BAEN 480 Final Report

The City of Jonestown: Wastewater Master Plan

By: Aaron Dunavant, Cassian LaDue, Kayla Rejcek, Andrea Saavedra,

Natalie Luhutsky, Thomas Marchetti, Amy Molina, & Niao Yan

April 27, 2015

Sponsor: Frank Phelan
Email: fphelan@jaeco.net

The City of Jonestown: Wastewater Master Plan

1. Executive Summary

The City of Jonestown is committed to responsible planned development, economic vitality, public service improvements, continued park expansions, and overall improved quality of life for its residents. Developing and maintaining a city-wide wastewater collection system is an important step in creating a clean, safe environment for the public, especially as the city looks forward to a blooming commercial and residential sectors.

The main design objectives were to develop city population projections, a vulnerability assessment map, a wastewater collection system design with manageable phases, an effluent disposal plan, and a cost estimate. The final wastewater collection system design consisted of gravity lines, force mains, low pressure system (LPS), lift stations, wastewater treatment plants (WWTPs), and drip irrigation fields.

The design includes three WWTPs, located based on topography, existing infrastructure, and regulations from the TCEQ, LCRA, and Jonestown City Ordinances. Two plant designs have adjoining drip irrigation fields for effluent disposal. Due to the suburban development surrounding the southernmost WWTP, an effluent reuse field for this plant was deemed to be out of the scope of this project. Construction projects should take the topography and soils in the areas into extreme consideration during the planning process of such projects. According to the data included in the vulnerability maps, the entire city of Jonestown is highly susceptible to contamination and is not an ideal candidate for construction of any type. The vulnerability maps show few decent locations for WWTP or wastewater pipe layouts.

The majority of pipes used were gravity pipes; however, due to topography constraints, lift stations and force mains were required to overcome this dilemma. Due to the highly developed nature of downtown, an LPS was selected for the main downtown area of the Commercial Corridor. Compared to gravity lines, LPS pipes are much smaller, require minimal ground intrusion, do not rely on gravity, and are a cheaper alternative due top minimal wastewater generation in this area.

In order to make the overall construction of the wastewater collection system more manageable, it was divided into several phases. These phases represent incremental development of the City of Jonestown over the next 10 to 20 years. First, wastewater collection service will be provided to the northern half of the commercial corridor through construction of wastewater lines, a chain of lift stations, and the LPS in the downtown area. This area will be serviced by an existing WWTP in the city of Leander. Second, the southern half of the corridor will be serviced through the construction of wastewater lines, lift stations, and a new WWTP. Additional residential developments throughout Jonestown will be serviced through the construction of further wastewater collection networks and two additional WWTPs.

In order to give the City of Jonestown an initial idea of the expenses of the overall project of each phase, a cost estimate was developed. It did not include contingency costs or soft costs, but only the costs of WWTPs, drip irrigation fields, lift stations, gravity lines, force mains, and the LPS. The total of estimated costs are just under $16 million, with the two phases in the corridor totaling $3.5 million. This initial estimate should give the City of Jonestown material from which to continue conversations on funding options and project feasibility.

2. Table of Contents

1. Executive Summary ii
2. Table of Contents iii
3. Table of Figures iv
4. Table of Tables v
5. Introduction 1
   5.1. Project Background 1
   5.2. Scope 1
6. Design Objectives 2
   6.1. WWTP and Collection Network Layout 2
   6.2. Vulnerability Map 2
   6.3. WWTP and Collection Network Sizing 2
   6.4. Population Projections 3
   6.5. Land Use Mapping 3
   6.6. Effluent Discharge and Reuse System Design 3
   6.7. Economic Impact Assessment 3
7. Design Constraints 3
   7.1. Funding Sources 3
   7.2. Topography 3
   7.3. Floodplain 4
   7.4. Water Wells 4
   7.5. Existing Infrastructure 4
   7.6. Commercial versus Residential Land Uses 4
8. Background Research and Literature Review 4
   8.1. Population Projections 4
      8.1.1. Census Data 4
      8.1.2. Cohort Component Method 5
   8.2. Vulnerability Mapping 5
   8.3. Collection System Design 6
      8.3.1. Conventional Gravity Sewers 6
      8.3.2. Force Main Sewer and Lift Stations 6
      8.3.3. Low-Pressure Systems 7
   8.4. Effluent Reuse Network Design 7
   8.5. Previously Proposed Solutions 8
9. Design 9
   9.1. Alternatives Considered 9
      9.1.1. Primary and Secondary Study Areas 9
      9.1.2. Short-Term Study Horizon 10
      9.1.3. Long-Term Study Horizon 10
   9.2. Analysis of Alternatives 10
      9.2.1. Study Boundary Definition 10
      9.2.2. Population Projections 11
         9.2.2.1. Conservative Population Projection 11
         9.2.2.2. Moderate Population Projection 12

