the City into eleven individual grids. The City's WCS Map shows each pump station receives wastewater flow from a network of gravity sewers. By "back tracing" the network of sewer pipes from the pump stations to the very first manhole upstream, the tributary boundaries of each pump station can easily be drawn. Planned Development (PD) areas that do not have sewer mains could be assigned to the pump stations' tributary areas based on the following considerations:

- Capacity available in nearby existing sewer lines for new flow from PD areas.
- Presence of stub-out from existing manholes planned for future connections.
- Space available for constructing new primary sewer collectors for draining new flows from PD areas to the existing pump station.

Based on the above approach and considerations, the tributary area of each pump station has been outlined in **Figure 3**. Because the pump stations have in effect created individual tributary areas, any deficiency of a tributary area will remain within that tributary area and does not exert any impact on the performance of the other tributary areas.

For this project, in general, the overall flow generations from each lift stations tributary area is estimated based on the SCIS standards adjusted for the more diversified City zoning system. The estimated flow is used to compare with the existing capacities of the pump stations. This is for verification whether the

existing pumps have the capacity to handle the estimated current or projected future flow when the City is completely developed. The insufficient pump stations will most likely be included as a future capital improvement project for the City.

To simplify identification, the tributary areas have been named according to the pump station that collects that area's wastewater. Using the AutoCAD zoning maps provided by the City, the acreages for each zone can be estimated. With the use of the City's development base map, the actual number of lots within each zone has been counted where land has been developed. The City also provided the actual number of units in several existing or under planning/construction apartment complexes in the City for use in the flow generation estimate of this chapter.

FLOW GENERATION FROM EACH TRIBUTARY AREA

Planned Development Areas

The existing sewer systems for the various developments in the City were, for the most part, designed and constructed based on the SCIS standards or the City's Development Standards in the Galt Code of Regulations (City Standards). Therefore, it is reasonable to apply these Standards to estimate the sewage flow generated from PD areas.

Table 1 summarizes the selected SCIS standards and the flow generation criteria adopted for the various PD areas. Multiplying the estimated acreage of an area with the data listed in the "Flow Determination Criteria" column for residential areas will determine the number of ESD's that may be built in the area. The flow generated in the tributary area is then calculated by multiplying the total number of ESD's (units) by associated unit multipliers. Using 310 gpd per unit to design sewer systems for new developments will most likely provide contingent capacities for unexpected conditions. As for commercial and industrial PD areas, the total known acreage will be multiplied by 1,860 gpd/acre to estimate the generated sewage flows.

TABLE 1 – FLOW GENERATION MULTIPLIERS FOR PLANNED DEVELOPMENT

Development Zones as shown on Galt General Plan	***Flow Determination Criteria***	***Flow per unit multiplier***
OS - Open Space	NA	NA
PQ - Public/Quasi Public	Schools(1) or/+ 6 ESD/acre	310 gpd/ESD
RA - Residential Agriculture	Assumed as R1A density at 4.3 ESD/acre(2)	310 gpd/ESD
R1A - Low-Density Single-Family	4.3 ESD/acre(2)	310 gpd/ESD
R1B - Intermediate-Density Single-Family	5.3 ESD/acre (2)	310 gpd/ESD

Development Zones as shown on Galt General Plan	*Flow Determination Criteria*	*Flow per unit multiplier*
R1C - Maximum-Density Single-Family	6.7 ESD/acre(2)	310 gpd/ESD
R2 - Medium Density	15 ESD/acre(2)	310 gpd/ESD
R3 - Multiple Family	18 ESD/acre(2)	310 gpd/ESD
C - Commercial	SCIS Industrial / Commercial Design Criteria	1,860 gpd/acre(3)
HC - Highway Commercial	SCIS Industrial / Commercial Design Criteria	1,860 gpd/acre(3)
NC - Neighborhood Commercial	SCIS Industrial / Commercial Design Criteria	1,860 gpd/acre(3)
OP - Office Professional	SCIS Industrial / Commercial	1,860 gpd/acre(3)
LM - Light Manufacturing	SCIS Industrial / Commercial	1,860 gpd/acre(3)
M - Industrial	SCIS Industrial / Commercial	1,860 gpd/acre(3)

(1) School flows have been generated based on the school population size
(2) Based on Table 18.20-1, Development Standards, Galt Code
(3) SCIS recommends 6 ESD/acre at 310 gpd/ESD

Developed Residential Areas

The City's General Plan Zoning Map identifies in detail the number of lots that have been subdivided for the various zoning areas. Therefore, the number of lots in each developed tributary area can be counted. When multiplying the counted units with a flow per unit multiplier, it estimates the flow generation from the tributary area. For flow generation estimates, all single-family developments (R1A, R1B and R1C) were considered as having one ESD per lot. As for multiple family developments (R2 and R3 zones) such as apartment complexes, the flow generation estimates were based on the actual number of units (ESD) that have been or are being constructed in the area. Each ESD within a R2 or R3 zone was multiplied by a flow rate of 232 gpd/ESD (recommended for area with the number of units known) to calculate the actual flow draining to the sewer mains and eventually collected into the appropriate lift stations.

Developed Nonresidential Areas

The design criteria for nonresidential areas as recommended by the SCIS for commercial, industrial and other nonresidential developed areas were used to estimate the flow from developed nonresidential areas. This approach was used because the equivalent number of units cannot be accurately determined for industrial/commercial areas within the City. Adopting this conservative approach as described in the SCIS standards may provide capacity in the associated sewer lines to handle unexpected changes of conditions.

School Flows

School flows were based on the SCIS criteria as listed in **Table 2** in concert with the actual school population data obtained by Boyle. **Table 3** displays each school in the City of Galt and its associated population. Since every school in the City met the capita limit, each school was given an estimated flow based on the type of school as defined by the SCIS

TABLE 2 – SACRAMENTO COUNTY SCHOOL DESIGN FLOW CRITERIA

Type of School	Avg. Daily Flow	Capita Limit
Elementary	0.025 MG	1,000
Middle	0.060 MG	1,500
High	0.080 MG	2,000

TABLE 3 – SCHOOL POPULATIONS AND SELECTED AVERAGE DAILY FLOW

School	Population	Capita Limit	Average Daily Flow (mgd)
Elementary & Preschool			
New Hope	136	1,000	5,000(3)
Fairsite	780	1,000	25,000
Marengo	690	1,000	25,000
River Oaks	713(1)	1,000	25,000
Middle			
Greer	780	1,500	60,000
McCaffrey	860	1,500	60,000
Valley Oaks	1,000(1)	1,500	60,000
High			
Galt	1,990(2)	2,000	80,000
Estrellita/Galt Adult Edu.	100(2)	2,000	4,000(3)

(1) Includes students, administration, teaching, and operating personnel
(2) Includes students and teaching staff only
(3) Proportioned based on population size

Infiltration and Inflow

Infiltration and inflow (I/I) was considered to be 1,600 gpd/acre in existing areas and 1,200 gpd/acre in newly developed areas (constructed in the last 15 years). These design flow criteria are recommended in the SCIS standards. The City has indicated which areas have been constructed in the last 15 years.

Flow Calculations

Based on the flow generation criteria summarized above, the Peak Wet Weather Flow (PWWF) was calculated with the MSWord Excel spreadsheet program for each lift station tributary area. The calculated flows are shown in details in **Table 4** for the existing conditions, and **Table 5** for future build-out conditions as indicated on the current General Plan. The calculated total inflow to each lift station has been summarized in **Figure 4** and **Figure 5**. These lift station flows will be used as comparison references in the model calibration process.

LIFT STATION EVALUATION

First tier lift stations –

The overall results of the flow generation calculation indicates all the first tier lift stations (see Chapter 2 of this report) have excess or minor deficient capacity to handle the estimated PWWF for the existing and build-out conditions. There are four lift stations, including: **E Street**, **Chabolla**, **Crystal**, **Sparrow** and **Elm & Amador**, that have not shown any difference in collected daily flow for both existing and future conditions because their tributary areas have been considered totally developed. The flow generation of **Kost** lift station will be doubled for the build-out condition because of PD areas in its tributary area. No expansion will be needed, however, for the **Kost** lift station because its current pumping capacity is five times the estimated flow. The build-out condition flow would only reduce **Kost**'s excess capacity from 500 percent to 250 percent. Due to PD areas, the **First Street** lift station's minor deficient capacity has increased from 8 percent to about 14 percent. The capacity of the **First Street** lift station may need to be increased to accommodate the additional flow.

Second tier lift stations – There are three second-tier lift stations:

A Street Lift Station pumps could pump up to 2.59 mgd when McFarland Lift Station pumps are not pumping. For the PWWF at existing condition, it is slightly under sized (deficient capacity), i.e. the duty pump capacity is less than the estimated tributary flow. When all the PD area in its tributary area is developed, the deficient capacity will increase to 32 percent. New higher capacity pumps will be needed, or switch on the standby pump to help pumping the occasional PWWF.

McFarland Lift Station's tributary areas are mostly developed; there is no difference in the estimated flows from the tributary areas for both existing and future conditions. This is also true for the first tier-lift stations that discharge their flow to McFarland, whose estimated flow remains the same for both existing and build-out conditions. A new station that will have a capacity of 1.27 mgd is replacing the McFarland pump station. When comparing the new capacity of the new pump station with the estimated flows, even doubling the new capacity to 2.4 mgd, the new pump station would not be sufficient for the estimated 3.83 mgd PWWF. However, no pump station problem has been experienced to date. This is due to the fact that the A Street lift station is relieving the McFarland lift station. At an over flow structure near the A Street station, excess flow collected in McFarland's tributary area has been allowed to overflow to the A Street lift station for direct pump to the Live Oak lift station. When both lift stations experience the unusual high flows, the standby pumps can be switched on to increase the total pumping capacity to pump the 6.46 mgd estimated. However, the primary duty of the standby pump should still be supporting the duty pump incase the duty pump fails, even though they provide the additional capacity needed to handle an emergency situation when both pumps are in good condition. Therefore, the City should ensure that both pumps are in good condition during the wet seasons of the year.

Vintage Oak lift station has a rated capacity of 2.16 mgd, even though it can pump up to 4.32 mgd when both pumps are pumping. Meanwhile, when compare the rated capacity of the lift station with the estimated flows; it is almost 26 percent under capacity for the existing PWWF conditions. At build-out condition, the deficiency will increase to almost 100 percent. Boyle is not aware of whether the City has recorded an instance when the pump failed to lift all incoming flow out of the station. The 26 percent

deficiency may have been the result of the conservative estimates by the computer model for non-residential areas. The estimated flows for the residential areas are considered reliable because it was done by actual counting of the constructed units and the same flow generation multipliers were used in other lift station tributary areas. Temporarily storing the high flow in the collection system will lower the peak to the pump station and provide more time for the pumps to lift the flow out of the station. A new lift station will eventually be needed for the **Vintage Oaks** tributary area to share the burden of additional flow from the PD areas.

Third tier lift station - The Live Oak lift station is the only third tier lift station that receives all sewage flow collected within the City boundary and pumps it to the City's only WWTP. Approximately 94 percent of the total flow pumped by the Live Oak pump station is from the second tier lift stations. The remaining portion of the flow is from the adjacent gravity sewers. When comparing the estimated flow with the current capacity of the lift station, it is obvious that the Live Oak lift station cannot send the collected 9.78 mgd PWWF to the WWTP with one pump. Even with both pump running simultaneously, the pumping capacity would not be sufficient. The estimated flow may be conservative, but not too far off because the City has recorded instances when both pumps were switched on automatically due to higher than usual flow to the station. The simultaneous operation of the Live Oak lift station pumps matches the designed peak influent capacity of the WWTP, which is 9.0 mgd. Therefore, the Live Oak lift station needs to be expanded to process the flow or a new station be constructed to relief its operation.

Summary - The overall lift station sufficiency evaluation presented in this chapter is for the PWWF and based on the assumption that all lift station pumps are operating at one time. Such assumed conditions, however, have not been experienced frequently at the City's lift stations. Besides, it is most likely that all the standby pumps have been programmed to run when there is an unusually high flow condition. Therefore, no alarming operation deficiency has been experienced at most of the lift stations.

The fact that an estimated total PWWF (9.78 mgd) could be sent to the WWTP supports the conclusion that the flow generation criteria/multipliers used in the spreadsheet calculation are valid and can be converted and applied to the Model for flow generation estimates. The WWTP has been designed to receive a peak hydraulic flow of 9.00 mgd according to record drawings. The existing Life Oaks lift station pumps can only pump a total of at most 8.8 mgd. The estimated 9.78 mgd PWWF should be considered within the acceptable limits since the SCIS standards are set conservatively to provide contingent capacity in the sewer lines and the temporary storage in the sewer system will lower the peak flow rates.

Chapter 5
SEWER MODEL DEVELOPMENT

NETWORK DEVELOPMENT

In general, wastewater collection system analysis computer models are used for major collection sewer lines in a system. A network of primary and secondary sewer lines will need to be selected from the overall sewer system for the model analysis. As such, for the computer modeling analysis of this project, a network of primary and secondary sewer lines has been developed. **Figure 6** is the WWCS Map of the City showing a layout of the network of selected key primary trunk and secondary collection sewer mains, highlighted in red, that will be entered into the computer model for capacity analysis. The primary collection trunk sewers are those final, larger diameter sewer mains that drain the flow directly into a lift station. They are the influent pipes of the lift stations. Their pipe diameters vary from 8-inch for smaller lift stations to 24-inch at the Live Oak lift station. The trunk sewer lengths also vary. For example, the Vintage Oak lift station has two influent primary sewer mains, one extending more than 5,000 feet to the north and the other 3,000 feet to the south, while First Street lift station's influent trunk sewers are less than 400 feet. The secondary collection sewers are those that discharge the collected flow into a primary trunk sewer and collect flows from a section of the tributary area. In general the secondary collection sewers have pipe diameters primarily 8-inch or larger. They are usually deeper in the ground; therefore, it is assumed that no house service connections have been connected directly to the secondary collection mains. The selection of the key collection sewer mains to be included in the computer model analysis has been based on the following primary considerations:

- The gravity influent sewer main(s) of a lift station.
- Sewer mains that have the potential of receiving additional flows from nearby PD areas.
- Sewer segments with major problems that have been identified by the City.

From visual review of the City's WWCS map, most of the collection sewers inside the developed residential areas are 6-inch diameter or larger, with only a few exceptions. A quick capacity calculation shows that a 6-inch sewer installed at minimum slope of 0.0035 (adopted slope recommended for 8-inch sewers in SCIS) can theoretically convey a flow of 130 gpm at 1.9 f/s velocity and 70% full (d/D = 0.7) or a daily flow of 187,200 gpd. When this total flow is divided by 900 gpd PWWF/ESD, the flow is equivalent to 200 single-family units. Therefore, it is assumed that all 6-inch sewers within developed residential zones have the capacity to convey the incoming PWWF, when they serve less than 200 ESD and overflow problems have not been experienced. This was the reason why not every stretch of sewer pipe in the City will be included in the computer model network for capacity analysis. Areas with small sewer lines (< 8-inch diameter) that have been identified with problems may need to be analyzed separately. The modeling effort will include all sewer mains, as defined as being pipelines 8-inch and larger or that have been identified as key collectors.

MODELING APPROACH

Scenarios/Loading Conditions - The purpose of this Wastewater Collection System Capacity Analysis Project tasked to Boyle by the City is to determine the capacity of the City's existing wastewater collection (sewer) system. For the analysis, using InfoSewer-Pro by MWH Soft® (the Model), Boyle

developed two sewer models, one for the existing developments of the City and one for the build-out condition including all PD areas as presented on the current General Plan. The models analyze the selected key collection sewer network of the system for its current and future performance under various loading conditions. The SCIS provides guidelines as described in Chapter 3 of this document to develop three flow conditions, and they are:

- Average Dry Weather Flows (ADWF),
- Peak Wet Weather Flows (PWWF), and
- Peak Dry Weather Flows (PDWF)

The SCIS standards require all sewer mains should be designed for the PWWF, which is the highest the sewer mains are anticipated to experience at normal conditions. When a sewer main has sufficient capacity to carry the carry the PWWF flows, it will certainly be adequate for the ADWF and PDWF flows. As such it seems that modeling the ADWF and PDWF flows is unnecessary, but model runs will be undertaken for several flow scenarios. The primary reason behind running six scenarios is to provide sufficient data for setting priorities for improvement. For example, a deficient sewer main under ADWF flows will certainly be considered for priority 1 improvement. Therefore, six scenarios based on three loading conditions have been selected for the model to determine the flows and capacity of each sewer pipe, fullness of pipe (e/D ratio), lift station required capacity, and total flow delivered to the wastewater treatment plant. Within the three flow conditions, sewer pipes that have difficulty carrying the calculated sewer flows will be identified and discussed.

The City has identified five problem areas in the sewer system, and four have been incorporated into the computer model for analysis. The one problem area omitted from the model is too small for the model to analyze. Therefore, the omitted area will be analyzed individually to determine improvement options.

The Model - The InfoSewer-Pro by MWH Soft® software allows the import and overlay of sewer system information for automatic processing. To make use of such capabilities in the model development process, the City provided AutoCAD drawing of the existing sewer system were imported into the Model. The Model recognized the drawing as a sewer system; however, the manholes as indicated in the AutoCAD drawing were also recognized as pipes instead of manholes. Therefore, manholes were manually created in the model to connect each sewer pipe segment. These created manholes are only those that will receive flows from the tributary areas. Therefore, not all the manhole on the WWCS map is shown on the selected network. As a result, there are a total of 405 gravity mains, 14 force mains, 11 pump stations, and 403 manholes included in the selected network analyzed by the modeling software. The Model assigns ID numbers to each pipe and manhole automatically, and it requires input of invert elevations at each manhole, and the segment length, diameter, and material of each pipe, as well as wet well and pumps information (see **Appendices 1 to 5**). The required data are primarily the unknowns of the Manning's Equation, based upon which the Model was written. With the input information, the Model calculates the steady state (normal) flow that each pipe segment can carry at an assigned slope. When a pipe becomes full, the model switches to the Hazen and Williams formula widely used for full pipes' headloss calculation. The backwater effect generated from downstream overloaded or full pipe is analyzed dynamically as variable flow in determining deficient pipes in the system. The backwater effect will slow down the flow velocity, and cause solid deposits and grease accumulation that lead to blockage and overflow into the service streets. As such, when the backwater

effect, from flat slope and/or undersized sewer pipe, is eliminated from a downstream pipe, problems in upstream pipe segments will most likely be eliminated.

Required Information - Subdivision development drawings provided by the City contained most of the information required by the model. However, sewer drawings in and near parts of "Old Town Galt" could not be located. In such instances, invert elevations were estimated based on known elevations and slopes of nearby similar pipes. The Model then measured the pipe length reference to the AutoCAD drawings to determine the pipe slope. When relatively little was known near the sewer mains in question, the SCIS recommended minimum slopes were used to select the invert elevations needed for the model. No minimum slope is recommended for 6-inch sewers. Therefore, the 0.0035 minimum slope recommended for 8-inch sewers was used for 6-inch sewers. For sewer mains larger than 15-inch, a slope of 0.0015 was used even though progressively, their slope could be flatter according to SCIS. The model automatically selects a friction factor based on the pipe material entered into the model. For gravity mains, the pipe material was assumed to be VCP in the model even though asbestos cement pipes (ACP) were used for older sewer mains. The model assigns a friction factor of 0.013 for VCP. This matches the friction factor recommendation in the SCIS. For force mains, the pipe material was assumed to be ductile iron (DI) for which the Model assigns a friction factor of 130 automatically. Ductile iron was selected because it usually has a cement liner that could be assumed similar to the interior surface of the existing ACP force mains. Asbestos cement pipe has not been included in Model's selection list perhaps it is no longer a material option for any new sewer installations or any other piping systems because of its hazardous rating. Appendix 1 presents all the input data of each segment of sewer mains included in the computer model runs.

Plates 1A, 1B and 1C are the Model version of **Figure 6** showing the network of the image from the Model containing all of the pipes, manholes, pump stations, and outlets in the selected network input into the model. The City's base map that shows all the present developments of the City was inserted in this figure as the background for referencing the locations of the selected primary and secondary collection sewers.

Figure 7 presents a layout with the City's zoning map overlaid into InfoSewer. The Model has the capability of estimating the sewage flow from a tributary area automatically. Based on a weighted averaging method called the Thiessen Polygon Method (used primarily for mean precipitation computation) the model can carve out/create tributary areas around each manhole. Once the tributary areas are created, the Model calculates the acreage of the carved out area, and based on the zoning map and loading factor (mgd/acre) input, it calculates the flow to the manhole of the carved out area. An initial run was conducted by allowing the Model to automatically create tributary areas around each manhole to calculate the tributary flows (see **Figure 9**). The calculated flow results were acceptable; however, the tributary areas carved do not represent the actual tributary area of each manhole. Therefore, the results of this exercise were used as a reference for crosschecking the results of the ultimate run based on manually carved tributary areas (see **Figure 8**) as presented in later sections of this report.

Flow generation multipliers/Loading factors - The total flows generated from each pump station's tributary area were manually estimated and presented in Chapter 4, under the section "Flow to Each Tributary Area." The estimated pump station total flows were used as a reference in adjusting the loading factors for existing developments after each trial run. **Table 6** presents the calculated load

factors developed for each land use for the Model as in the final model run. Due to the Model requiring an input of gpd/acre, the ESD/acre x gpd/ESD flow estimation formula was simplified to gpd/acre based on information as listed in **Tables 4 and 5**. The use of Table 6's loading factors for existing developments have produced an existing ADWF total flow (see **Table 7**) from model runs similar to the current flow being received at the WWTP. However, the model estimated future build-out PWWF flow of 11.86 is far from the 18 mgd as projected for the WWTP when built to full treatment capacity.

In the process of adjusting the loading factors to produce a flow similar to what is actually being received at the WWTP, the loading factors adopted for the existing non-residential areas have been adjusted from the SCIS recommendation for commercial/industrial areas of 1,860 gpd/acre. Instead, these zones have been based more reasonably on low-density R1A residential flows, which were established (in Chapter 4) to be 330 gpd/acre. The SCIS equates commercial/industrial with low-density residential by estimating their flow generation to be from 6 ESD/acre at 310 gpd/ESD. This calculation formula is used if the flow is not known or unable to be accurately estimated. Using the 330 gpd/acre for the non-residential areas follows the same assumption of the SCIS standards.

The low-density residential flows from existing developments have been estimated by counting the number of units in the various zones and multiplying them by 232 gpd/unit (see Chapter 4 results). Since the estimate of 330 gpd/acre is based on actual existing conditions, it was also used for non-residential zones by the same reason in the previous paragraph. All other developed areas were given a load factor based on their actual developed densities.

The infiltration and inflow (I/I) factor added to the PDWF is 1,200 gpd/acre for the entire City. In a meeting with the City, it was discussed to use 1,200 gpd/acre and 1,600 gpd/acre for newly developed and established areas respectively. However, it was selected to use 1,200 gpd/acre for all areas in the model scenarios because wet weather and dry weather influent flows recorded at the WWTP are very similar. Boyle's approach using the 1,200 gpd/acre for I/I may still be very conservative for the actual I/I that the City's sewer system experiences. For all new developments, 1,200 gpd/acre is suggested by the SCIS.

Table 4 displays a column with peaking factors calculated for each zone based on SCIS' peaking factor formula. The average of the column's peaking factors is approximately 2.0. For the PDWF and PWWF, this average peaking factor of 2.0 will be used in the model analysis.

Table 6 – Calculated Flow Generation Load Factors for Computer Model

Based on Actual Number of Units X 232 gpd/acre				
Land Use Zone	**Range (mgd/acre)**	**Load Factor ADWF (mgd/acre)**	**Load Factor PWWF (mgd/acre)**	**Load Factor PDWF (mgd/acre)**
R1A	0.00010-0.00055	0.00033	0.00306	0.00066
R1B	0.00039-0.00087	0.00063	0.00366	0.00126
R1C	0.00080-0.00011	0.00095	0.00430	0.00190
R2	0.00070-0.00250	0.00160	0.00560	0.00320
R3	0.00100-0.00450	0.00275	0.00790	0.00550
Based on SCIS Low and Medium Density (Acres X #ESD/acre X 310 gpd/ESD)				
Land Use Zone	**SCIS Recommendation**	**Load Factor ADWF (mgd/acre)**	**Load Factor PWWF (mgd/acre)**	**Load Factor PDWF (mgd/acre)**
R1A-PD	6 ESDs per acre	0.00186	0.00612	0.00372
R1B-PD	6 ESDs per acre	0.00186	0.00612	0.00372
R1C-PD	6 ESDs per acre	0.00186	0.00612	0.00372
R2-PD	15 ESDs per acre	0.00465	0.01170	0.00930
R3-PD	15 ESDs per acre	0.00465	0.01170	0.00930
Based on Acreage Similar Loading as Low Density Residential (SCIS)				
Land Use Zone	**Estimated Average Flow for Low Density Residential (mgd/acre)**	**Load Factor ADWF (mgd/acre)**	**Load Factor PWWF (mgd/acre)**	**Load Factor PDWF (mgd/acre)**
PQ C HC NC OP LM M	0.00033	0.00033	0.00306	0.00065
Based on SCIS Low, Medium, and Commercial Density (Acres X #ESD/acre X 310 gpd/ESD)				
Land Use Zone	**SCIS 6 ESDs per acre X 310 gpd/acre**	**Load Factor ADWF (mgd/acre)**	**Load Factor PWWF (mgd/acre)**	**Load Factor PDWF (mgd/acre)**
PQ-PD C-PD HC-PD NC-PD OP-PD LM-PD M-PD	0.00186	0.00186	0.00612	0.00372

MODELING SCENARIOS AND RESULTS

Model set up - There was one model created for the capacity analysis. The model's tributary areas were carved by hand based on the contributing sewer grid. It generates and presents the true tributary areas for each manhole. As a result, the computer estimated flow represents the actual flow being collected in the connection system. There were a total of six scenarios created for the model to analyze the sewer system's capacity. For simplicity, the six scenarios of the models are labeled as follows: Model #1 - A1, A2, B1, B2, C1, and C2;

- **Scenarios A1** is ADWF for the existing developments of the City. This flow is the average daily flow without including infiltration and inflow (I/I) or multiplying by a peaking factor. It is simply the total flow collected from the tributary areas.
- **Scenarios A2** is ADWF for build-out conditions. The average daily flow is calculated similar to scenarios A1; however, it includes the planned build-out of all developments within the City's limits.
- **Scenarios B1 is** PWWF for existing developments. This flow is the total of ADWF multiplied by an average 2.0 peaking factor, plus an I/I multiplier of 1,200 gpd/acre.
- **Scenarios B2 is** PWWF for build-out condition. This scenario is similar to B1. However, they include planned build-out developments within the City's limits.
- **Scenarios C1** is PDWF for existing conditions. Peak Dry Weather Flow, which is considered as the daily peak hourly flow, is the product of ADWF multiplied by a 2.0 peaking factor without including I/I.
- **Scenarios C2 is** PDWF for future conditions, which include all planned build-out within the City's current limits.

Modeling results – Table 7 shows the scenarios and their corresponding total flow to the treatment plant as computed by the models using loading factors presented in **Table 6**.

Table 7 – Total Flow to Galt WWTP for each Scenario

Scenario Label	Scenario	Model Run Total Flow to WWTP (mgd)
A1	ADWF – Existing	2.3
A2	ADWF – Future	3.5
B1	PWWF – Existing	9.1
B2	PWWF – Future	11.9
C1	PDWF – Existing	4.2
C2	PDWF – Future	6.1

All the wastewater collected in the City eventually reaches the City's WWTP. The amount of flow received at the treatment plant each day is measured and recorded. According to the WWTP's

construction record drawings, the WWTP facilities were designed for an average flow of 3.0 mgd in the most recent upgrade and expansion. At build-out, the ultimate average capacity of the WWTP will be 6.0 mgd. The plant influent peaking factor used for calculating the hydraulic peak flow to the WWTP is 3. In such case, the plant should be hydraulically sufficient to handle a sustainable peak flow of 9.0 mgd. Currently, the average daily flow pumped to the WWTP during the dry months is about 2.0 mgd, while that of the months during the rainy season is about 2.7 mgd. The fact that the plant could receive an influent of 9 mgd is supported by the ability of the existing Live Oak Lift Station to pump a total flow of about 8.0 mgd when both pumps are running.

The estimated ADWF-Existing flows to the WWTP listed in **Table 7** are similar to the current plant influent that has been delivered at the WWTP, which varies from about 2.0 mgd to 2.7 mgd. The model analysis results show that the flow generation conditions applied to the model are appropriate.

- Loading factors / flow generation multipliers
- Tributary area boundaries
- Sewer pipe parameters

One of the results of the model capacity analysis is identifying capacity deficiencies of the network. The primary factor used in identifying the capacity deficiency of a sewer main is the flow depth in the pipe. By comparing its depth/diameter ratio (d/D) computed by the model with a maximum d/D preset by the engineer, the deficiency is identified. Sacramento County Improvement Standards (SCIS) specify the maximum depth of flow at design conditions in any collector 12" diameter or less shall be 0.70 of the pipe diameter. However, lines larger than 12" diameter may be designed to flow full unless direct service sewer connections (from households) are planned; in which case the 0.70 diameter maximum depth shall govern. When the model recognizes a computed d/D in a sewer segment is higher than 0.7, even though just a fraction higher, the model will highlight the sewer segment in red ("red flag") the segment indicating a deficiency/possible problem with capacity. When a sewer segment is flowing full, the model will indicate the d/D as 1. The value of 0.70 from the SCIS may be considered conservative for the depth of flow. It is possible to reduce the number of segments that are "red flagged" by increasing the d/D used. The following section will discuss the results of each model run. The conservative approach in the d/D ratio setting provides contingency capacity for the sewer operation. It provides capacity to deal with unforeseen high flow conditions. However, allowing adjustment of the d/D ratio and accepting a high ratio for certain sewer segments may avoid the difficulty of replacing or installing a relief sewer in a street that is congested with other utilities. This is especially true for developed areas.

Model runs evaluation

Computer model runs were undertaken for the six scenarios established for the model. The following paragraphs present a general evaluation of the run results with focus on the "Red Flagged" sewer segments.

Model Run -

The model run was based on actual tributary drainage areas indicated on the City's existing wastewater collection system map. The actual tributary areas were hand sketched into AutoCAD and uploaded into the Model. These actual tributary areas are displayed in **Figure 8**. Subsequently, the scenarios - ***A1, A2, B1, B2, C1 and C2*** were run in the Model with the hand-carved tributary areas. Each area drains to its associated main collector manhole. The results of this second scenario set will represent the actual collection of the wastewater flows in the sewer network.

The Model runs generated a future ADWF much smaller than the projected 6 mgd build-out capacity of the treatment plant based on the existing City boundary. The projected 6 mgd treatment capacity may have included the City boundary future expansions or an overall flow estimate based on SCIS standards without having taken developed area actual flow into condition. The additional wastewater flow would most likely be collected in a new collection system and delivered to the WWTP by another trunk sewer main. The City has in the past experienced a PWWF condition where both of the pumps at the Live Oaks lift station were switched on simultaneously to pump the inflow to the WWTP. Therefore, the existing developments of the City could already generate a PWWF flow as high as 8.8 mgd.

Scenario A1*: Average Dry Weather Flow (ADWF) – Existing*
Results – The total flow to the WWTP for this scenario is **2.3** mgd. This is quite close to the current plant influent flow being received at the WWTP. It further confirms the flow generation conditions adopted for the model are acceptable. **Figure A1** shows a map of the network with the computer analysis results (see Appendix 6 for tabulated list) for ADWF existing conditions. All sewer segments can carry flow with a d/D less than 0.7 and no sewer main is red-flagged for deficiency.

Evaluation – The re-carving of the manhole tributary area has shown the sufficiency of the network. As such it appears that there is no problem in the City's sewer network. However, contrary to the Model results, the City has experienced problems in various areas and the model has not identified those known problem segments. This shortcoming of the Model is due to the fact that the Model only computes theoretical hydraulic conditions and does not identify problem conditions created due to low flow velocities that lead to settlement built up.

***Scenario A2**: Average Dry Weather Flow (ADWF) – Future*

Results – The total flow to the WWTP for this scenario is **3.59** mgd. **Figure A2** is a network map shows the computer analysis results (see Appendix 7 for tabulated list) for ADWF at future conditions. There are 11 sewer segments appearing in the map "red flagged" with a d/D greater than 0.7. After future build-out, there will be an additional 6 more sewer segments that are deficient as compared with existing conditions. The following **Table 8** lists all of the segments and their locations identified by the sewer model having deficient capacity and causing bottlenecks in the system:

Table 8 – ADWF Future Deficient Segments

Pipe ID	Location	Diameter	Slope	d/D
P-2201	A Street	8	0.0025	***0.74***
P-2205	A Street	8	0.0025	***0.76***
P-2209	A Street	8	0.0025	***0.78***
P-2213	A Street	8	0.0025	1.00
P-2263	E Street	6	0.0036	1.00
P-2327	E Street	6	0.0035	1.00
P-2331	E Street	6	0.0035	***0.81***
P-3179	McFarland Ave	8	0.0025	1.00
P-3181	McFarland Ave	8	0.0025	1.00
P-3183	McFarland Ave	8	0.0025	1.00
P-3185	McFarland Ave	8	0.0025	1.00

Evaluation – Again, the additional flow from PD area included in the future conditions have caused the model to "red flag" 11 segment of sewers as being deficient, and the 8-inch sewer in McFarland and A Street are now flowing full. Four of the segments, however, may not need improvement because they are still at a d/D below or little over 0.8. These segments will continue to be "red flagged" for the remaining scenario analysis when the flows will be higher.

***Scenario B1**: Peak Wet Weather Flow (PWWF) – Existing*

Results – The total flow to the WWTP for this scenario is **9.1 mgd**. **Figure B1** shows the computer analysis results (see **Appendix 10** for tabulated list) for the PWWF at existing conditions. Pipes highlighted in red are those with d/D being greater than 0.7. The computer run identified 85 segments that are insufficient to carry the existing PWWF at 0.70 d/D or less, and 48 of the deficient segments are actually flowing full. The following **Table 9** lists all of the segments and their locations identified by the sewer model having deficient capacity and may have caused bottlenecks in the system:

Table 9 – PWWF Existing Deficient Segments

Pipe ID	Location	Diam	Slope	d/D		Pipe ID	Location	Diam	Slope	d/D
P-1837	Oberlin Way	10	0.0037	1.00		P-2331	E Street	6	0.0035	1.00
P-1853	Trudy Way	10	0.0020	1.00		P-2335	E Street	6	0.0035	1.00
P-1991	Carillion Blvd	12	0.0031	***0.76***		P-2339	E Street	6	0.0035	1.00
P-2035	Carillion Blvd	21	0.0009	***0.74***		**P-2361**	Lincoln Ave	8	0.0035	***0.82***
P-2039	Carillion Blvd	21	0.0009	***0.75***		P-2367	Chabolla Ave	6	0.0025	1.00
P-2043	Carillion Blvd	21	0.0009	***0.75***		P-2473	Elm Amador	10	0.0011	1.00
P-2047	Carillion Blvd	21	0.0009	***0.75***		**P-2491**	Elm Amador	10	0.0011	***0.81***
P-2051	Carillion Blvd	10	0.0025	***0.81***		**P-2495**	Elm Amador	10	0.0011	***0.81***
P-2071	Carillion Blvd	10	0.0025	***0.77***		**P-2497**	Elm Amador	10	0.0011	***0.81***
P-2133	C Street	8	0.0025	1.00		P-2501	Elm Amador	8	0.0011	1.00
P-2137	C Street	8	0.0025	1.00		**P-2505**	N Lincoln Way	10	0.0011	***0.74***
P-2141	C Street	8	0.0025	1.00		**P-2671**	Live Oak Ave	24	0.0010	***0.79***
P-2145	C Street	8	0.0025	1.00		**P-2687**	Live Oak Ave	24	0.0010	***0.80***
P-2149	C Street	8	0.0025	1.00		P-2693	Live Oak Ave	24	0.0010	1.00
P-2153	C Street	8	0.0025	1.00		**P-2735**	Vintage Oak Ave	21	0.0015	***0.71***
P-2157	C Street	8	0.0025	1.00		**P-2753**	Vintage Oak Ave	21	0.0012	***0.79***
P-2161	C Street	8	0.0025	1.00		**P-2773**	Vintage Oak Ave	21	0.0013	***0.81***
P-2165	Lincoln Way	8	0.0025	1.00		P-2779	Vintage Oak Ave	21	0.0013	1.00
P-2169	Lincoln Way	8	0.0025	1.00		**P-2787**	Vintage Oak Ave	24	0.0008	***0.74***
P-2185	A Street	8	0.0025	1.00		P-2789	Vintage Oak Ave	24	0.0004	1.00
P-2189	A Street	8	0.0025	1.00		**P-3051**	Carillion Blvd	10	0.0025	***0.77***
P-2193	A Street	8	0.0025	1.00		**P-3053**	Carillion Blvd	10	0.0025	***0.80***
P-2197	A Street	8	0.0025	1.00		**P-3055**	Vintage Oak Ave	21	0.0011	***0.82***
P-2201	A Street	8	0.0025	1.00		**P-3057**	Vintage Oak Ave	21	0.0013	***0.81***

Table 9 – PWWF Existing Deficient Segments (Continues)

Pipe ID	Location	Diam	Slope	d/D		Pipe ID	Location	Diam	Slope	d/D
P-2205	A Street	8	0.0025	1.00		P-3099	Third Street	18	0.0015	1.00
P-2209	A Street	8	0.0025	1.00		P-3109	Downing Dr	6	0.0025	1.00
P-2213	A Street	8	0.0025	1.00		**P-3139**	Live Oak Ave	24	0.0010	***0.80***
P-2225	A Street	6	0.0025	***0.75***		**P-3141**	Live Oak Ave	24	0.0010	***0.80***
P-2239	Third Street	18	0.0015	1.00		P-3143	Live Oak Ave	24	0.0009	1.00
P-2243	Third Street	18	0.0015	1.00		P-3149	Elm Ave	8	0.0020	1.00
P-2247	Third Street	18	0.0015	1.00		P-3153	McFarland Ave	10	0.0025	1.00
P-2251	Third Street	18	0.0015	***0.71***		P-3155	McFarland Ave	12	0.0028	1.00
P-2255	Third Street	18	0.0015	***0.70***		P-3157	McFarland Ave	12	0.0040	1.00
P-2263	E Street	6	0.0036	1.00		P-3165	McFarland Ave	10	0.0024	1.00
P-2265	E Street	12	0.0020	1.00		P-3179	McFarland Ave	8	0.0025	1.00
P-2267	Third Street	18	0.0015	1.00		P-3181	McFarland Ave	8	0.0025	1.00
P-2271	Fourth Street	12	0.0020	1.00		P-3183	McFarland Ave	8	0.0025	1.00
P-2275	Fourth Street	12	0.0020	1.00		P-3185	McFarland Ave	8	0.0025	1.00
P-2277	Fourth Street	12	0.0020	***0.79***		P-3267	First St	6	0.0025	1.00
P-2285	H Street	12	0.0020	***0.80***		P-3269	First St	6	0.0032	1.00
P-2289	H Street	12	0.0020	***0.79***		**P-3273**	Fourth St	6	0.0046	***0.76***
P-2327	E Street	6	0.0035	1.00		P-3277	Oak Ave	6	0.0027	1.00
P-2331	E Street	6	0.0035	1.00						

Evaluation – The model has identified 85 deficient sewer segments and 52 segments will be flowing full, and of these full segments, 13 segments are larger than 12-inch in diameter that are allowed to flow full with permission from the City There are also 29 segments identified as deficient because the Model noted their d/D exceeded 0.7. Therefore, the number of segments that are truly deficient ends up being just 40. The true "red flagged" lines are mostly 10-inch and under and in the areas with reported problems in the southern part of the City, such as the lines in, Trudy, McFarland, A Street and E Street. The northern "red-flagged" sewer lines detected as being deficient are in Corillian Boulevard, Vintage Oak Avenue and Live Oaks Avenue. However, they are mostly just carrying the flow higher than e/D equals 0.7. Many that will be flowing full are smaller than 12-inch in diameter. Improvements will be required for many of these segments. Their priority should be set based on the need to reduce their maintenance. Overall, the deficient sewer lines have appeared at the anticipated locations. The 8-inch northbound sewer line in McFarland has shown up red in many scenarios, even for the ADWP flow. However, no problem has surfaced in the area. According to the City, again, most likely, it is due to the presence of a diversion box at the beginning of this red flagged line that allows excess flow to overflow into the A Street for pumping to the Live Oaks Lift Station directly. If the capacity of A Street Lift Station were to be reserved for its tributary area, the diversion would need to be stopped. The 8-inch deficient sewer line, therefore, should improve to increase its capacity along with upgrading the McFarland Lift Station to process the entire flow from its tributary areas.