9.2.2.3. Aggressive Population Projection....14
9.2.3. Wastewater Capacity Calculations....16
9.2.3.1. Relevant Assumptions....16
9.2.3.2. Developable Land Estimates....16
9.2.3.3. Capacity Calculations....16
9.2.4. Design Spreadsheets....16
9.2.4.1. The Wastewater Capacity Modeler Spreadsheet....17
9.2.4.2. The Lift Station Design Spreadsheet....19
9.2.4.3. Force Mains Spreadsheet....19
9.2.5. Vulnerability Mapping....20
9.2.5.1. DRASTIC Methodology....20
9.2.5.2. DRASTIC Data Collection and Assumptions....20
9.2.5.3. DRASTIC Vulnerability Calculations....21
10. Final Recommendation....22
10.1 WWTP Locations and Sizing....22
10.1.1. WWTP1 Location....22
10.1.2. WWTP2 Location....23
10.1.3. WWTP3 Location....24
10.1.4. WWTP Sizes and Specifications....25
10.2. Effluent Discharge....25
10.2.1. Drip-field Locations....25
10.2.2. Re-use Plan....26
10.3. Collection Network Layout....26
10.3.1. Pipes....26
10.3.2. Lift Stations....31
10.3.3. Phasing Plan....32
10.3.4. Cost Estimate....33
11. References....37
12. Appendices....39
A. Vulnerability Mapping....39
B. Design Spreadsheets....46

3. Table of Figures

Figure 1: Previous design solution rejected by city....9
Figure 2: Map of Study Areas....10
Figure 3: Conservative model for population projections....12
Figure 4: Moderate model for population projections....14
Figure 5: Aggressive model for population projections....15
Figure 6: Projection comparison for population in all study areas....15
Figure 7: Location of WWTP 1 and Drip Irrigation Field in Study Area C....23
Figure 8: Location of WWTP 2 and Drip Irrigation Field in Study Area G....24
Figure 9: Location of WWTP 3 in Study Area B....24
Figure 10: Pipe layout for the commercial corridor....27
Figure 11: Pipe layout for the Study Areas C, D, E, and F....27
Figure 12: Pipe layout for Study Area H....28
Figure 13: Pipe layout for Study Area B....28

Figure 14: Pipe layout for Study Area G 29
Figure 15: Overall map of wastewater collection system 33

4. Table of Tables

Table 1: Secondary Treatment Single-Grab 1 7
Table 2: Census data projections for a conservative model 12
Table 3: Study Area Map areas used to estimate moderate population 13
Table 4: Population projection data fro a moderate scenario 13
Table 5: Study Area Map areas used to estimate aggressive population 14
Table 6: Population projetion data for an aggressive scenario 14
Table 7:Vulnerability Mapping table 21
Table 8: General DRASTIC scale and revalued scale used for Jonestown 22
Table 9: WWTP Capacity for ultimate development 25
Table 10: Final Pipe Sizes 29
Table 11: Lift station pump and wet well specifications. 31
Table 12: Total cost estimates for each study area 33
Table 13: Cost estimates broken down for each study area 34

5. Introduction

5.1. Project Background

Public health is a major concern across the globe. Developing and maintaining a city-wide wastewater collection system is an important step in creating a clean and safe environment for the public. A centralized wastewater collection system takes the contaminated sewage water and transports it to a city wastewater treatment plant for cleaning and water recycling. This process allows cities to take control of wastewater treatment and move from a septic system to a city-wide system. In addition, the reclaimed water can be reused as a non-potable water source for various purposes throughout a city and in homes.