***Scenario B2**: Peak Wet Weather Flow (PWWF) – Future*

Results – The total flow to the WWTP for this scenario is **12.13 mgd**, which may be the highest flow that the WWTP will receive from the City. It is only about 70% of the projected peak flow of 18 mgd for the WWTP. **Figure B2** shows the computer analysis results (see Appendix 11for tabulated list) for the PWWF future conditions. The model "red-flagged" 113 segments of sewer mains as deficient and highlighted in red on the drawing indicating the d/D are greater than 0.7. Of the 113 deficient segments, 95 of them are actually flowing full. Among the full segments, 41 segments are larger than 12-inch diameter. These larger pipes are allowed to flow full, but the remaining 55 segments were not designed to carry a full pipe of sewage. The larger segments that are allowed to flow full must not carry a high flow that will generate sufficient friction headloss to spill sewage out of any manhole. The following **Table 10** lists all of the deficient sewer segments and their locations.

Table 10 – PWWF Future Deficient Segments

Pipe ID	Location	Diam	Slope	d/D		Pipe ID	Location	Diam	Slope	d/D
P-1805	Quail Hollow Dr	12	0.0015	***0.71***		P-2339	E Street	6	0.0035	1.00
P-1807	Quail Hollow Dr	12	0.0020	***0.73***		P-2361	Lincoln Way	8	0.0035	1.00
P-1823	Quail Hollow Dr	12	0.0020	***.0.73***		P-2367	Chabolla Way	6	0.0025	1.00
P-1827	Lorna Way	12	0.0020	***0.77***		P-2473	Elm Amador	10	0.0011	1.00
P-1829	Lorna Way	12	0.0021	***0.75***		P-2491	Elm Amador	10	0.0011	1.00
P-1837	Oberlin Way	10	0.0037	1.00		P-2495	Elm Amador	10	0.0011	1.00
P-1839	Lorna Way	12	0.0020	***0.76***		P-2497	Elm Amador	10	0.0011	1.00
P-1853	Trudy Way	10	0.0020	1.00		P-2501	Elm Amador	8	0.0011	1.00
P-1973	Carillion Blvd	12	0.0020	1.00		**P-2505**	N. Lincoln Way	10	0.0011	***0.74***
P-1975	Carillion Blvd	12	0.0024	***0.78***		**P-2507**	North Lincoln Way	10	0.0021	***0.74***
P-1991	Carillion Blvd	12	0.0031	1.00		**P-2515**	North Lincoln Way	10	0.0023	***0.74***
P-2017	Carillion Blvd	18	0.0011	***0.79***		P-2671	Live Oak Ave	24	0.0010	1.00
P-2035	Carillion Blvd	21	0.0009	1.00		**P-2675**	Live Oak Ave	24	0.0018	***0.77***
P-2039	Carillion Blvd	21	0.0009	1.00		P-2687	Live Oak Ave	24	0.0010	1.00
P-2043	Carillion Blvd	21	0.0009	1.00		P-2693	Live Oak Ave	24	0.0010	1.00
P-2047	Carillion Blvd	21	0.0009	1.00		P-2735	Vintage Oak Ave	21	0.0015	1.00
P-2051	Carillion Blvd	10	0.0025	1.00		P-2753	Vintage Oak Ave	21	0.0012	1.00
P-2071	Carillion Blvd	10	0.0025	1.00		P-2773	Vintage Oak Ave	21	0.0013	1.00
P-2075	Carillion Blvd	10	0.0025	***0.77***		P-2779	Vintage Oak Ave	21	0.0013	1.00
P-2127	Frankston St	6	0.0021	1.00		P-2783	Vintage Oak Ave	21	0.0021	1.00
P-2133	C Street	8	0.0025	1.00		P-2787	Vintage Oak Ave	24	0.0008	1.00
P-2137	C Street	8	0.0025	1.00		P-2789	Vintage Oak Ave	24	0.0004	1.00
P-2141	C Street	8	0.0025	1.00		P-2793	Vintage Oak Ave	24	0.0010	1.00
P-2145	C Street	8	0.0025	1.00		P-2797	Vintage Oak Ave	24	0.0010	1.00

Table 10 – PWWF Future Deficient Segments (Continues)										
Pipe ID	**Location**	**Diam**	**Slope**	**d/D**		**Pipe ID**	**Location**	**Diam**	**Slope**	**d/D**
P-2149	C Street	8	0.0025	1.00		P-3013	E Street	12	0.0020	1.00
P-2153	C Street	8	0.0025	1.00		P-3051	Carillion Blvd	10	0.0025	1.00
P-2157	C Street	8	0.0025	1.00		P-3053	Carillion Blvd	10	0.0025	1.00
P-2161	C Street	8	0.0025	1.00		P-3055	Vintage Oak Ave	21	0.0011	1.00
P-2165	Lincoln Way	8	0.0025	1.00		P-3057	Vintage Oak Ave	21	0.0013	1.00
P-2169	Lincoln Way	8	0.0025	1.00		P-3071	Joy Dr	12	0.0020	1.00
P-2185	A Street	8	0.0025	1.00		P-3073	Joy Dr	12	0.0020	1.00
P-2189	A Street	8	0.0025	1.00		P-3081	Frankston St	8	0.0029	1.00
P-2193	A Street	8	0.0025	1.00		P-3099	Third Street	18	0.0015	1.00
P-2197	A Street	8	0.0025	1.00		P-3109	Downing Dr	6	0.0025	1.00
P-2201	A Street	8	0.0025	1.00		**P-3123**	First Street	10	0.0041	***0.78***
P-2205	A Street	8	0.0025	1.00		P-3125	First Street	10	0.0037	1.00
P-2209	A Street	8	0.0025	1.00		P-3127	First Street	10	0.0037	1.00
P-2213	A Street	8	0.0025	1.00		**P-3129**	First Street	10	0.0042	***0.77***
P-2225	A Street	6	0.0025	1.00		P-3135	Live Oak Ave	24	0.0010	1.00
P-2239	Third Street	18	0.0015	1.00		P-3139	Live Oak Ave	24	0.0010	1.00
P-2243	Third Street	18	0.0015	1.00		P-3141	Live Oak Ave	24	0.0010	1.00
P-2247	Third Street	18	0.0015	1.00		P-3143	Live Oak Ave	24	0.0009	1.00
P-2251	Third Street	18	0.0015	1.00		P-3149	Elm Ave	8	0.0020	1.00
P-2255	Third Street	18	0.0015	1.00		P-3153	McFarland Ave	10	0.0025	1.00
P-2263	E Street	6	0.0036	1.00		P-3155	McFarland Ave	12	0.0028	1.00
P-2265	E Street	12	0.0020	1.00		P-3157	McFarland Ave	12	0.0040	1.00
P-2267	Third Street	18	0.0015	1.00		P-3165	McFarland Ave	10	0.0024	1.00
P-2271	Fourth Street	12	0.0020	1.00		P-3179	McFarland Ave	8	0.0025	1.00
P-2275	Fourth Street	12	0.0020	1.00		P-3181	McFarland Ave	8	0.0025	1.00
P-2277	Fourth Street	12	0.0020	1.00		P-3183	McFarland Ave	8	0.0025	1.00
P-2285	H Street	12	0.0020	1.00		P-3185	McFarland Ave	8	0.0025	1.00
P-2289	H Street	12	0.0020	1.00		**P-3203**	Quail Hollow Dr	12	0.0021	***0.75***
P-2297	Joy Dr	12	0.0020	***0.73***		P-3267	First Street	6	0.0025	1.00
P-2311	Kost Road	10	0.0025	***0.72***		P-3269	First Street	6	0.0032	1.00
P-2327	E Street	6	0.0035	1.00		P-3273	Fourth Street	6	0.0046	1.00
P-2331	E Street	6	0.0035	1.00		P-3277	Oak Ave	6	0.0027	1.00
P-2335	E Street	6	0.0035	1.00						

Evaluation–The PWWF future condition would probably be the highest flow the sewer network could experience. The capacity analysis results are astonishing as, primarily all the main trunk sewer

segments in Carillion Boulevard and Live Oaks Avenue will be full in carrying the additional flow from the PD areas at build-out of the City. These pipe segments are only partially full at the existing PWWF. The older southern areas, which experienced problems under other scenarios, continue to display "red flags". However, some of these deficient segments may have been improved before the City reaches the build-out condition. The number of "red flags" will be reduced. Further, the County SCIS design criteria suggests that sewer lines 12-inch or larger may be designed to flow full when there is no direct (household) services connections to the line. Some full segments might not required replacement. A rough count indicates 52 or nearly half of the deficient sewer lines in the list are 12-inch or larger. If, after thorough investigation, it is found that no service connections were present for the sewer segment, and it would not cause a spill in the system, the larger full flow lines might not need to be relieved and be allowed to operate flowing full at the occasional PWWF conditions. The remaining full lines with diameter ranging from 6-inch to 10-inch would certainly need to be relieved with a parallel line or replaced with a larger diameter pipe. Increasing pipe slope is one other option, but the possible need to modify downstream pipe conditions might make this option not feasible.

***ScenarioC1**: Peak Dry Weather Flow (PDWF) – Existing*

Results – The total flow to the WWTP for this scenario is **4.29 mgd**. This flow is within the hydraulic capacity of the WWTP. Furthermore, one duty pump at the Live Oaks Lift Station will accommodate this flow to the WWTP; therefore, the estimated total (outlet) flow is reasonable. **Figure C1** shows the computer analysis results (see **Appendix 8** for tabulated list) for the PDWF at existing conditions. Pipes highlighted in red indicate the d/D ratio being greater than 0.7. The model analysis has identified **20** segments that will likely be unable to carry the PDWF at a d/D of less than 0.7 for the existing condition, and all of them are located in the older southern part of the City. The following **Table 11** lists all of the segments and their locations as identified by the Model for having deficient capacity:

Table 11 – PDWF Existing Deficient Segments

Pipe ID	Location	Diam	Slope	d/D		Pipe ID	Location	Diam	Slope	d/D
P-2189	A Street	8	0.0025	**0.76**		P-2335	E Street	6	0.0035	1.00
P-2193	A Street	8	0.0025	**0.76**		P-2339	E Street	6	0.0035	1.00
P-2197	A Street	8	0.0025	1.00		**P-2367**	Chabolla Way	6	0.0025	**0.72**
P-2201	A Street	8	0.0025	1.00		**P-2693**	Live Oak Ave	24	0.0010	**0.74**
P-2205	A Street	8	0.0025	1.00		P-3179	McFarland Ave	8	0.0025	1.00
P-2209	A Street	8	0.0025	1.00		P-3181	McFarland Ave	8	0.0025	1.00
P-2213	A Street	8	0.0025	1.00		P-3183	McFarland Ave	8	0.0025	1.00
P-2263	E Street	6	0.0036	1.00		P-3185	McFarland Ave	8	0.0025	1.00
P-2327	E Street	6	0.0035	1.00		P-3267	First Street	6	0. 0032	1.00
P-2331	E Street	6	0.0035	1.00		P-3269	First Street	6	0.0032	1.00

Evaluation – Again, the PDWF is the peak flow that the network could experience on a daily basis. This is a flow condition that the network must be sufficient to handle to avoid a problem occurring every day. Most of the deficiencies are due to small pipe diameters coupled with shallow pipe slopes. For example, most of the sewer lines in Meadowview subdivision were constructed at a slope of 0.0025, and have been carrying flow at below minimum velocity of 2 fps. As a result, solids are deposited and built up over time creating blockage in the lines. All lines in the known problem areas are being maintained in a fixed 3-month schedule. The computer analysis results also indicate all sewer segment in the northern part of the City have sufficient capacity to handle the current daily peak flows. The only large diameter pipe red-flagged as deficient is P-2693, which is a 20-foot segment of 24 pipe set a relatively flag slope for discharging the collected flow into the Live Oak Lift Station. Its calculated e/D is barely above the 0.7 default limit. This is not a concern to the existing system.

***Scenario C2**: Peak Dry Weather Flow (PDWF) – Future*

Results – The total flow to the WWTP for this scenario is **6.24 mgd**. If the WWTP were not expanded by then, there would still be sufficient hydraulic capacity for the WWTP to receive 6.24 mgd peak flow from the City sewer system. Most likely the Live Oaks lift station will be improved or expanded by then otherwise; the existing duty pump at the Live Oak Lift Station will not have the capacity to pump 6.24 mgd singly. **Figure C2** shows the computer results (see **Appendix 9** for tabulated list) for the PWWF at future conditions. There are 38 pipes appearing in the figure with a d/D greater than 0.7. Re-carving the tributary areas has increased the insufficient segments from 36 to 38. **Table 12** below identifies these pipes that are flowing at a d/D greater than 0.7.

Table 12 – PDWF Future Deficient Segments

Pipe ID	Location	Diam	Slope	d/D		Pipe ID	Location	Diam	Slope	d/D
P-1853	Trudy Way	10	0.0020	**1.00**		P-2339	E Street	6	0.0035	1.00
P-2157	C Street	8	0.0025	**0.71**		P-2367	Chabolla Ave	6	0.0025	1.00
P-2161	C Street	8	0.0025	**0.73**		**P-2473**	Elm Amador	10	0.0011	***0.71***
P-2165	Lincoln Way	8	0.0025	**0.73**		P-2501	Elm Amador	8	0.0011	1.00
P-2169	Lincoln Way	8	0.0025	**0.73**		P-2693	Live Oak Ave	24	0.0010	1.00
P-2185	A Street	8	0.0025	**0.74**		**P-2789**	Vintage Oak Ave	24	0.0004	**0.71**
P-2189	A Street	8	0.0025	1.00		**P-3109**	Downing Dr	6	0.0025	**0.75**
P-2193	A Street	8	0.0025	1.00		P-3149	Elm Ave	8	0.0020	1.00
P-2197	A Street	8	0.0025	1.00		**P-3153**	McFarland Ave	10	0.0025	**0.75**
P-2201	A Street	8	0.0025	1.00		**P-3165**	McFarland Ave	10	0.0024	**0.76**
P-2205	A Street	8	0.0025	1.00		P-3179	McFarland Ave	8	0.0025	1.00
P-2209	A Street	8	0.0025	1.00		P-3181	McFarland Ave	8	0.0025	1.00
P-2213	A Street	8	0.0025	1.00		P-3183	McFarland Ave	8	0.0025	1.00
P-2263	E Street	6	0.0036	1.00		P-3185	McFarland Ave	8	0.0025	1.00
P-2265	E Street	12	0.0020	1.00		P-3267	First Street	6	0.0025	1.00
P-2327	E Street	6	0.0035	1.00		P-3269	First Street	6	0.0032	1.00
P-2331	E Street	6	0.0035	1.00		P-3277	Oak Ave	6	0.0027	1.00
P-2335	E Street	6	0.0035	1.00						

Evaluation – The additional flow from the PD areas have caused an additional 15 segments of pipe to flow at a d/D higher than 0.7. Most of these additional deficient segments are, however, actually those segments that have caused to flow higher than 0.7 d/D due to additional flows. Further, the deficient sewer lines are located in the southern part of the City that has been mostly developed. The additional flow from PD area is quite limited. When the red flagged lines are improved to accommodate more flow and reduce maintenance, they will all be eliminated from the deficient sewer list.

SUMMARY

The existing sewer network as input for this capacity analysis has the capacity to handle the collected flows under most conditions. The more recently constructed northern part of the City fares better than

the southern part of the City because its sewer system was most likely constructed with contingent capacity as suggested in the County SCIS standards. The southern part of the sewer grids are older and the slopes used for the construction were mostly less than those recommended by SCIS standards. However, many of the lines in the network become deficient in the model analysis only when there is a downpour and the I/I have reached the design rate. Hydraulically, all the sewer lines in the troubled areas that have experience blockage and required frequent maintenance have not been "red flagged" for PDWF, the flow that the system experience daily. This was because the judging parameter for deficiency was not "less than minimum flow velocity" that may cause solids deposit and form blockages. To eliminate the problem lines, it will require individual evaluation of each problem line for improvement solutions.

The number of "red flagged" lines for future PWWF flow (Figure B1) showing up in the northern part of the City is quite alarming. It involves mostly the primary trunk sewers of 21 to 24 inches in diameter that deliver collected flows to the Vintage Oak and Live Oak lift stations. Because of their size, it would most likely be too congested to relieve them by a parallel pipe. Therefore, in planning the development of the PD areas, new trunk sewer lines will need to be installed to convey the flow to a new lift station for delivery to the WWTP. Increasing the capacity of the existing network is necessary to accommodate the additional flows from the PD area. Fortunately, the deficiency would not materialize until the PD areas are totally developed. There is still time for planning the improvement to eliminate all the "red flags" in the system. The Appendix of this report contains a total 11 appendices, which are:

- **Appendix 1 – Gravity Sewer Pipe Description Report**. It provides all the input of each sewer pipe segment of the computer model, which includes Pipe ID (Label), Inlet MH #, Outlet MH #, Diameter, Length, Material, Manning Coefficient (n), Upstream Invert, Downstream Invert and Slope.
- **Appendix 2 – Force Main Sewer Pipe Description Report**: It provides information of all the sewer pipes that carries flow discharged from a lift station, which includes Pipe ID (Label), Diameter, Length, Material and Hazzen-Williams Coefficient "C".
- **Appendix 3 – Sewer Manholes Description Report**: It provides basic description of all the manholes used in the computer model, which includes Manhole ID (Label), Diameter, Rim Elevations and Sewer Load.
- **Appendix 4 – Existing Lift Station Capacity**: It provides the pumping capacity of the duty pump of each station.
- **Appendix 5 – Wet Well Description Report**: It provides information of all the lift station wet well physical and operation levels.
- **Appendix 6 – Existing Average Dry Weather Flow (ADWF) Scenario Model Results**: It is a tabulated model run results of each sewer pipe segment, including Pipe ID (Label), Diameter, Velocity, d/D, Pipe Flow and Pipe Capacity. The pipe flow is the flow collected and received by each segment of pipe as calculated by the computer model.
- **Appendix 7 - Future Average Dry Weather Flow (ADWF) Scenario Model Results**
- **Appendix 8 – Existing Peak Dry Weather Flow (PDWF) Scenario Model Results**
- **Appendix 9 - Future Peak Dry Weather Flow (PDWF) Scenario Model Results**
- **Appendix 10 - Existing Peak Wet Weather Flow (PWWF) Scenario Model Results**
- **Appendix 11 - Future Peak Wet Weather Flow (PWWF) Scenario Model Results**

Chapter 6
SYSTEM IMPROVEMENTS

PRIORITIZATION

The principle end product of the Model capacity analysis is producing a list of deficient sewer lines that cannot handle the various present and future flow generation conditions. Several lists of deficient sewer lines have been presented as results of the scenario analysis in Chapter 5. The sewer lines that require immediate improvements should be those that are deficient for the existing conditions. The deficiencies from the existing conditions will have priority for near future improvement. The deficiencies for future conditions will be delayed until future growth. The computer results show all sewer lines in the model will handle the existing ADWF flow. However, when the PDWF is collected, 20 segments are identified deficient and 16 of them will be flowing full. As the PDWF is a flow that could occur daily, their model results will be used as a guide to set the priorities for the capacity improvements. Smaller sewer lines that will flow full during an existing PWWF condition will be the second priority improvements as it could happen only during the wet seasons. The larger than 12" diameter sewer lines that allows flowing full, if any improvement would needed, will be considered as third priority or forth priority.