Septic systems work well for less populated areas in which a large sewer system would not be feasible. In larger cities with a commercial sector, a sewer system is a necessity to keep up with increased wastewater generation. Such is the case for Jonestown, Texas. Located just north-west of Austin in Travis County, Jonestown is currently at a population of approximately 2000 and growing, with the surrounding area experiencing significant growth and an influx of homeowners. The land use is mainly residential with a commercial corridor running along a major road through town. The majority of Jonestown is currently serviced by septic systems, and the city has expressed a desire to move away from individual septic wastewater disposal towards a city-wide sewer system. Located in Texas hill country, Jonestown has variable topography that has made the design and implementation of a centralized system difficult; however, the city-wide collection and treatment system would increase town revenue, accessibility and growth.

This project is sponsored in part by the Jonestown City Council with Marilee Pfannstiel as the Community Development Director of Jonestown, and in part by Frank Phelan, P.E. from Jay Engineering Company. Mr. Phelan was commissioned by the city to design their public sewer system and is the engineering lead for the students in the Biological and Agricultural Engineering department at Texas A&M University working in conjunction with Mr. Phelan.

5.2. Scope (Basic Objectives, Design Study Areas/Horizons)

Objectives:

1. Develop environmental vulnerability and risk assessment maps for the study areas.
2. Design a wastewater collection network for different population projections.
3. Size and locate wastewater treatment plants.
4. Create an effluent disposal or re-use plan.

This report provides the following deliverables:

1. Population Projections - includes:
    - a. 10 year population estimate for study areas.
    - b. Ultimate population estimate corresponding to fully developed condition of study areas.
2. Sewer Pipe Design as CAD drawings - includes:
    - a. Phase A and Phase B implementations for the commercial corridor corresponding to 10 year and ultimate time horizons that plan for future land and population development.
    - b. Piping layout, pipe diameters and lengths sizing, and capacity of flow.

3. Wastewater treatment plant (WWTP) suggested locations as CAD drawings, influent flow rates, and capacity sizing.
4. Vulnerability Mapping as GIS maps of all study areas for risk assessment towards the environment.
5. Suggested locations as CAD drawings for treated effluent irrigation reuse area.
6. Estimated design cost as Excel spreadsheet – includes sewage system installation and material costs.

This wastewater collection and treatment system will help the city of Jonestown develop and accommodate for growth pressures from Austin and the surrounding cities. This report outlines the design and design strategies utilized by the students at Texas A&M University.

6. Design Objectives

6.1. WWTP and Collection Network Layout

The objective of the collection network layout is to provide wastewater service to all areas within the study boundaries. The layout must accommodate for future areas of development that will be able to connect to the wastewater sewage main. Additional sub-main piping will be necessary for these connections, but sub-mains are out of the scope of this project. They are assumed to be accommodated for by the future land planner.

6.2. Vulnerability Mapping

The objective of the vulnerability mapping is to assess the risk against the environment involved with construction of the piping layout. Environmental hazard could result from a break in the system, which would allow untreated wastewater to flow into the soil.

6.3. WWTP and Collection Network Sizing

The objective of the collection network and wastewater treatment plant sizing is to ensure that current and future wastewater services are being provided to the residents of Jonestown. The WWTPs and adjoining piping must be sized according to the amount of wastewater that will be generated. The unit used for the capacity calculations was the LUE. An LUE, or living unit equivalent, is a unit of measurement used to define the typical wastewater flow produced by a single family residence.

6.4. Population Projections

The projected number of people living in the Jonestown area will be determined utilizing population growth history from Travis County and Jonestown. Historically, Jonestown has seen less growth than the surrounding areas of Travis County, but the projections for this project will assumed that it will grow at the same aggressive rate as the rest of Travis County, as requested by the Jonestown City Council. By sizing the wastewater system for this aggressive growth rate, Jonestown will be able to accommodate all projected upcoming growth. The rates collected will also be compared to traditional urban planning methods, in order to compare effectiveness and accuracy.

6.5. Land Use Mapping

The land use mapping is a requirement for developing the final system design. Two areas will be zoned, residential and commercial, based on the current existing development and city zoning maps. This is done in order to calculate the amount of wastewater that will drain from each area, as a residential area drains less than a commercial area. Land use mapping will aid in final design as it will enable pipe layout to be more accurate and efficient.