The raise of red-flag may be due to one downstream segment at the end that does not have the capacity to carry the additional flow and cause a surcharge of several upstream segments, making them full and raising the red-flag. Possibly once this segment at the end is relieved, the surcharge effect is reduced and eliminated, the red flags of the upstream segment would be lowered. This explanation also applies to a relative flag segment in between might have cause a bottleneck and raised many red-flags upstream from the bottle neck. Therefore, additional modeling effort will be needed to confirm the need of upstream relieve sewer mains.

From reviewing the deficient sewer lines in **Table 9** and **Table 11**, the following priority lists were developed. In order to avoid upgrading for the future PWWF, the improvement sewer lines are sized for the estimated future PWWF. Therefore, when all the sewers are upgraded, it will eliminate the deficiency of all existing and future flows.

Table 13 – Priority 1 Sewer Lines – Smaller Sewer Lines flow full at existing PDWF

Pipe ID	Location	Diam	From MH	To MH or Wet Well	Length (ft)	D_P (in)	D_P Cost ($)	D_R (in)	D_R Cost ($)
P-2197	A Street	8	320	322	365	10	**59,150**	12	**80,700**
P-2201	A Street	8	322	324	275	12	**46,800**	14	**65,100**
P-2205	A Street	8	324	326	85	12	**14,458**	14	**20,100**
P-2209	A Street	8	326	328	185	12	**31,500**	14	**43,800**
P-2213	A Street	8	328	330	350	14	**63,750**	16	**89,000**
P-3179	McFarland Ave	8	330	886	635	14	**115,700**	16	**161,400**
P-3181	McFarland	8	886	888	600	14	**109,300**	16	**152,500**

	Ave								
P-3183	McFarland Ave	8	888	890	400	14	**72,900**	16	**101,700**
P-3185	McFarland Ave	8	890	WW-9026	40	14	**7,300**	16	**10,200**
P-3267	First Street	6	952	954	425	8	**57,000**	10	**89,600**
P-3269	First Street	6	954	WW-9030	340	8	**45,500**	10	**71,600**
P-2263	E Street	6	354	356	50	10	**8,100**	12	**11,100**
P-2327	E Street	6	388	354	175	10	**28,400**	12	**38,700**
P-2331	E Street	6	390	388	400	10	**64,800**	12	**88,500**
P-2335	E Street	6	392	390	400	10	**64,800**	12	**88,500**
P-2339	E Street	6	394	392	185	8	**24,800**	10	**39,000**
Total Improvement Costs							**814,000**		**1,151,300**

Note: D_P = Diameter of parallel pipe. D_P costs are costs to install a parallel pipe
D_R = Diameter of replacement pipe. D_R costs are cost to install a replacement pipe

Table 14 – Priority 2 Sewer Lines – Smaller Sewer Lines flow full at existing PWWF

Pipe ID	Location	Diam	From MH	To MH or Wet Well	Length (ft)	D_P (in)	D_P Cost ($)	D_R (in)	D_R Cost ($)
P-1837	Oberlin Way	10	114	120	150	8	**20,100**	12	**33,200**
P-1853	Trudy Way	10	120	122	250	10	**40,500**	12	**55,300**
P-2133	C Street	8	288	290	100	6	**11,400**	10	**21,100**
P-2137	C Street	8	290	292	275	6	**31,300**	10	**58,000**
P-2141	C Street	8	292	294	360	6	**41,000**	10	**75,800**
P-2145	C Street	8	294	296	512	6	**58,300**	10	**10,800**
P-2149	C Street	8	296	298	330	6	**37,600**	10	**69,500**
P-2153	C Street	8	298	300	450	6	**51,200**	10	**94,800**
P-2157	C Street	8	300	302	200	6	**22,800**	10	**42,100**
P-2161	C Street	8	302	304	300	6	**34,200**	10	**63,200**
P-2165	Lincoln Way	8	304	306	300	6	**34,200**	10	**63,200**
P-2169	Lincoln Way	8	306	308	300	6	**34,200**	10	**63,200**
P-2185	A Street	8	308	316	60	6	**6,800**	10	**12640**
P-2189	A Street	8	316	318	615	10	**99,700**	12	**136,000**
P-2193	A Street	8	318	320	100	10	**16,200**	12	**22,100**
P-2331	E Street	6	390	388	400	10	**64,800**	12	**88,400**
P-2367	Chabolla Ave	6	408	WW-9010	370	8	**49,600**	10	**78,000**
P-2473	Elm &	10	470	WW-	130	10	**21,100**	12	**28,800**

	Amador			9016					
P-2501	Elm & Amador	8	486	484	210	10	**34,000**	12	**46,400**
P-3109	Downing Dr	6	514	520	420	6	**47,800**	8	**73,100**
P-3149	Elm Ave	8	462	870	175	10	**28,400**	12	**38,700**
P-3153	McFarland Ave	10	828	874	20	10	**3,200**	14	**4,700**
P-3165	McFarland Ave	10	122	828	50	10	**8,100**	14	**11,800**
P-3277	Oak Ave	6	964	322	650	8	**87,100**	10	**136,900**
Total Improvement Costs							**883,300**		**1,424,900**

Table 15 – Priority 3 Sewer Lines – Smaller Sewer Lines flow less than full but higher than 0.7 e/D ratio at existing PWWF

Pipe ID	Location	Diam	From MH	To MH or Wet Well	Length (ft)	D_P (in)	D_P Cost ($)	D_R (in)	D_R Cost ($)
P-2051	Carillion Blvd	10	240	238	349	8	**46,700**	12	**77,200**
P-2071	Carillion Blvd	10	252	248	345	6	**39,300**	12	**76,300**
P-2361	Lincoln Ave	8	406	394	300	6	**34,200**	10	**63,200**
P-2491	Elm & Amador	10	480	470	430	8	**57,600**	12	**95,100**
P-2495	Elm & Amador	10	482	480	800	8	**107,200**	12	**176,900**
P-2497	Elm & Amador	10	484	482	350	8	**46,900**	12	**77,400**
P-2505	N. Lincoln Way	10	488	486	310	6	**35,200**	12	**68,600**
P-3051	Carillion Blvd	10	248	244	560	6	**63,800**	12	**123,800**
P-3053	Carillion Blvd	10	244	240	700	6	**79,700**	12	**154,800**
P-3273	Fourth St	6	956	328	650	6	**74,000**	8	**113,200**
Total Improvement Costs							**584,500**		**1,026,300**

Table 16 – Priority 4 Sewer Lines – Larger Sewer Lines flow full at existing PWWF

Pipe ID	Location	Diam	From MH	To MH or Wet Well	Length (ft)	D_P (in)	D_P Cost ($)	D_R (in)	D_R Cost ($)
P-2239	Third Street	18	342	344	325	12	**55,300**	21	**100,200**
P-2243	Third Street	18	346	342	415	12	**70,600**	21	**127,900**
P-2247	Third Street	18	348	346	385	12	**65,500**	21	**118,700**
P-2265	E Street	12	356	358	460	12	**78,200**	16	**116,900**
P-2267	Third Street	18	358	352	40	12	**6,800**	21	**12,300**
P-2271	Forth Street	12	360	356	315	8	**42,200**	14	**74,600**
P-2275	Forth Street	12	362	360	420	8	**56,300**	14	**99,500**
P-2693	Live Oak Ave	24	586	WW-9020	20	28	**5,900**	34	
P-2779	Vintage Oak Ave	21	640	644	241	14	**43,900**	24	**82,700**
P-2789	Vintage Oak Ave	24	650	652	125	21	**29,600**	28	**47,500**
P-3099	Third Street	18	344	WW-9028	55	14	**10,000**	21	**17,000**
P-3143	Live Oak Ave	24	866	586	600	16	**117,300**	28	**228,200**
P-3155	McFarland Ave	12	874	876	85	10	**13,800**	16	**21,600**
P-3157	McFarland Ave	12	876	WW-9026	25	10	**4,100**	16	**6,400**
Total Improvement Costs							**599,400**		**1,053,200**

The sewer mains in this **Priority 4** might have been designed to flow full. However, it might be only for a specific design flow. Additional flow might generate addition friction headlosses in the pipe and cause spill out of street manholes. Therefore, parallel or replacement pipes have been considered for their improvement. However, the requirement of several pipes, for example segment P-2789 in Vintage Oak Avenue, to install a large parallel pipe to relieve its capacity might not be feasible due to existing utilities in the street. Therefore, alternative routes for additional sewer mains could be necessary to intercept the additional flow to the existing sewer mains.

Included in **Priority 5** is a group of larger sewer mains (12" or larger and total 20 segments) in Table 9 that are not full at existing PWWF but become full at future PWWF. No cost estimates have been

prepared for these sewer mains due to their deficiency is associated with future flow factors cannot be predicted at the moment. There is a high possibility that new sewer mains will be installed to divert additional flows from these sewer mains to relieve their deficient capacity. Similar to **Priority 4**, adding an equal or larger parallel pipe next to an existing larger diameter pipe might not be a viable option. Replacing the existing pipe with a larger pipe will require a significant effort to bypass the flow during construction that might render the construction infeasible.

Priority 6 is a group of 29 sewer pipes in Table 10, but not in **Table 9**, that will flow full or higher than 0.7 d/D ratio at future PWWF at the built-out of the City's General Plan. Their pipe sizes range from 6" to 24". This group of pipe is considered as the last ones that the City could consider improving their capacities eventually. However, the need will depend on the ultimate development of the City and the installation of additional sewer mains to divert additional flows from these sewer mains might render the improvement of this group of sewer mains not necessary. As several of them are larger than 12", the existing utilities in the street might render it too congested to install a parallel pipe next to it and replacing it would require extensive bypass effort. For the same reason given to the Priority 5 group of sewer mains, not cost estimates have been prepared for this group of sewer mains.

UNIT COST ESTABLISHMENT

The Model has the ability to create design cost curves that estimate cost for pipe replacement and/or adding a parallel pipe to increase the capacity of the deficient sewer lines. A design cost curve was created based on unit costs input into the model separately for both replacement and parallel pipe addition. Based on references available, with the addition of 50% contingency for reconnection of services, temporary bypasses and other unknown factors in the street, in-place unit costs of sewer line are established as listed in **Table 17** for estimating construction costs to install a parallel line next to the deficient sewer main. Additional percentage of contingency has been added to account for costs associated with removal of existing pipes for installing a replacement sewer pipe. Again, actual costs would differ based on site and location conditions. These unit costs are the based of the cost estimates listed in Table 13 through 16 above.

Table 17 – Unit Costs Used for estimating construction costs

Pipe Diameter (inches)	Unit Cost ($/Linear Foot)
6	111
8	130
10	158
12	166
14	177
16	190
18	205
21	231
24	257
26	271
28	285

TROUBLE AREA SEWER MAINS

There 18 known sewer lines scattered among areas that have experienced problems in the existing system and they are in the following streets:

1) Quail Hollow Drive, Lorna Way, and Trudy Way to McFarland Lift Station
2) Meadowview Developments to First Street Lift Station
3) Galt Market area to Chabolla Lift Station
4) Fourth Street, Spruce Avenue, and Oak Avenue all North of A Street
5) F Street between First Street and Third Street

The City maintenance group identified the problem sewer mains, and they are expected to be red-flagged by the computer model. Unexpectedly, the 12" sewer mains in Quail Hollow Drive and Lorna Way were not red-flagged for any of the flow scenarios. The 10" sewer main in Trudy Way has been included in the **Priority 2** group for improvement. Very obvious, this 10" line is a bottle neck of the collection system as it was installed against general practices that downstream sewer lines should be at least equal in size of the immediate upstream pipe if not bigger. This will avoid clogging. Once the 10" sewer line in Trudy Way is relieved, theoretically, the surcharge of the upstream sewer lines should be reduced if not eliminated. Another cause of problem is most likely due to slow flow velocity that causes solid deposit or grease build-up in the sewer lines. The deposit will create a blockage in the sewer line and restrict the sewage flow. It has not been set that slow flow velocity is a red-flag element in the model run as an evaluation factor. If the current problem persists, a survey of the manholes may be required to further determine the problems. The deficient segment in Oberlin Way has been included in the Priority 2 group of sewer mains for improvement.

It was also expected the sewer main in Meadow View Drive would be red-flagged by the computer model. However, they are not red-flagged to show deficient in all the flow scenarios. Similar to the 12" sewer mains in Quail Hollow Drive, the problem might have been caused by slow sewage flow velocity that have caused solids and grease deposit and clogs the pipe. Frequent maintenance will most likely reduce the problem. The deficient 6" sewer pipe in Downing Drive has been included in the Priority 2 group of sewer mains required improvements.

The identified problem segments in Chabolla Way from the Galt Market to the Chabolla Lift Station may be due to the high flow from the market gathering, and the possibility of higher amount of solids and grease poured into the sewer segment that causes clogging. The improvement of this sewer segment has been included in the Priority 2 group of sewer mains that required improvements.

The improvement of the 6" sewer line in Oak Avenue has been included in the Priority 2 group of sewer mains required improvements.

Problem mains in First, Third and Fourth Streets have been included among sewer mains requirements in the priority groups.

The 6-inch sewer mains on F Street near the convalescent home that have on occasion backed up into the building were not incorporated into the model due to lack of sewer drawings for that area. Therefore, this problem area was analyzed separately and it has been determined that increasing the size

of these mains would sufficiently resolve the problem of inadequate capacity. The mains running along F Street should be replaced with 8-inch lines.

LIFT STATIONS

Table 18 presents a summary of all the flows the computer model has calculated for each lift stations. Following is an evaluation of each lift station:

Crystal Lift Station – Both the initial flow estimates in **Chapter 4** and the computer model runs have estimated a much lower flow than the current capacity of the Crystal Lift station. It might have been designed to accept some of the Chabolla Lift Station flows in the future. Meanwhile, the pump station will not have problem to handle the existing and future estimated flows. No improvement is recommended for this lift station.

Kost Lift Station – The computer model has estimated an existing and future PWWF much higher than shown in **Figure 4 and 5**, but closer to the rated capacity of the pump station. Similar to Crystal Lift Station, the existing duty pump has the capacity to deal with the existing PWWF. In the future, it will need the help of the standby pump to handle the estimated future PWWF flows. It probably will not be necessary to increase the pumping capacity of the lift station for future flows. However, the structure integrity of the wet well should be evaluated for necessary improvement if corroded.

Chabolla Lift Station - The computer has estimated higher flows than shown in **Figures 4 and 5**. The estimated flow for the existing PWWF is just 11% than the rated capacity of the existing pumps. For future PWWF, the duty pump itself could not pump the estimated flow. With the help of the standby pump, the lift station will be able to handle the future PWWF without increasing its pump capacity. However, the pump station structure integrity should be evaluated separately if improvement is needed to extend its year of services.

E Street Lift Station – The computer model run estimated future PWWF flow for the E Street Lift Station is only about 45% of the station's pumping capacity. It has spare capacity to receive additional flow. No improvement is recommended if the wet well structure is sound to serve the future flows.

Elm & Amador Lift Station – The computer model run estimated a lower inflow for the existing PWWF, but almost identical for the future PWWF flow. However, the existing station has the capacity to handle the estimated future flow. Therefore, no capacity increase is recommended, but wet structure integrity improvements require separate evaluations.

Live Oak Lift Station – The existing Live Oak Lift Station does not have the capacity to handle the estimated PWWF flows even with the help of the standby pump. It is more than 10% deficient in pumping the existing PWWF flows. The temporary storage in the existing system might have lowered the existing peak flow to within its pumping capacity. As a result, no alarm has been raised for its deficiency. As for the future PWWF, the lift station is certainly not big enough to handle the flow. Expansion is necessary, however, will depend on whether all flows from future developments will be piped to this lift station. If that is the plan, besides new higher capacity pumps, a new wet well and a new force main will be necessary for conveying the flows to the City WWTP. Carrying the estimated

9.1 mgd flow to the WWTP will generate a flow velocity of 10 fps that might not tolerated by the existing ACP force main for a long period. Moreover, increasing the existing gravity sewer mains to carry more flow to the lift station site might not be feasible due to utility congestion in the service street. It is recommended that a new lift station will be constructed to receive additional flows from new developments and a new force main will be constructed to deliver the flow to the City WWTP. As Boyle understands the wet well structure of the existing lift station has been corroded beyond repair. A new station will need to be constructed near the existing.

Vintage Oak Lift Station – Due to the many designated PD-developments near the Vintage Oak Lift Station and they have been included in the lift station's tributary area, the existing lift station does not and will not have the capacity to handle the estimated existing and future PWWF flows by just running the duty pump. The standby pump can help pumping the estimated PWWF flow, but it will not be sufficient for the future PWWF flows. Because of the additional flow, it has also red-flagged many segments of the gravity sewer mains in which the pump station discharge its flow for conveying to the Live Oak Lift Station. This lift station requires upgrade for the future PWWF flows. However, the upgrade could be eliminated if a new station would be constructed to receive the new development flows and pump it to the WWTP directly. This will relieve the extra loading on the gravity mains and the16" force main of the Live Oak Lift Station. No capacity increase of the lift station is recommended since it is sufficient to handle the current PWWF. Since it is a relatively new station, the structure should be sound and no improvement would be needed in the near future.

McFarland Lift Station – A new lift station is being constructed to replace the existing. Because it shares a 14" force main with the A Street Lift Station, it has two rated pumping capacities. When pumping alone, the duty pump can pump about 1.9 mgd. However, when A Street Lift Station is pumping, the pump rate is reduced to about 1.3 mgd. For the estimated existing PWWF of about 3.0 mgd (see **Table 18**), running both pumps will probably handle the incoming flow with some storage in the sewer system. However, for the future PWWF, it will not be sufficient. The lift station capacity will need to be increased or it would be necessary to let some of the additional flow diverted to another lift station. At the moment, actually part of the McFarland flow is diverted to the A Street Lift Station when there is too much for McFarland pumps to pump. This practice helps in handling the existing PWWF as the combined lift station capacity will pump a total of about 6.9 mgd, which is more than the combined inflow of about 6 mgd estimated for the stations. For the future flow, the standby pumps will need to be switched on to help pumping. The lift station has the capacity to handle the estimated flows, and it deems immediate improvement for the McFarland Lift Station is not necessary. However, the continuation of this practice is viable only when the existing 14" force main has the integrity to carry flow at 11 fps regularly without causing extensive scouring damages. Therefore, the foreseeable improvement to the lift station would be replacing the existing force main with a new one. If the configuration of the new pump station allows, eventually the existing pumps can be replaced easily with higher capacity pumps if necessary. Since the standby pump will be necessary for the high flows, both pumps should be in condition during the wet months. All necessary repairs should be undertaken during the dry months.

A Street Lift Station – Similar to the McFarland Lift Station, the duty pump of the station can pump up to 2.8 mgd when pumping alone, and it reduces to 2.6 mgd when the McFarland duty pump is pumping. The duty pump is not large enough to pump the estimated PWWF flows alone, but the station become sufficient when both pumps are pumping for the high flows. Therefore, it is important that both pumps

are in good condition during the wet months of the year. Eventually, the existing pumps can be high capacity pumps if the pump station configuration allows. No immediate improvements for the lift station is recommended, but new force main may need to be installed to supplement the 14" force main when its condition is evaluated to have been deteriorated.

First Street Lift Station – The computer model runs have estimated higher flows for the First Street Lift Station than presented in **Figure 4 and 5**. This is most likely due to the inclusion of the flows to be generated by the PD-zone located southeast of the lift station. When both pumps are pumping, the station will handle the existing PWWF flow, but will not be in the future. The deficiency of the this lift station can be resolved by replacing the existing pumps with higher capacity pumps if the existing wet well configuration allows or by sending the additional flow from future development to a new lift station or the Kost Lift Station nearby provided the gravity system could handle the additional flows. Further study will be necessary to select the final improvement approach.

Sparrow Lift Station – It serves a tributary area that is well established and located next to the west City boundary. Unless the City would annex lands to its east, there would most likely be no additional flow sent to the Sparrow Lift Station even though it has up to 30% spare capacity to handle extra flow. However, whether the station can receive additional flow will also depend on whether the gravity system to which the lift station discharges has the capacity to handle the additional flow. For this report, no improvement of the lift station is recommended.