6.6. Effluent Discharge and Reuse System Design

After treatment in the WWTP, the effluent must be disposed of appropriately. Due to regulations forbidding the disposal of effluent in Lake Travis or any adjoining streams, a dispersal field for each WWTP will be utilized for this disposal. Drip fields were chosen for this project since they are more versatile in application and allow for more alternate location possibilities. The effluent will be pumped from the WWTP to the appropriate drip field with force mains, and a drip field line will be used to dispose of the effluent into the field.

6.7. Economic Impact Assessment

In order to assess the feasibility of the overall project and of each of the phases of the project, a cost estimate needed to be developed. This would allow the City of Jonestown to decide exactly how it would like to phase its development plan. The City of Jonestown would be able to evaluate its financial resources and explore proper funding options to see how it would approach the problem of implementing a wastewater collection network.

7. Design Constraints

7.1. Funding Sources

One of the most fundamental constraints for the construction of this project is whether or not it can be financially supported. According to Ron Wilde (City of Jonestown, City of Jonestown public meeting, 25 March 2015), several funding options exist and deserve exploration, including city financing, tax increment financing, state grants, developer contributions, and municipal utility districts. It is likely that this project will need to be funded by a combination of several of these options. Much coordination and correspondence will need to be completed to secure sufficient funding.

7.2. Topography

A major factor in the site selection of the three wastewater treatment plants was topography. The plant sites needed to be areas of land within the study boundaries that had relatively low slopes so that the building plots could be as flat as possible. According to the city ordinances, “development is prohibited on a slope with a gradient that exceeds thirty-five percent,” so areas with a greater slope than this were excluded from consideration (City of Jonestown, 2014).

The elevation of the plants relative to the rest of the study areas was taken into account. The most ideal location for a plant is at the lowest elevation of the service area to allow for gravity flow pipes leading from all reaches of the study areas directly to the plant. Where this was not possible, force mains were designed from the gravity flow collection points at the lower elevations back up to the treatment plants using lift stations. This was in accordance with the city ordinance which states: “All new public wastewater systems shall be designed and constructed to operate on a gravity flow basis by taking advantage of natural topographic conditions and thereby reducing the need for lift stations and force mains" (City of Jonestown, 2014).

7.3. Floodplain

To reduce costs of building the plants, the possible plant locations were chosen away from the 100-year floodplain that is located throughout Jonestown. In order to build a plant within the floodplain, it must be "protected from inundation and damage that may occur during that flood event" (TCEQ, 2009). These special requirements would add extra construction costs to the plants, and so areas within the floodplain were excluded from consideration.

7.4. Water Wells

A plant location must be at least 500 feet away from a public water well or 250 feet away from a private water well to avoid contamination of these drinking water sources (TCEQ, 2009). Therefore, all recorded wells within the area were located and accounted for using the TCEQ Water Well Report Viewer online.

7.5. Existing Infrastructure

Siting of the wastewater treatment plants took into account the existing infrastructure of the study areas. The plants could not be located on plots of land with existing buildings. Google Maps was used to locate and account for all existing infrastructure.

7.6. Commercial versus Residential Land Uses

The capacity of the sewer system must be able to handle any future commercial and residential development. The City Zoning map designated many of the study areas as either residential or commercial for future development, and any areas that were not included on the zoning map were assumed to be future residential zoning. The capacity requirements of the sewer system must be able to handle the varying wastewater loads that are generated from commercial businesses versus residential locations, since commercial businesses generally generate more wastewater than typical residential homes.

8. Background Research and Literature Review

8.1. Population Projections

8.1.1 Census Data

The United Nations defines a population census as the total process of collection, compiling, and publishing demographic, economic, and social data pertaining to a specific time to all persons in a country or delimited part of a country. There are four essential features of census data (Lacey, 2015):

1) Each individual is enumerated separately; the characteristics of each person within the household are recorded separately,

2) The census covers a precisely defined territory and includes every person present or residing within its scope. The housing census should include every type of building and living quarters,

3) Each person and each type of building and living quarters is enumerated with respect to a well-defined point of time,

4) The census is taken at regular defined intervals, usually every 10 years.
Population projections are important for Jonestown because it helps planners, such as the local governor, stockholders, and landowners evaluate and revise the local development plans. These developments then lead to further examination of needed utilities.

However, uncertainties and errors could happen due to shortcomings in data collection, such as not accounting the homeless population, residents who provide incorrect information, etc. Therefore, probability of errors is introduced in the Census Data.