Table 18 - Summary of Model Calculated Flows of Lift Station

Lift Station Name	Lift Station ID	Existing pump capacity, mgd	Existing ADWF, mgd	Future ADWF, mgd	Existing PDWF, mgd	Future PDWF, mgd	Existing PWWF, mgd	Future PWWF, mgd
Crystal	WW-9004	**0.43**	**0.003**	**0.005**	**0.011**	**0.016**	**0.025**	**0.032**
Kost	WW-9008	**0.55**	**0.083**	**0.126**	**0.218**	**0.316**	**0.474**	**0.617**
Chabolla	WW-9010	**0.43**	0.073 + 0.008 = **0.081**	0.11 + 0.012 = **0.122**	0.194 + 0.027 = **0.221**	0.281 + 0.039 = **0.320**	0.421 + 0.059 = **0.480**	0.548 + 0.077 = **0.625**
E Street	WW-9012	**0.72**	**0.042**	**0.063**	**0.118**	**0.171**	**0.257**	**0.334**
Elm & Amador	WW-9016	**0.72**	**0.107**	**0.163**	**0.274**	**0.397**	**0.595**	**0.775**
Live Oak	WW-9020	**4.44**	**2.294**	**3.489**	**4.187**	**6.082**	**9.107**	**11.86**
Vintage Oak	WW-9022	**2.16**	**0.772**	**1.174**	**1.588**	**2.306**	**3.453**	**4.497**
McFarland	WW-9026	**1.27 – 1.90**	0.298 + 0.298 = **0.596**	0.453 + 0.453 = **0.906**	0.68 + 0.681 = **1.361**	0.988 + 0.989 = **1.977**	1.479 + 1.48 = **2.959**	1.927 + 1.928 = **3.855**
A Street	WW-	**2.59 –**	**0.652**	**0.992**	**1.367**	**1.985**	**2.972**	**3.871**

	9028	**2.80**						
First Street	WW-9030	**0.36**	0.038 + 0.083 = **0.121**	0.058 + 0.126 = **0.184**	0.109 + 0.218 = **0.327**	0.158 + 0.317 = **0.475**	0.236 + 0.475 = **0.711**	0.307 + 0.618 = **0.925**
Sparrow	WW-9032	**1.00**	**0.095**	**0.145**	**0.246**	**0.358**	**0.536**	**0.697**

OVERALL EVALUATION

From the results of the computer model runs, there is no immediate need to increase the pumping capacity of the existing lift stations. However, they might need immediate structural repairs or piping improvement due to corrosion from many years of services. A thorough evaluation of each lift station is recommended in a separate project to determine structure integrity of the lift stations for repair or replacement. One significant finding of the capacity analysis is that the City's WWTP could receive a peak flow of 9.0 mgd as the plant was design hydraulically. However, it may not receive a peak flow of 18.00 mgd as forecast. There are deficiencies in the existing WCS, but not significant, and they can all be improved

City of Galt Wastewater Collection System Capacity Analysis Phase I

City of Galt

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Associate Civil Engineer
Public Works Department

Boyle Engineering Corporation

Project Manager — Jerry Au, PE

Associate Engineer — Noe Martinez, PE

Assistant Engineer — Kari Holmes, EIT

SC-G21-110-10

October 2005

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FIGURE 1
LOCATION MAP
GALT, CALIFORNIA

FIGURE 2
CITY OF GALT
ZONING MAP

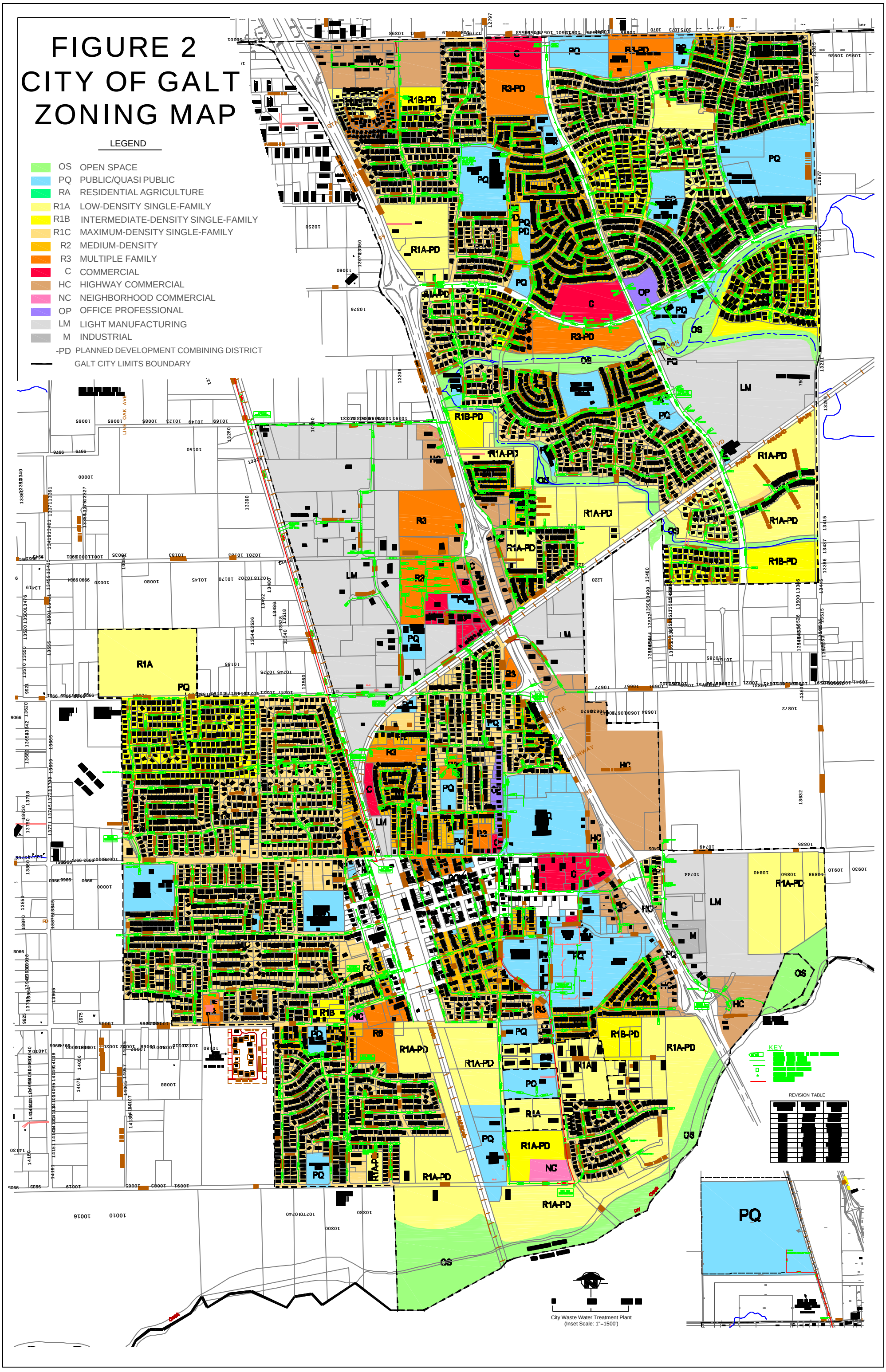

FIGURE 3
PUMP STATION TRIBUTARY AREAS

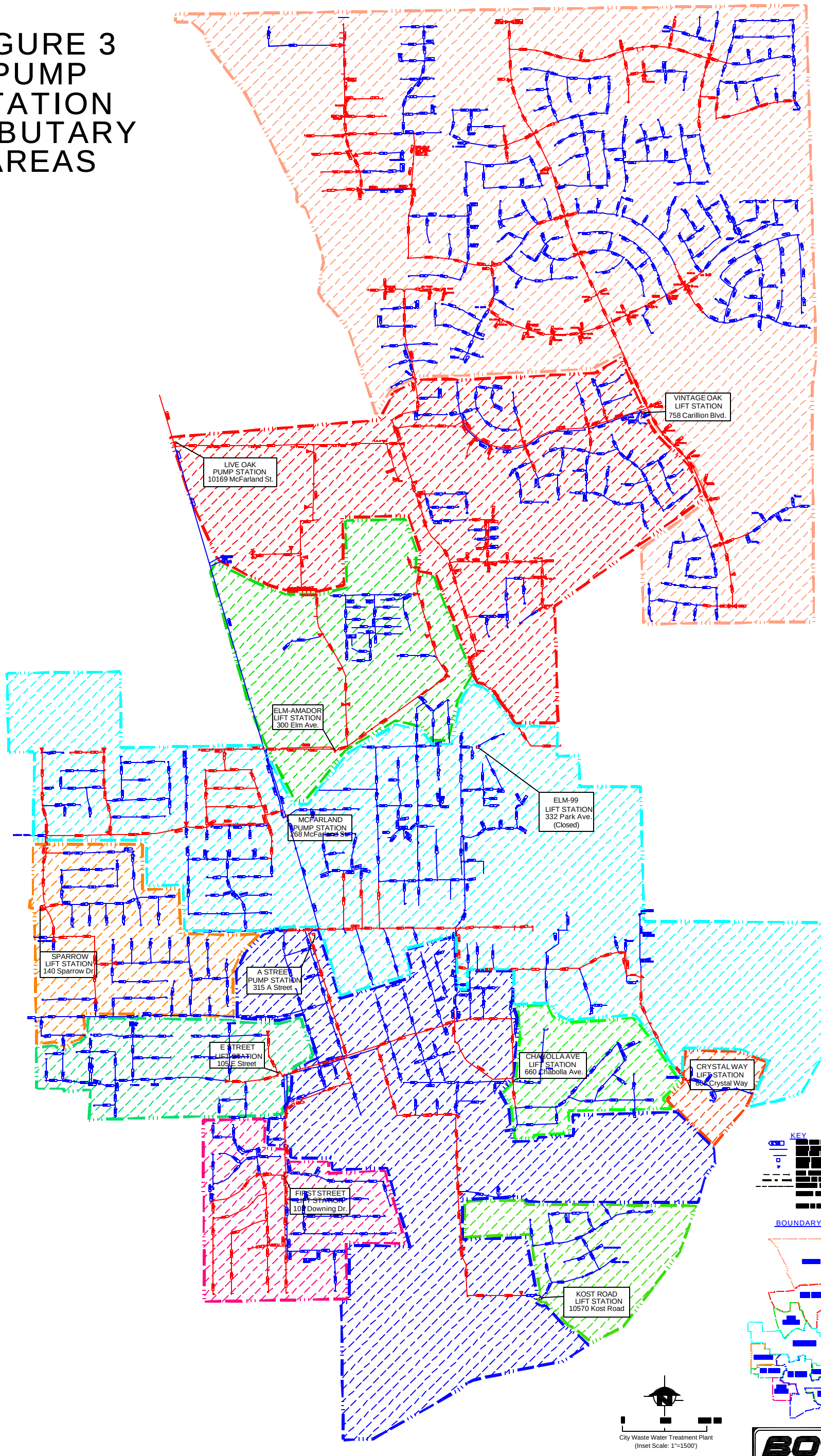

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Figure 4: PWWF Flow to Each Lift Station – Existing Conditions
Hand Calculated Based on Lift Station Tributary Areas

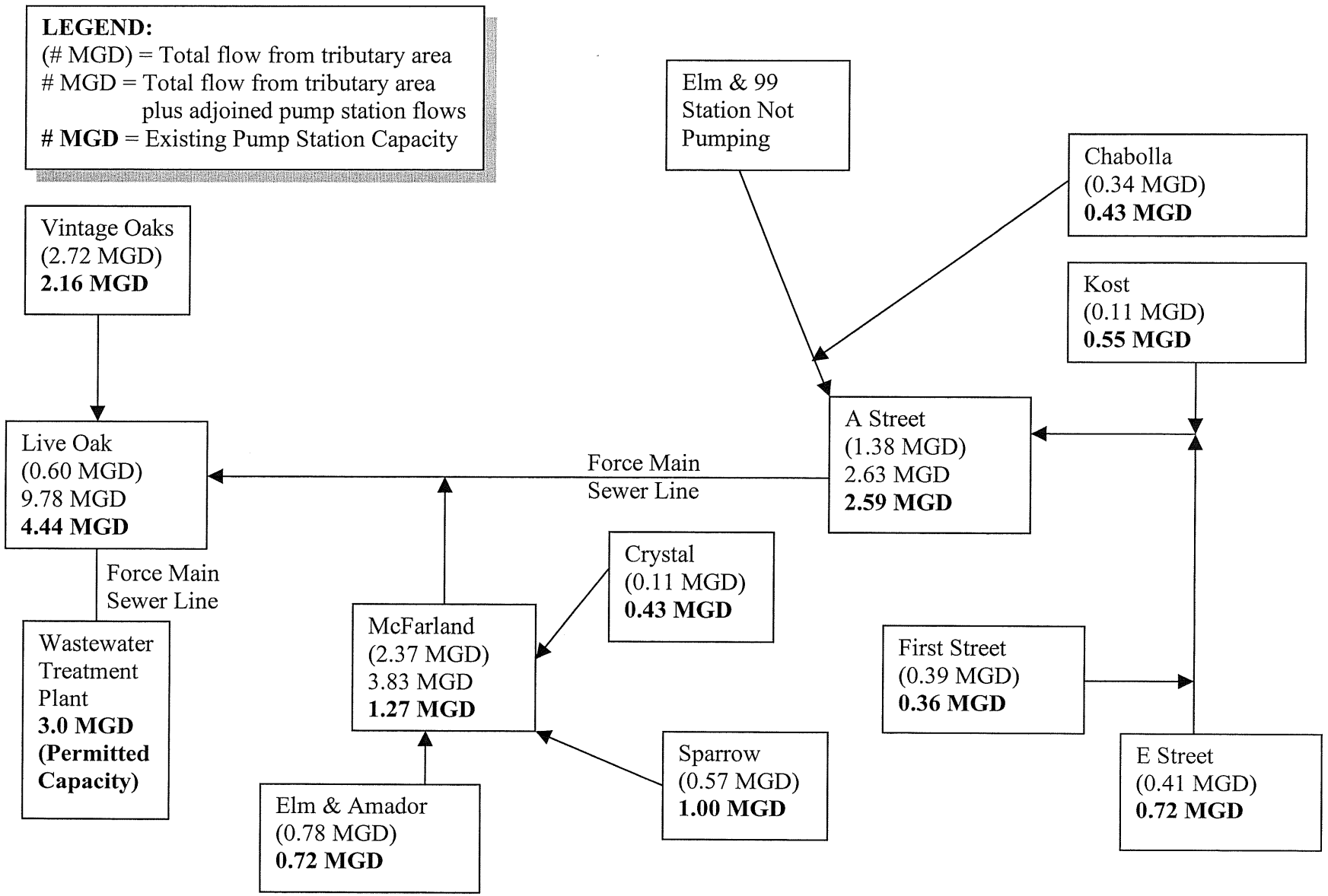


Figure 5: PWWF Flow to Each Lift Station – Including Future Build-out
Hand Calculated Based on Lift Station Tributary Areas