8.1.2 Cohort Component Method

Population projections are estimates of the population for future dates. They are usually based on the most recent census and are produced using the cohort component method. This method uses components of demographic change, such as age, ethnicity, and sex, to predict population growth (Lacey, 2015). The main component of population change is based on assumptions about future birth, death and net migration. In some cases, the projections also needed to consider future fertility and life expectancy in the local area. In the cohort component method, these components are assumed to remain constant throughout the projection time. As a forecasting tool, planners sometimes alter the vital statistics to reflect their view. It allows them to accurately project the total size of the population.

There are also disadvantages to this method. The total projection would be highly dependent on the past census data, which mainly takes into consideration the reliable birth, death and migration rates over any other factors. Also, the birthrates and estimates of migration are assumed to remain the same throughout the whole period, which is unlikely to happen in reality. In conclusion, the cohort method is an efficient tool to project the potential growth or decline of a locale by age and sex, but it does not consider any non - demographic factors that could also affect the population, such as disaster, regulation, land uses, etc.

8.2. Vulnerability Mapping

A vulnerability or “risk” map, according to the *Handbook for Vulnerability Mapping,* "gives the precise location of sites where people, the natural environment or property are at risk due to a potentially catastrophic event that could result in death, injury, pollution or other destruction” (Edwards, et al.) A simple interpretation of this definition is how susceptible an area is to contamination. There are several types of methods in developing a vulnerability map. These methods can be tailored to the specifics of each project, which makes the process of map-making as versatile as ever. The “DRASTIC” methodology is primarily used for groundwater contamination.

The DRASTIC method is a simple mathematical computation consisting of data readily available from online sources and previously made maps. The method takes into account: depth to water table, net recharge of aquifer, aquifer media, soil media, and topography, the impact of the vadose media zone, and the hydraulic conductivity of the aquifer. The DRASTIC method can be altered to meet the criteria of your project; each parameter does not need to be used in order to complete the mathematical computation. The flexibility of this model is why it is widely used in vulnerability mapping. A disadvantage to this model is that it can only be used for homogeneous aquifers. An aquifer with two or more media types cannot be evaluated using the DRASTIC model. The DRASTIC model's accuracy may be considered unreliable. There are other models, like the PI-Method, that are more accurate and should be used for high impact and costly projects or operations. The PI-Method requires: extensive knowledge of ArcGIS, difficult mathematical computation, knowledge of fracturing, and lithology (Goldscheider, et al.)

Babiker et al, created a vulnerability map to assess the possibility of contamination in groundwater at Kakamigahara Heights, central Japan. Because many of the aquifers in Japan are used for drinking water purposes, the cleanliness of the aquifer is of upmost importance. They realized the numerous methods in creating a vulnerability map and ultimately decided to utilize the DRASTIC model. Their reasons are the following: “the process-based methods use simulation models to estimate the contaminant migration but they are constrained by data shortage and computational difficulties, statistical methods use statistics to determine associations between spatial variables and actual occurrence of pollutants in the groundwater.” With this approach the “limitations included insufficient water quality observations, data accuracy and careful selection of spatial variables.” Advantages to using the DRASTIC method include rainfall and depth to groundwater, which are readily available for large areas, making the DRASTIC model more suitable for large regional areas (Babiker et al.) Babiker saw the simplicity of using the DRASTIC model and GIS for this project. “The simple definition of its vulnerability index as a linear combination of factors shows the feasibility of the computation using GIS” (Babiker et al.).

8.3. Collection System Design

Wastewater collection systems carry wastewater from residential, commercial or industrial properties to a treatment plant or discharge point. Several methods of conveyance may be combined in a collection system for maximum efficiency. The three methods researched by the Jonestown team are the conventional gravity lines, force mains, and low-pressure systems.

8.3.1. Conventional Gravity Lines

Conventional gravity sewers transport wastewater by gravity along a downward sloping pipe. Gravity lines are designed to drain all the sewage to central lowland. The pipe is designed so that the slope and size are adequate to maintain flow in the direction of the discharge point without pressurizing the pipe. Gravity lines are the most common collection sewers used to collect and transport domestic water due to reliability, durability and consistent minimum velocity. Maintaining a minimum velocity decreases blockages, pipe corrosion and odors. However, gravity sewers can require large-scale excavations in varied terrain to maintain consistent slope.