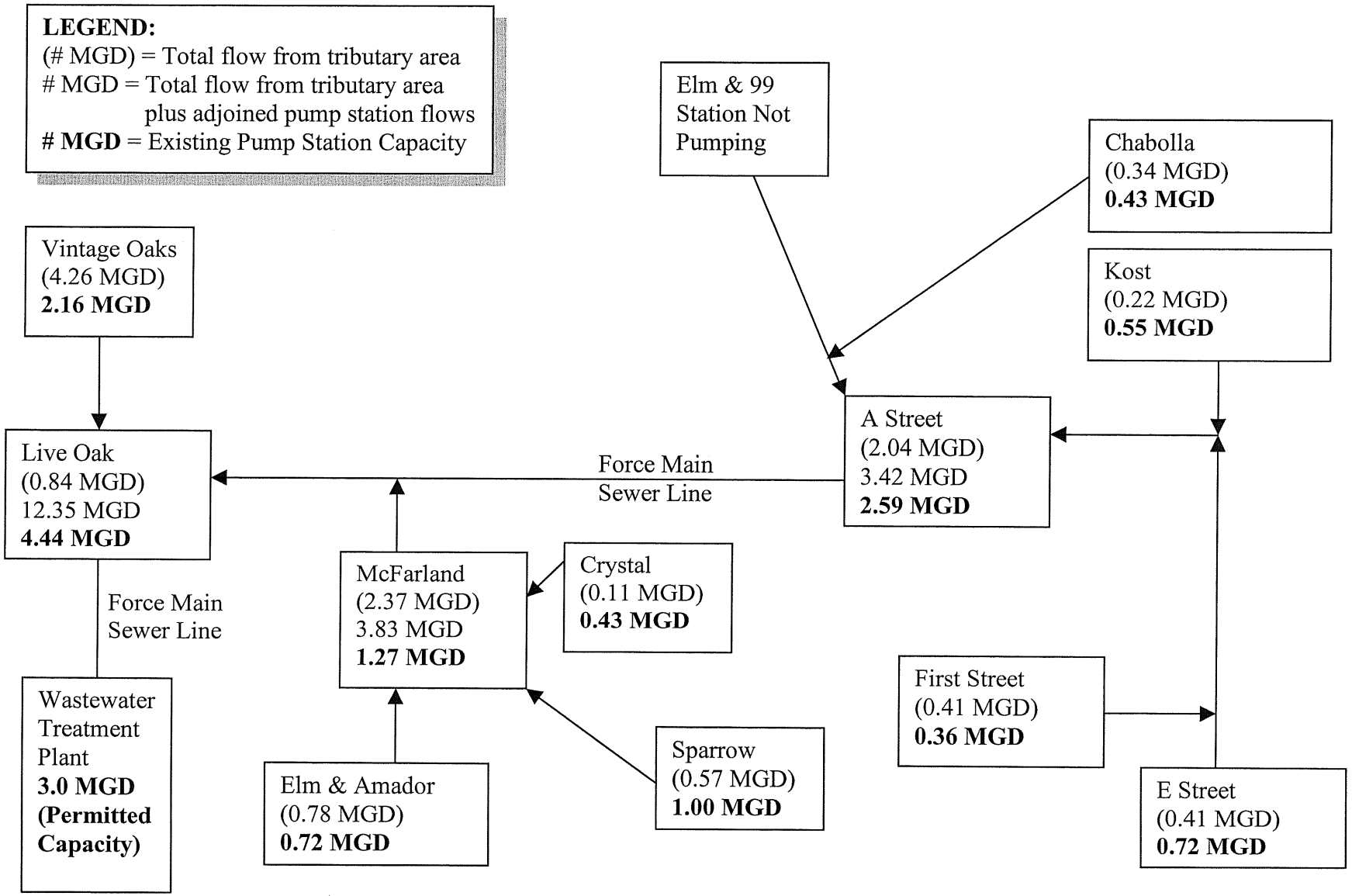
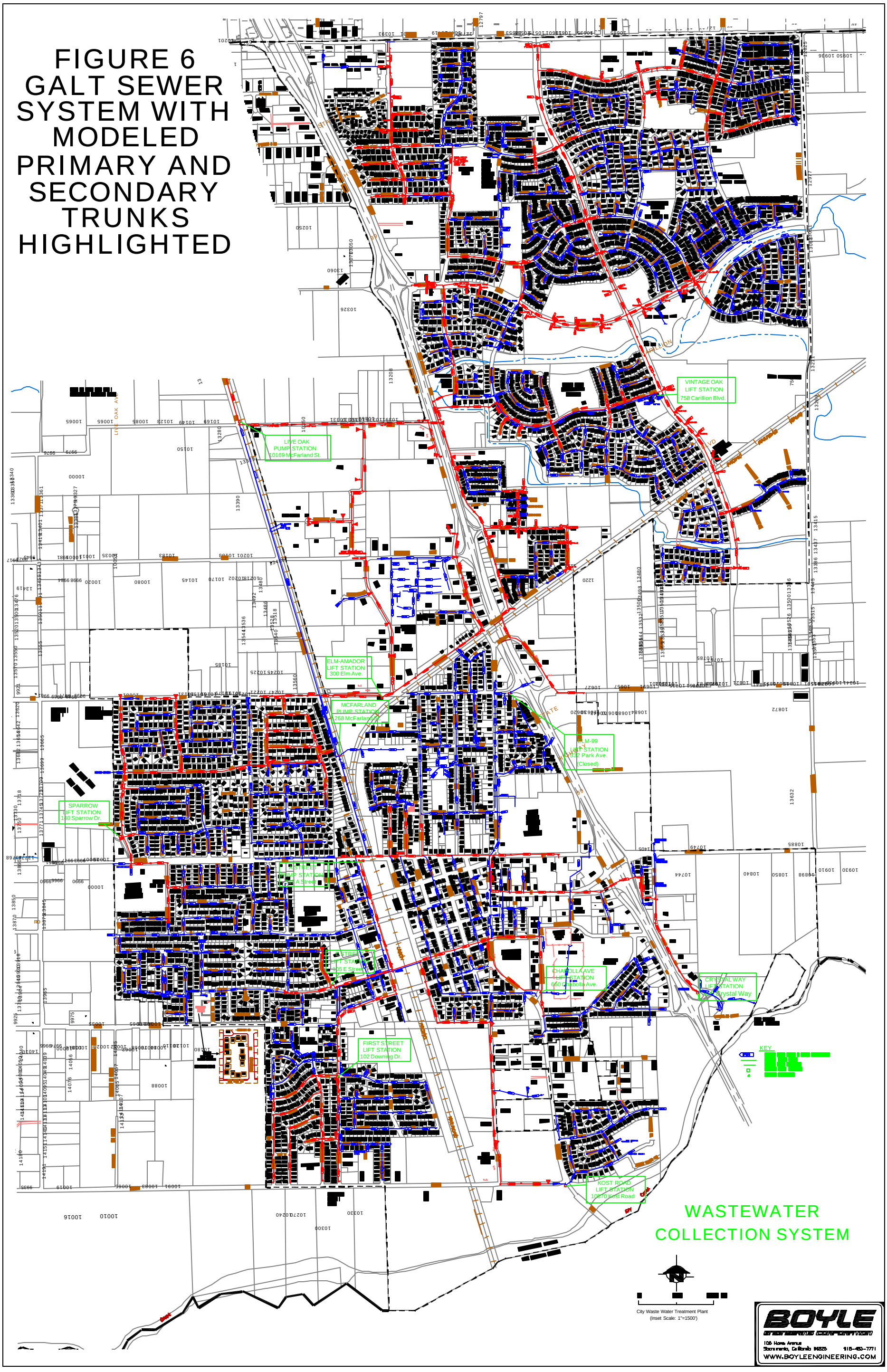
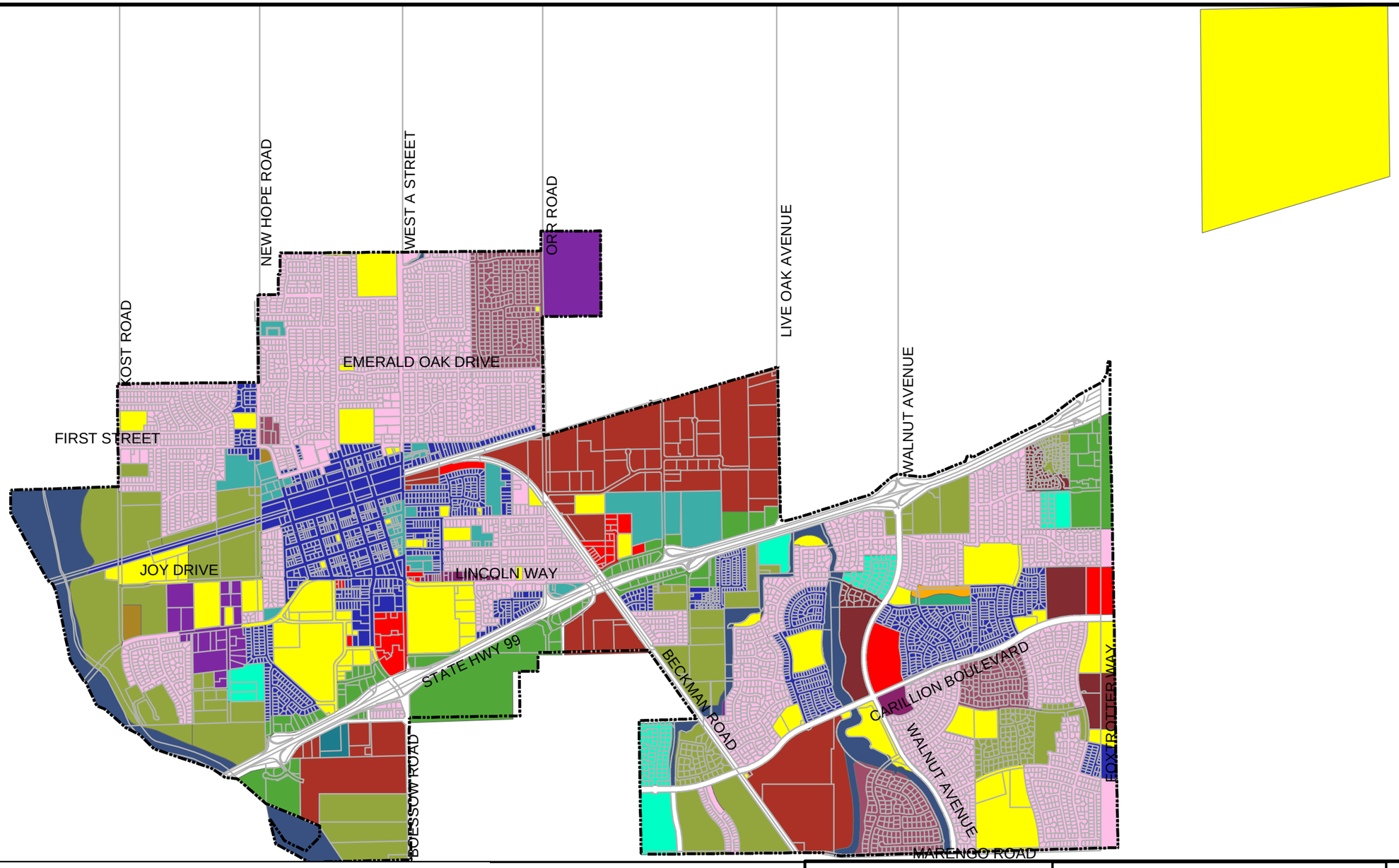


FIGURE 6
GALT SEWER SYSTEM WITH MODELED PRIMARY AND SECONDARY TRUNKS HIGHLIGHTED





LEGEND

- City Limits
- Streets

C	M	OS	R1A	R1B_PD	R2_PD
HC	NC	PQ	R1A_PD	R1C	R3
LM	OP	PQ_PD	R1B	R2	R3_PD

SCALE: 1" = 2000'

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CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

EXISTING ZONING MAP

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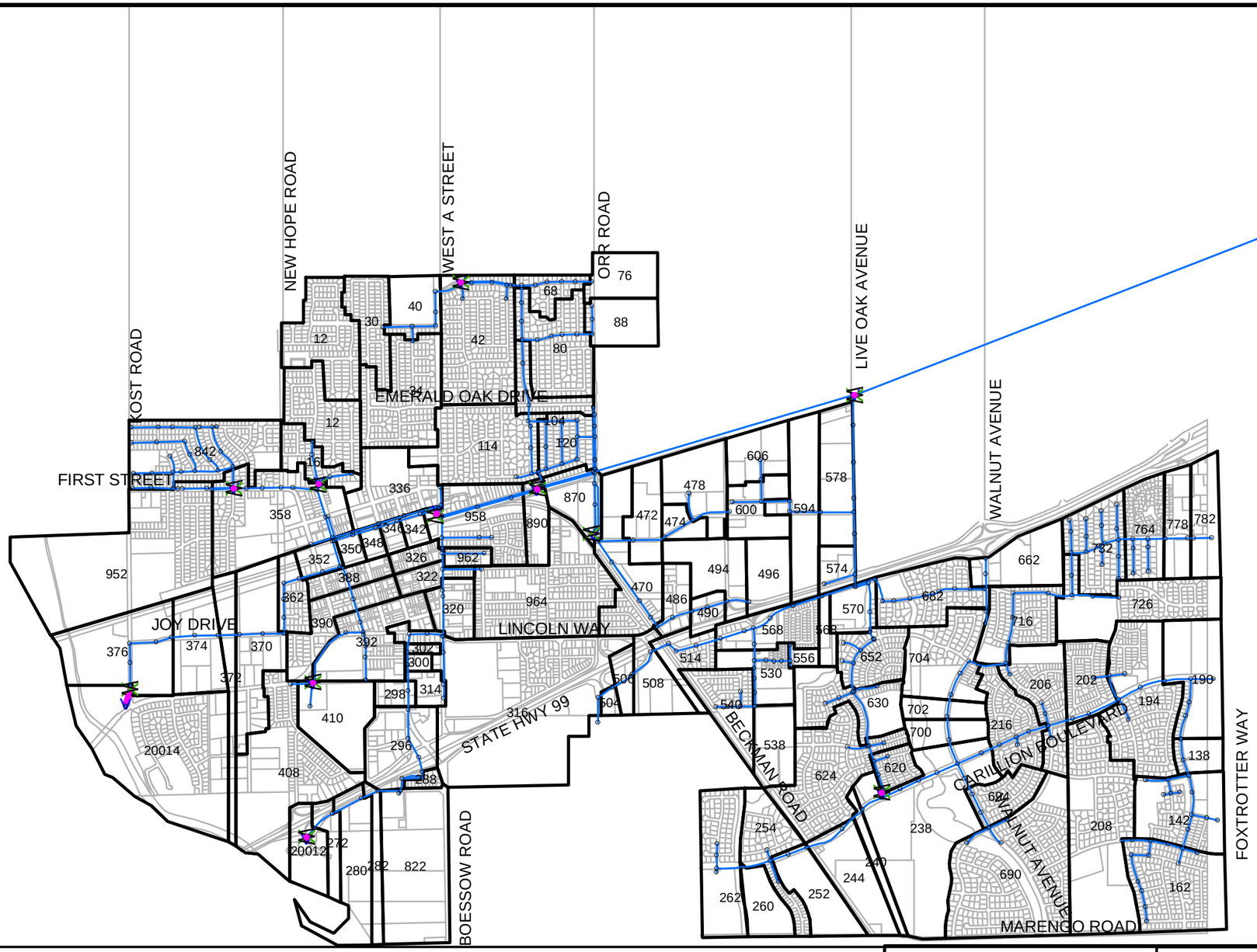
SC-G21-110-10

FIGURE
7



LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

Pipes

- Mh Tributary Area

SCALE: 1" = 2000'

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CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

MH LOADING TRIBUTARY AREA

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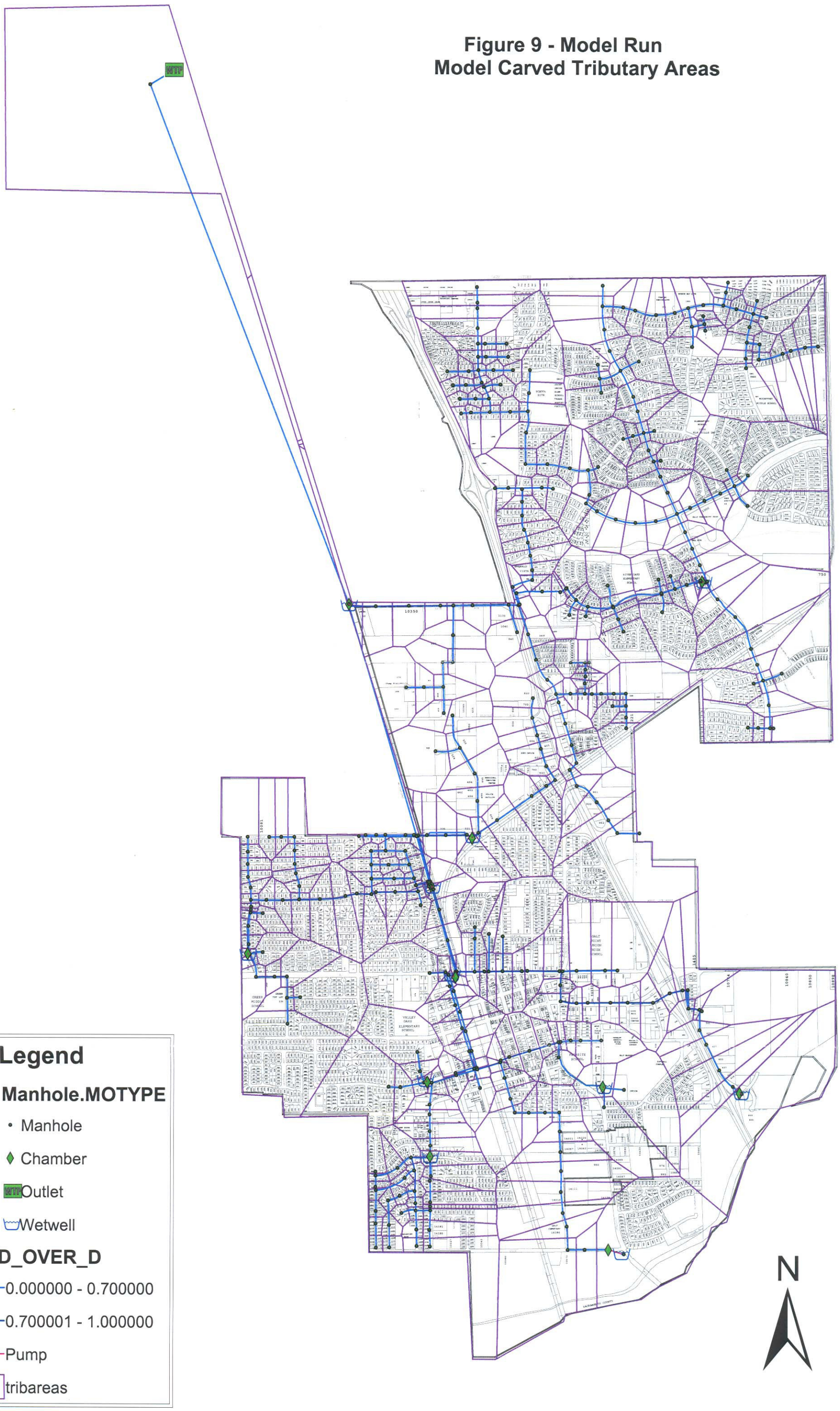
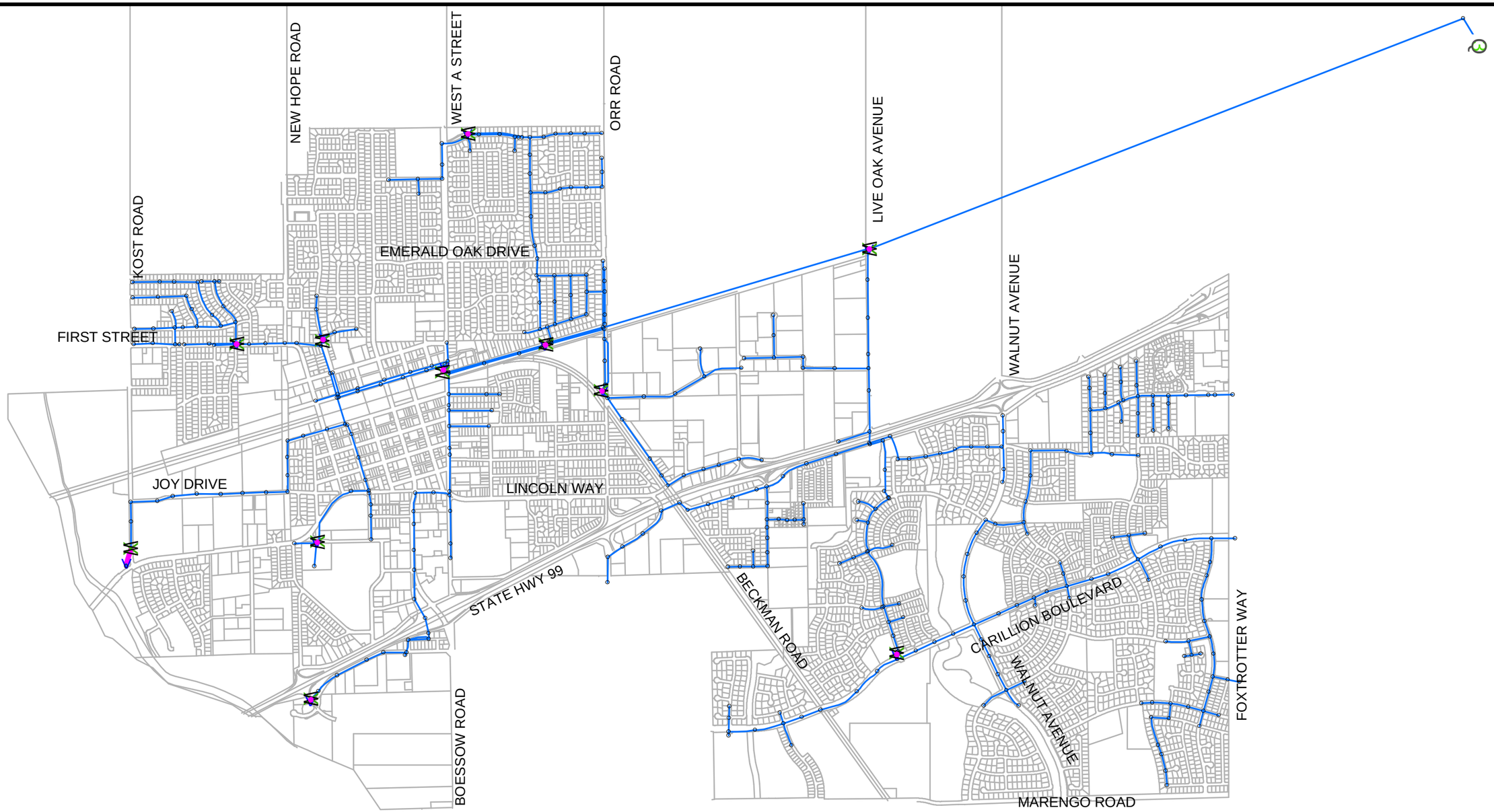
FIGURE
8

Figure 9 - Model Run
Model Carved Tributary Areas





LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

D_OVER_D

- 0.000000 - 0.700000
- 0.700001 - 1.000000

Pipes

SCALE: 1" = 1700'

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CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

EXISTING ADWF SCENARIO

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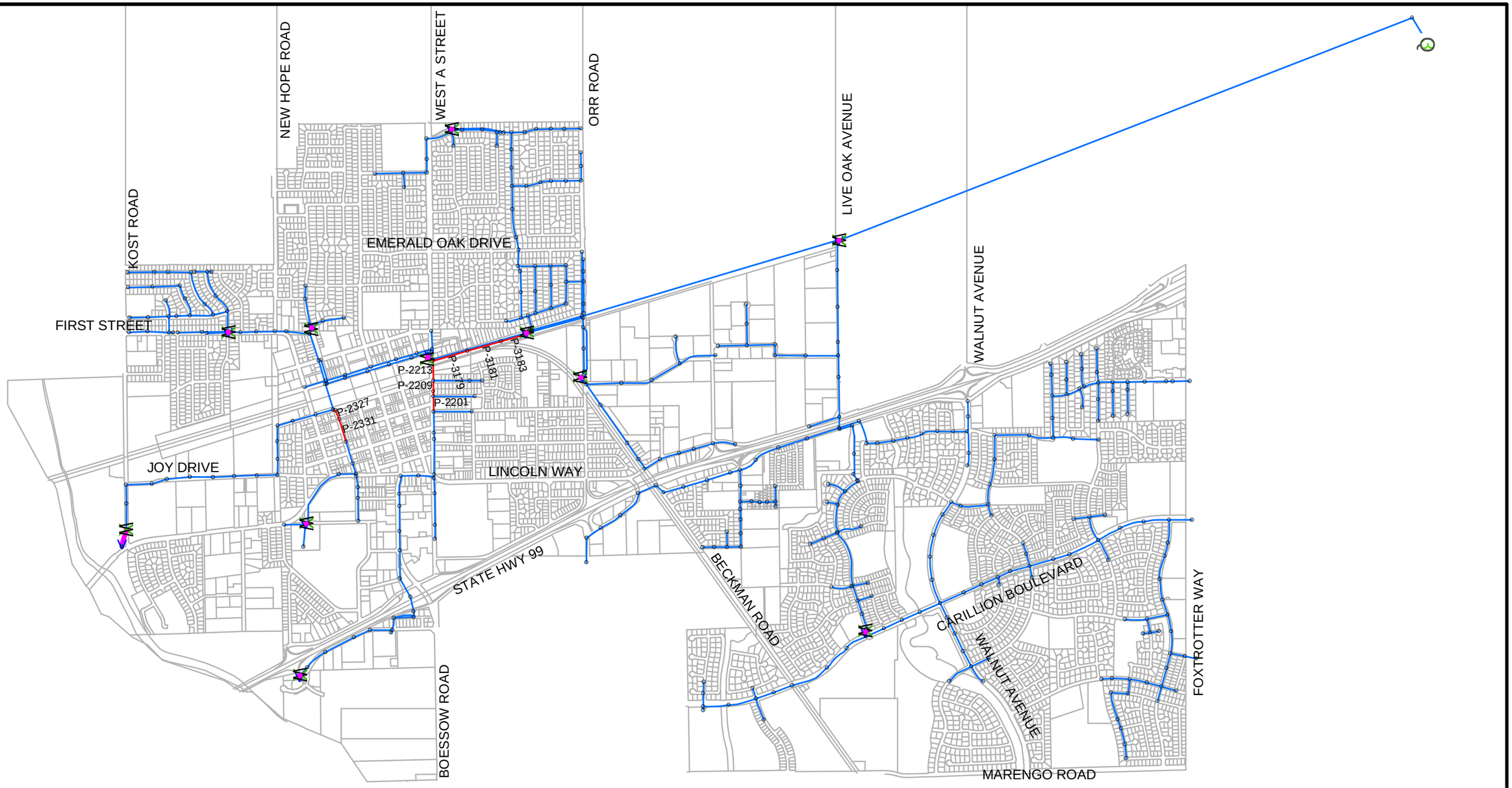
SC-G21-110-10

FIGURE
A1



LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

D_OVER_D

- 0.000000 - 0.700000
- 0.700001 - 1.000000

Pipes

SCALE: 1" = 1700'

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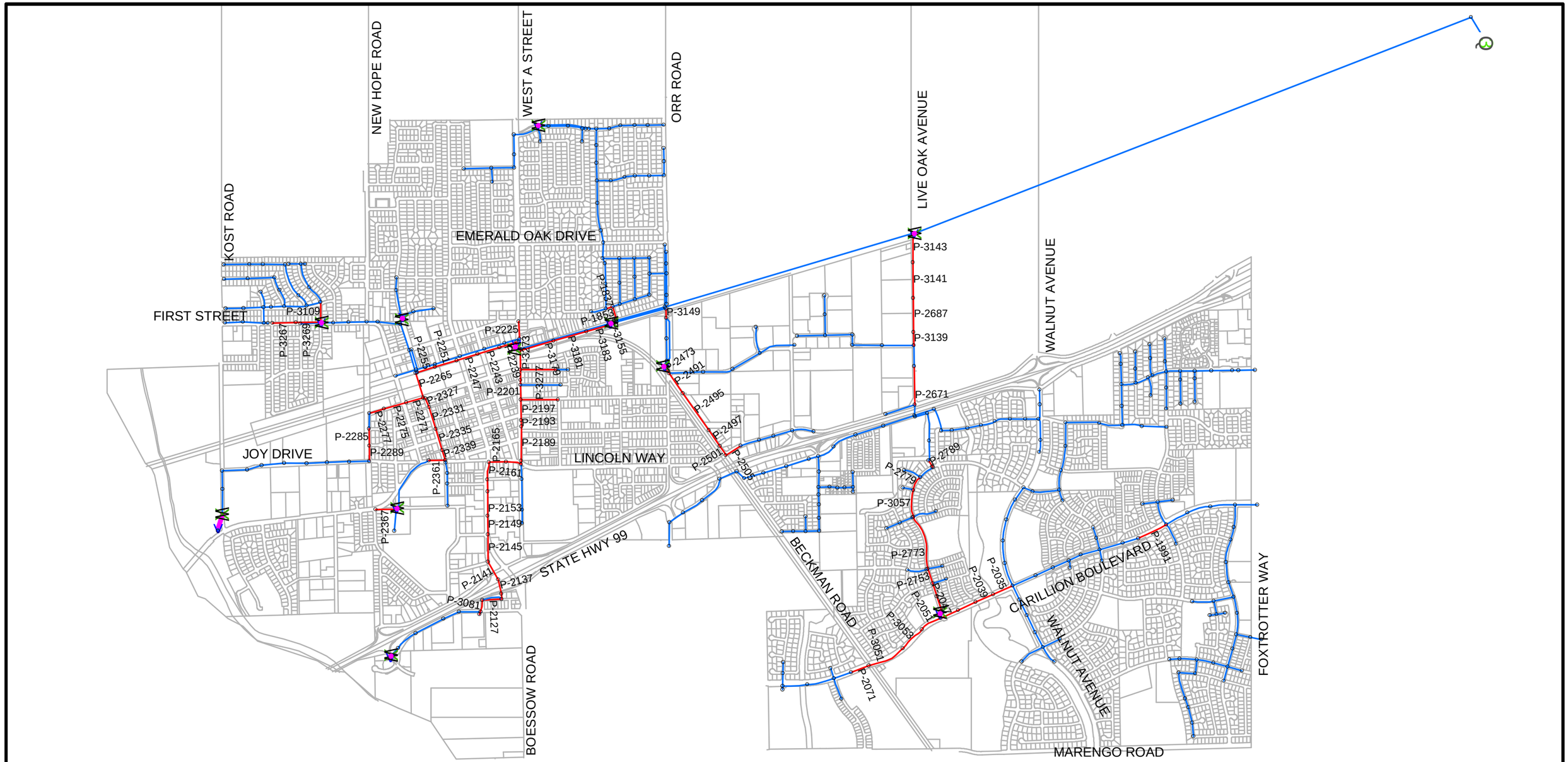
CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

FUTURE ADWF SCENARIO

BEC PROJECT NO. SC-G21-110-10

FIGURE A2



LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

D_OVER_D

- 0.000000 - 0.700000
- 0.700001 - 1.000000

Pipes

SCALE: 1" = 1700'

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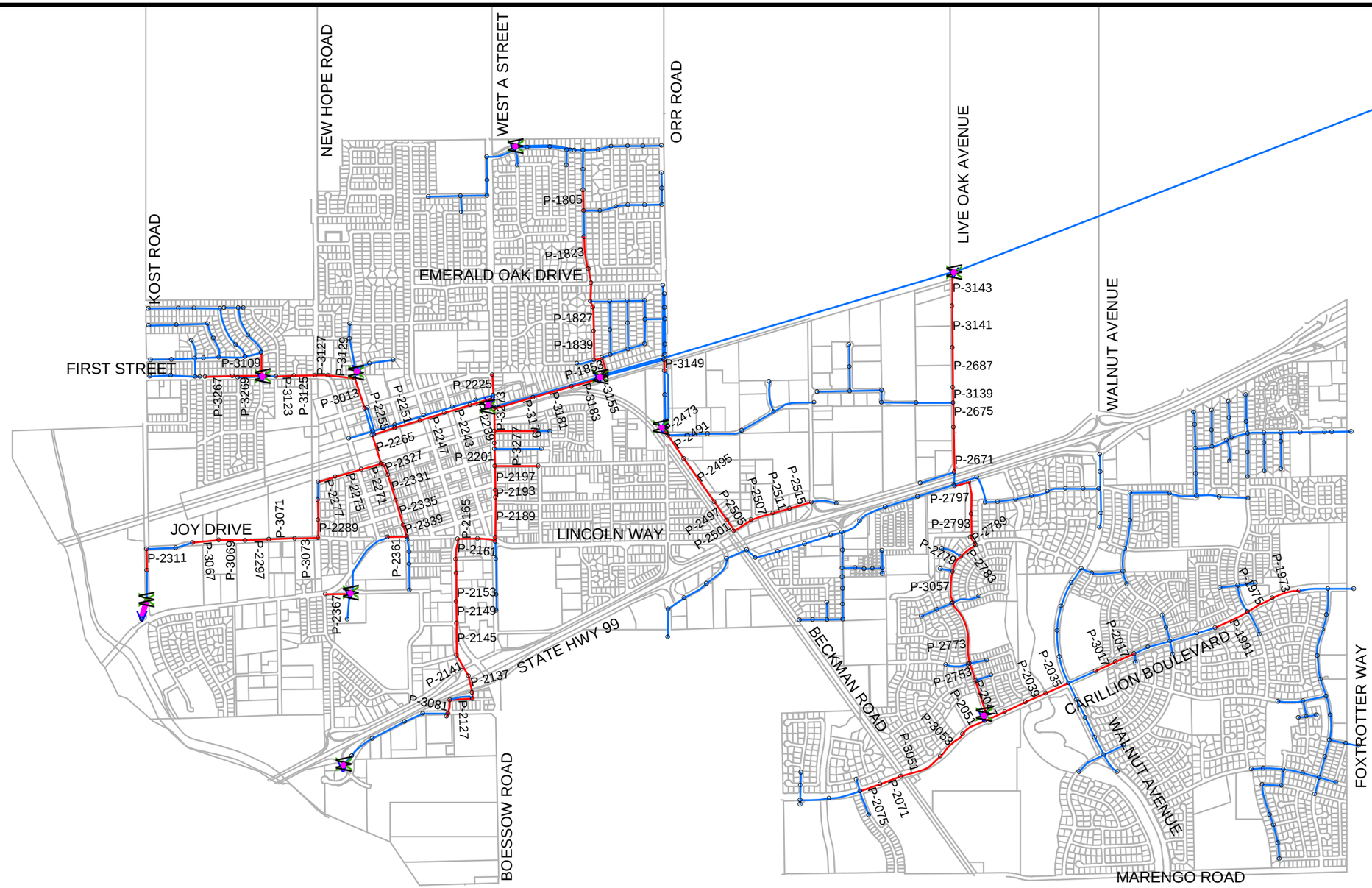
CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

EXISTING PWWF SCENARIO

BEC PROJECT NO.
SC-G21-110-10

FIGURE
B1



LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

D_OVER_D

- 0.000000 - 0.700000
- 0.700001 - 1.000000

Pipes

SCALE: 1" = 1700'

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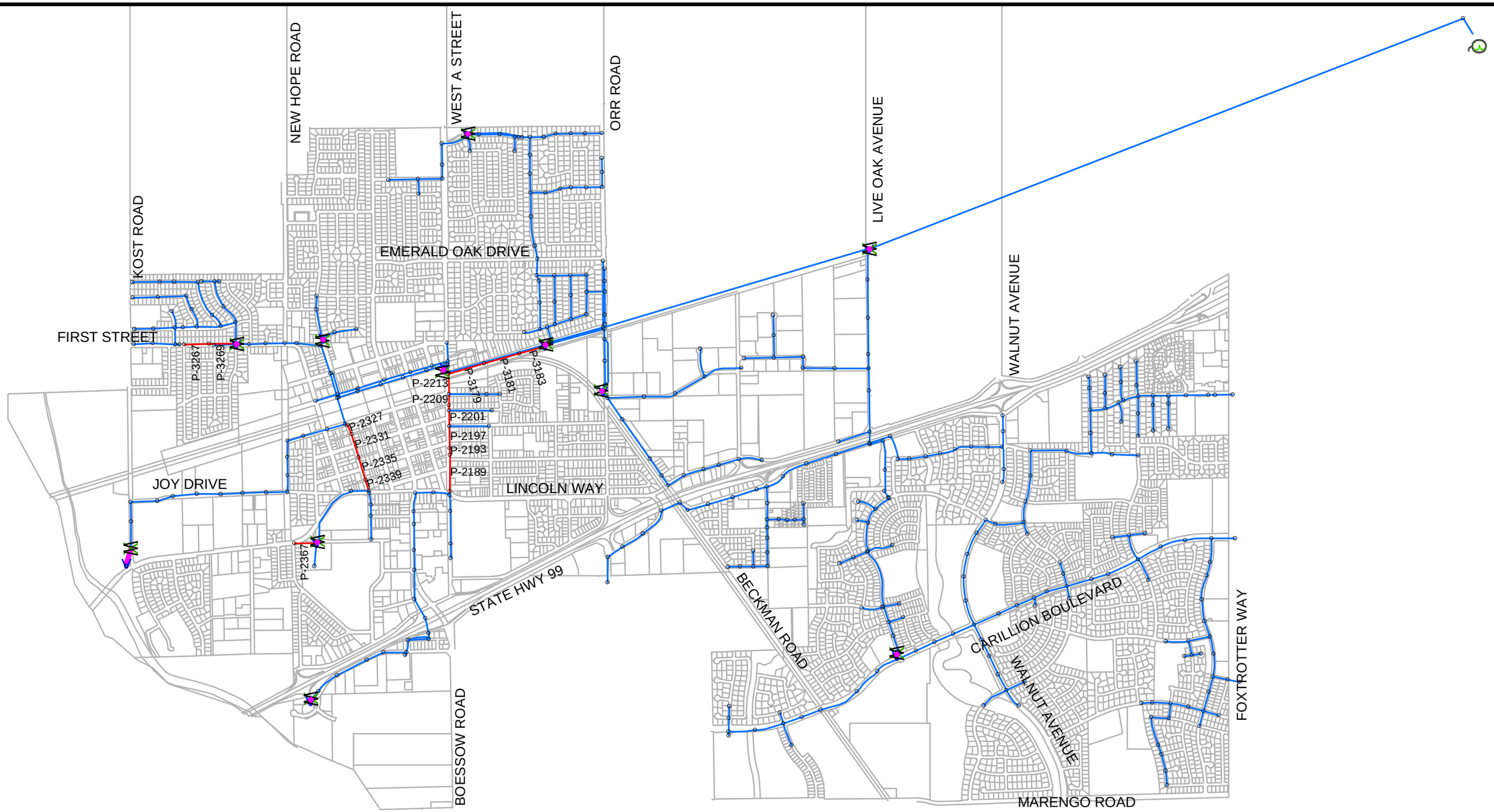
CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

FUTURE PWWF SCENARIO

BEC PROJECT NO.
SC-G21-110-10

FIGURE
B2



LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

D_OVER_D

- 0.000000 - 0.700000
- 0.700001 - 1.000000

Pipes

SCALE: 1" = 1700'

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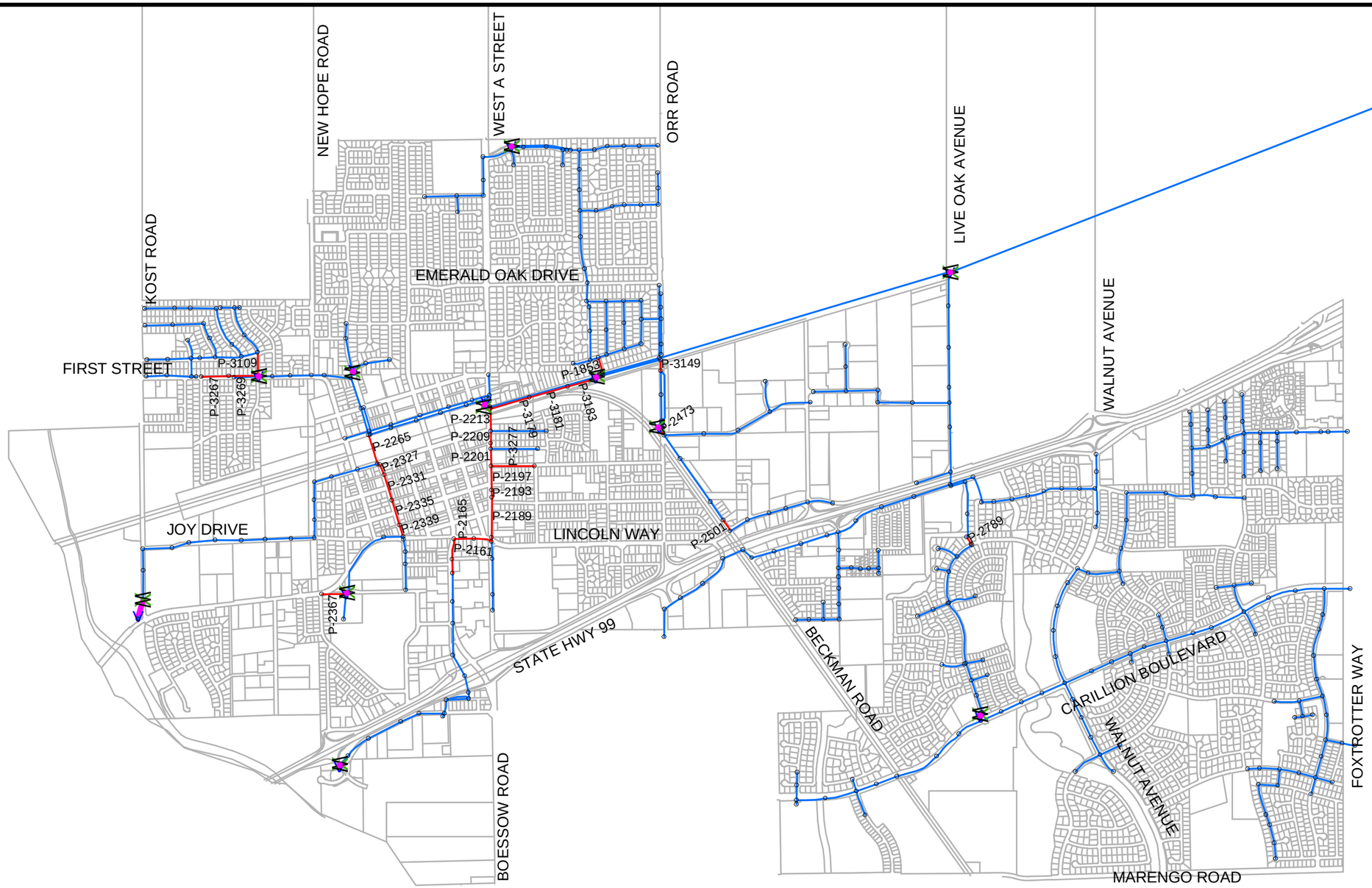
CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

EXISTING PDWF SCENARIO

BEC PROJECT NO. SC-G21-110-10

FIGURE C1



LEGEND

- Manhole
- Chamber
- Outlet
- Wetwell
- Pump
- Streets

D_OVER_D

- 0.000000 - 0.700000
- 0.700001 - 1.000000

Pipes

SCALE: 1" = 1700'

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CITY OF GALT
SEWER SYSTEM CAPACITY ANALYSIS - I

FUTURE PDWF SCENARIO

BEC PROJECT NO.
SC-G21-110-10

FIGURE
C2

LEGEND

SEWER MANHOLE	GRAVITY SEWER PIPE	STREET/PARCEL	MH-100	SEWER MANHOLE NUMBER
SEWER PRESSURE JUNCTION	FORCE MAIN SEWER PIPE	SHEET MATCH LINE	P-001	SEWER PIPE NUMBER
SEWER PUMP STATION	CITY BOUNDARY	PMP-000 PUMP NUMBER	WW-010	WET WELL NUMBER

SCALE: 1"=500'

PLATE 1A
PLATE 1B
PLATE 1C
KEY MAP

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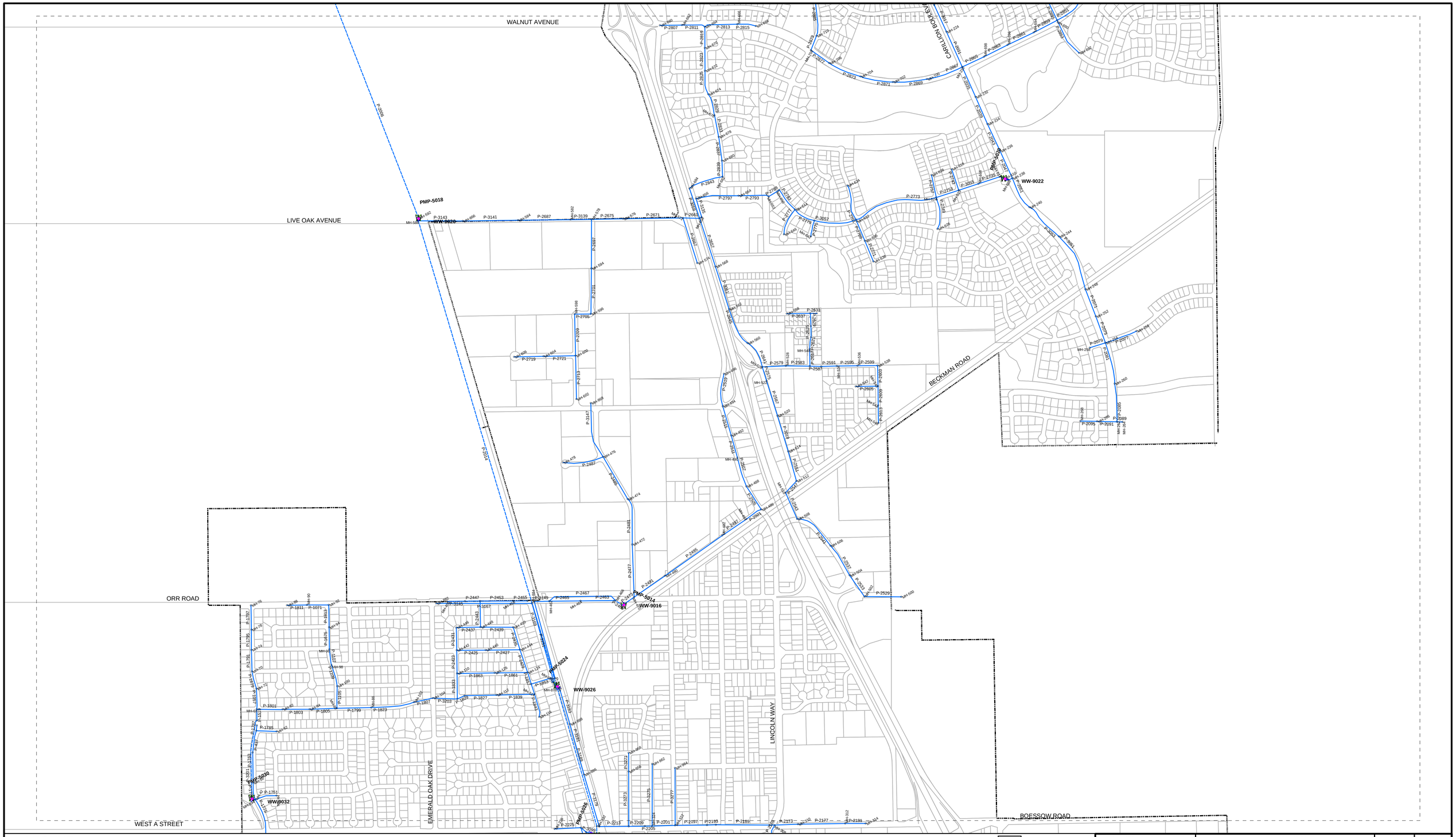
CITY OF GALT
SS SYSTEM CAPACITY ANALYSIS-PHASE I

SEWER SYSTEM MODEL

BEC PROJECT NO. SC-G21-110-10

PLATE 1A



LEGEND

- SEWER MANHOLE
- SEWER PRESSURE JUNCTION
- SEWER PUMP STATION
- GRAVITY SEWER PIPE
- FORCE MAIN SEWER PIPE
- CITY BOUNDARY
- STREET/PARCEL
- SHEET MATCH LINE
- PMP-000 PUMP NUMBER
- MH-100 SEWER MANHOLE NUMBER
- P-001 SEWER PIPE NUMBER
- WW-010 WET WELL NUMBER

SCALE: 1"=500'

KEY MAP: PLATE 1A, PLATE 1B, PLATE 1C

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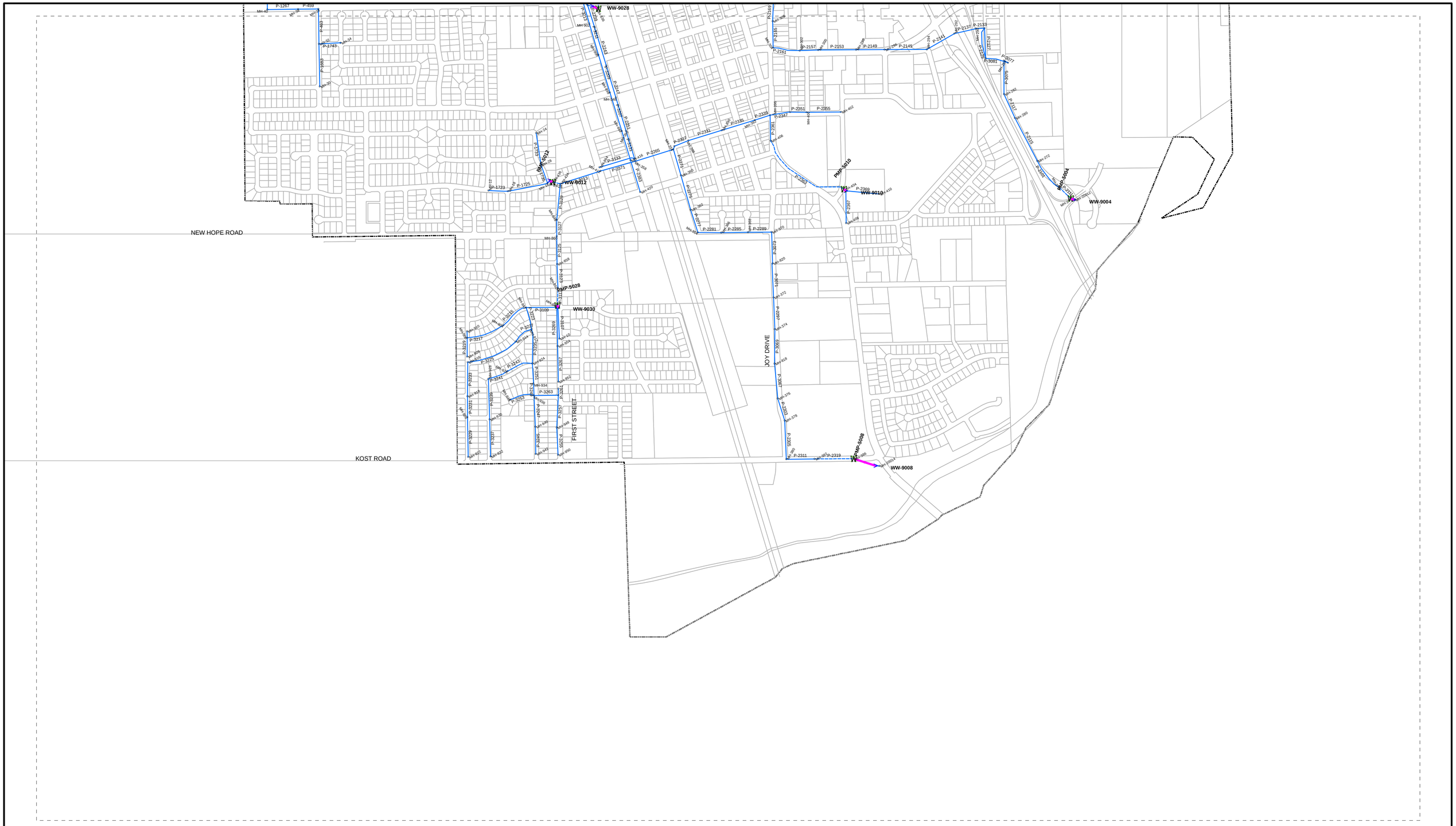
SEWER SYSTEM MODEL

BEC PROJECT NO. SC-G21-110-10

PLATE 1B



LEGEND

○	SEWER MANHOLE		GRAVITY SEWER PIPE		STREET/PARCEL	MH-100	SEWER MANHOLE NUMBER
○	SEWER PRESSURE JUNCTION		FORCE MAIN SEWER PIPE		SHEET MATCH LINE	P-001	SEWER PIPE NUMBER
≥	SEWER PUMP STATION		CITY BOUNDARY	PMP-000	PUMP NUMBER	WW-010	WET WELL NUMBER

SCALE: 1"=500'

PLATE 1A
PLATE 1B
PLATE 1C
KEY MAP

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CITY OF GALT
SS SYSTEM CAPACITY ANALYSIS-PHASE I

SEWER SYSTEM MODEL

BEC PROJECT NO.
SC-G21-110-10

PLATE
1C

Table 4:

Estimated Gravity Flow in Million Gallon per Day to Each Lift Station - Existing Conditions

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor ($PF = 3.5 - 1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
First Street	PQ (New Hope Preschool)	7	2	6		0.01	1.7	0.02	0.01	0.04
	R1C	105	461		0.11		1.6	0.17	0.17	0.34
	R2	5	30		0.01		1.8	0.01	0.01	0.02
TOTALS:		117	493		0.11	0.01		0.21	0.19	0.39
E Street	R1B	4	8		0.002		2.2	0.004	0.01	0.01
	R1C	106	396		0.092		1.9	0.17	0.17	0.34
	R2	1	4		0.001		2.2	0.002	0.001	0.003
	R3	3	100		0.023		2.0	0.05	0.01	0.05
TOTALS:		114	508		0.12			0.23	0.18	0.41
Sparrow	R1C	124	531		0.12		1.9	0.23	0.20	0.43
	PQ (Greer Middle School)	16	2	6		0.06	1.9	0.12	0.02	0.15
TOTALS:		139	533		0.12	0.06		0.35	0.22	0.57
Kost	R1C	33	131		0.03		2.0	0.06	0.05	0.11
TOTALS:		33	131		0.03	0.00		0.06	0.05	0.11
Crystal	HC	17	2	6		0.03	2.0	0.06	0.03	0.09
	LM	4	1	6		0.01	2.1	0.02	0.01	0.02
TOTALS:		21	3			0.04		0.08	0.03	0.11
Chabolla	R2	32	119		0.03		2.0	0.06	0.05	0.11
	R3	2	12		0.003		2.2	0.01	0.003	0.01

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor ($PF = 3.5 - 1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
	PQ	37	3	6		0.07	1.9	0.13	0.06	0.19
	HC	7	9	6		0.01	2.0	0.03	0.01	0.04
TOTALS:		77	143		0.03	0.08		0.22	0.12	0.34
"A" Street	R1C	22	80		0.019		2.0	0.04	0.04	0.07
	R2	306	400		0.093		1.9	0.18	0.49	0.67
	R3	18	100		0.023		2.0	0.05	0.03	0.07
	R1A	26	16		0.004		2.1	0.01	0.04	0.05
	C	1	4	6		0.001	2.2	0.00	0.00	0.00
	NC	6	2	6		0.01	2.1	0.02	0.01	0.03
	PQ (Fairsite, Valley Oaks)	63	11	6		0.21	1.8	0.38	0.10	0.48
TOTALS:		442	613		0.14	0.22		0.67	0.71	1.38
Elm-Amador	R3	41	281		0.07		1.9	0.126	0.05	0.17
	LM	98	17	6		0.18	1.8	0.34	0.12	0.46
	C	12	18	6		0.02	2.0	0.04	0.01	0.06
	PQ	8	3	6		0.01	2.0	0.03	0.01	0.04
	HC	10	10	6		0.02	2.0	0.04	0.01	0.05
TOTALS:		168	329		0.065	0.238	10	0.57	0.20	0.78
McFarland	R1A	41	123		0.03		2.0	0.06	0.05	0.11
	R1B	67	300		0.07		1.9	0.13	0.08	0.21
	R1C	161	625		0.15		1.9	0.27	0.19	0.46
	R2	60	350		0.08		1.9	0.16	0.07	0.23
McFarland	C	20	19	6		0.04	2.0	0.07	0.02	0.10
	HC	82	20	6		0.15	1.9	0.29	0.10	0.38
	M	7	2	6		0.01	2.1	0.03	0.01	0.03
	LM	56	7	6		0.10	1.9	0.20	0.07	0.26

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor ($PF = 3.5 - 1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
McFarland	R3	62	360		0.08		1.9	0.16	0.075	0.23
	PQ (Galt Joint Union High School)	42	11	6		0.16	1.9	0.294	0.0504	0.344
TOTALS:		598	1817		0.41	0.4651		1.65	0.72	2.37
Live Oak	R1A	0.42	1		0.0002		2.3	0.001	0.001	0.001
	R2	54	353		0.08		1.9	0.16	0.06	0.22
	LM	131	13	6		0.024	2.0	0.05	0.16	0.21
	HC	22	16	6		0.03	2.0	0.06	0.03	0.09
	PQ (River Oaks)	19	5	6		0.03	2.0	0.07	0.02	0.09
TOTALS:		226	388		0.08	0.09		0.33	0.27	0.60
Vintage Oak	R1B	116	506		0.12		1.9	0.22	0.14	0.36
	R1C	297	1188		0.28		1.8	0.50	0.36	0.86
	R2	80	403		0.09		1.9	0.18	0.10	0.27
	HC	36	15	6		0.07	1.9	0.13	0.04	0.17
	LM	76	1	6		0.14	1.9	0.26	0.09	0.35
	C	27	1	6		0.05	2.0	0.10	0.03	0.13
	OP	6	1	6		0.01	2.1	0.02	0.01	0.03
	PQ (McCaffrey, Marengo)	85	12	6		0.24	1.8	0.44	0.10	0.55
TOTALS:		723	2127		0.49	0.51		1.85	0.87	2.72

Table 5:

Estimated Gravity Flow in Million Gallon per Day to Each Lift Station Future Including Buildout

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor ($PF = 3.5 - 1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
First Street	R1A-PD[1]	3	1			0.004	1.8	0.01	0.004	0.01
	PQ (New Hope Preschool)	7	2	6		0.01	1.7	0.02	0.01	0.04
	R1C	105	461		0.11		1.6	0.17	0.17	0.34
	R2	5	30		0.01		1.8	0.01	0.01	0.02
TOTALS:		121	494		0.11	0.02		0.21	0.19	0.41
E Street	R1B	4	8		0.002		2.2	0.004	0.01	0.01
	R1C	106	396		0.092		1.9	0.17	0.17	0.34
	R2	1	4		0.001		2.2	0.002	0.001	0.003
	R3	3	100		0.023		2.0	0.05	0.01	0.05
TOTALS:		114	508		0.12			0.23	0.18	0.41
Sparrow	R1C	124	531		0.12		1.9	0.23	0.20	0.43
	PQ (Greer Middle School)	16	2	6		0.06	1.9	0.12	0.02	0.15
TOTALS:		139	533		0.12	0.06		0.35	0.22	0.57
Kost	R1C	33	131		0.03		2.0	0.06	0.05	0.11
	R1A-PD[1]	27	1			0.04	2.0	0.07	0.03	0.10
TOTALS:		60	132		0.03	0.04		0.13	0.09	0.22
Crystal	HC	17	2	6		0.03	2.0	0.06	0.03	0.09
	LM	4	1	6		0.01	2.1	0.02	0.01	0.02
TOTALS:		21	3			0.04		0.08	0.03	0.11

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor ($PF = 3.5 - 1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
Chabolla	R2	32	119		0.03		2.0	0.06	0.05	0.11
	R3	2	12		0.003		2.2	0.01	0.003	0.01
	PQ	37	3	6		0.07	1.9	0.13	0.06	0.19
	HC	7	9	6		0.01	2.0	0.03	0.01	0.04
TOTALS:		77	143		0.03	0.08		0.22	0.12	0.34
"A" Street	R1C	22	80		0.019		2.0	0.04	0.04	0.07
	R2	306	400		0.093		1.9	0.18	0.49	0.67
	R3	18	100		0.023		2.0	0.05	0.03	0.07
	R1A	26	16		0.004		2.1	0.01	0.04	0.05
	C	1	4	6		0.001	2.2	0.00	0.00	0.00
	R1A-PD(1)	165	6			0.22	1.8	0.41	0.20	0.61
	R1B-PD(2)	11	1			0.02	2.0	0.04	0.01	0.05
	NC	6	2	6		0.01	2.1	0.02	0.01	0.03
	PQ (Fairsite, Valley Oaks)	63	11	6		0.21	1.8	0.38	0.10	0.48
TOTALS:		618	620		0.14	0.46		1.12	0.92	2.04
Elm-Amador	R3	41	281		0.07		1.9	0.126	0.05	0.17
	LM	98	17	6		0.18	1.8	0.34	0.12	0.46
	C	12	18	6		0.02	2.0	0.04	0.01	0.06
	PQ	8	3	6		0.01	2.0	0.03	0.01	0.04
	HC	10	10	6		0.02	2.0	0.04	0.01	0.05
TOTALS:		168	329		0.065	0.238	10	0.57	0.20	0.78
	R1A	41	123		0.03		2.0	0.06	0.05	0.11
	R1B	67	300		0.07		1.9	0.13	0.08	0.21

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor (PF = 3.5 - $1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
McFarland	R1C	161	625		0.15		1.9	0.27	0.19	0.46
	R2	60	350		0.08		1.9	0.16	0.07	0.23
	C	20	19	6		0.04	2.0	0.07	0.02	0.10
	HC	82	20	6		0.15	1.9	0.29	0.10	0.38
	M	7	2	6		0.01	2.1	0.03	0.01	0.03
	LM	56	7	6		0.10	1.9	0.20	0.07	0.26
	R3	62	360		0.08		1.9	0.16	0.075	0.23
	PQ (Galt Joint Union High School)	42	11	6		0.16	1.9	0.294	0.0504	0.344
TOTALS:		598	1817		0.41	0.4651		1.65	0.72	2.37
Live Oak	R1A	0.42	1		0.0002		2.3	0.001	0.001	0.001
	R2	54	353		0.08		1.9	0.16	0.06	0.22
	R1A-PD(1)	53	11			0.07	1.9	0.14	0.06	0.20
	R1B-PD(2)	7	1			0.01	2.1	0.03	0.01	0.03
	LM	131	13	6		0.024	2.0	0.05	0.16	0.21
	HC	22	16	6		0.03	2.0	0.06	0.03	0.09
	PQ (River Oaks)	19	5	6		0.03	2.0	0.07	0.02	0.09
TOTALS:		286	400		0.08	0.17		0.50	0.34	0.84
Vintage Oak	R1B	116	506		0.12		1.9	0.22	0.14	0.36
	R1C	297	1188		0.28		1.8	0.50	0.36	0.86
	R2	80	403		0.09		1.9	0.18	0.10	0.27
	HC	36	15	6		0.07	1.9	0.13	0.04	0.17
	R1A-PD(1)	134	181			0.18	1.8	0.33	0.16	0.49
	R1B-PD(2)	53	123			0.09	1.9	0.17	0.06	0.24

Lift Station Tributary Area	Designated Zones in Galt General Plan	Approx. Area (acres)	Existing Number of Units in Zone	SCIS Multiplier, direct or adjusted (ESDs/acre)	Average Dry Weather Flow (ADWF, mgd) Based on Number of Units Counted (232 gpd/unit)	Average Dry Weather Flow (ADWF, mgd) Based on Acreage (310 gpd/ESD)	Peaking Factor ($PF = 3.5 - 1.8Q_a^{0.05}$)	Peak Dry Weather Flow (PDWF), ADWF x PF, mgd	I/I (mgd) Based on 1,200 or 1,600 gpd/acre	Peak Wet Weather Flow (PWWF), PDWF + I/I, mgd
Vintage Oak	LM	76	1	6		0.14	1.9	0.26	0.09	0.35
	R3-PD[3]	63	237			0.35	1.8	0.63	0.08	0.71
	C	27	1	6		0.05	2.0	0.10	0.03	0.13
	OP	6	1	6		0.01	2.1	0.02	0.01	0.03
	PQ-PD	21	3	6		0.04	2.0	0.08	0.03	0.10
	PQ (McCaffrey, Marengo)	85	12	6		0.24	1.8	0.44	0.10	0.55
TOTALS:		994	2671		0.49	1.17		3.07	1.19	4.26

(1) R1A-PD 0.23 acres - Based on Galt Code Table 18.20-1 Development Standards for Minimum Lot Size

(2) R1B-PD 0.18 acres - Based on Galt Code Table 18.20-1 Development Standards for Minimum Lot Size

(3) R3-PD 18 units/acres - Based on Galt Code Table 18.20-1 Development Standards for Minimum Lot Size