Gravity sewers are designed to manage sanitary waste in peak conditions. (USEPA, 2002) The pipelines must maintain a minimum pipe size to reduce probability of clogging, and a minimum velocity to ensure self-cleaning of the lines. The standard minimum size and velocity are enforced to reduce maintenance and related costs. In addition, manholes must accompany gravity sewer construction, increasing baseline costs. (USEPA, 2002)

8.3.2. Force Mains and Lift Stations

Force main sewers are pipelines that transport wastewater under pressure from lower to higher elevation. Lift stations contain pumps or compressors to provide the energy for wastewater conveyance. Force mains and accompanying lift stations are installed when gravity line construction will result in deep excavations. Typically, if the gravity lines require trenches deeper than 20 feet, than force mains are more economical.

Force mains can decrease the overall sewer construction cost due smaller pipe size and depth. Installation is also simpler and cheaper due to shallow pipe trenches. (USEPA, 2000) However, the construction cost of lift stations, in addition to force main lines, may be comparable to the cost of gravity lines. Passage of wastewater through lift stations has a higher likelihood of septic discharge. Force mains and lifts stations require frequent maintenance and cleaning to avoid clogging and pipe corrosion. The design of force main lines is accompanied by lift station design and must incorporate friction losses, pressure surges and maintenance. (USEPA, 2000)

8.3.3. Low-Pressure Systems

Low-pressure systems use individual residential pumps to push the wastewater flow to a main pipeline or master lift station. Low-pressure sewer systems are beneficial where conventional sewer methods are not feasible, such as in low population rural areas or steep terrain areas. Low-pressure systems are also efficient when “cluster” pumps may be installed to connect several residences. Small diameter pipes, due to low flow capacity in a low-pressure system decrease construction and repair costs (USEPA, 1998).

8.4. Effluent Reuse Network Design

8.4.1. Effluent Requirements

The requirement found in TAC Chapter 309.1 specifies that the wastewater in the WWTP must be treated by secondary treatment before leaving the plant as effluent. The secondary treatment must treat the water within standards for Biochemical Oxygen Demand, 5-Day (BOD_5), Total Suspended Solids (TSS), Dissolved Oxygen (DO), and pH (TAC Chapter 309.1). These standards are shown in Table 1: Secondary Treatment Standards for Single Grab Samples.

Table 1: Secondary Treatment Single-Grab Standards (TAC Ch. 309.1)

Pollutant	Required Quality
BOD_5	65 mg/l
TSS	65 mg/l
DO	2.0 mg/l
pH	Within 6.0-9.0 standard units

The Lower Colorado River Authority does not have any regulations of the quality of the effluent in addition to what TCEQ stipulates. There are no additional regulatory authorities that have jurisdiction in the area of Jonestown. Only the TCEQ regulations must be considered for the effluent quality requirements.

8.4.2. Effluent Reuse Options

Since the city of Jonestown is within the LCRA boundaries, the LCRA Parks Land and Water Use Regulations Rule 17 dictates that the effluent may not be disposed into Lake Travis or the surrounding streams and rivers. The practice of disposing the effluent into lakes, streams, or rivers is common throughout most of Texas, as the treated effluent is treated to be as clean as or cleaner than the natural water resources.

Without the effluent draining into Lake Travis or elsewhere, a field must be constructed for the effluent disposal. Two such fields can be considered: a drip field or a spray field. A drip field used a track buried underground to distribute the effluent uniformly across the field area. This can be done either continuously or during regulated intervals. The drip field is viable for slopes up to 1% lateral slope (TAC §222.121**)**. A spray field is different than this; it works similarly to a lawn sprinkler system. Drawbacks to this system include that the wastewater must be treated as stringently as possible, due to potential human contact, and that the land utilized for the field cannot reach as great a maximum slope. Drip fields, as opposed to spray fields, are more common throughout most of Texas.

Other reuse options include using the effluent for beautification for city areas like parks and golf courses. This is a common practice, since an additional field will not need to be developed and the water maximizes potential.

8.5. Previously Proposed Solutions

Currently, Jonestown has two separate agreements to provide wastewater service to the city. The first agreement is with the city of Leander on the North East side of the city. Leander will provide 2000 LUE's of water and 1000 LUE's of wastewater service to the city. Under this agreement Jonestown is responsible for all the necessary piping and equipment needed to move the water up to the city limits between the two cities. The agreement has a 15-year limit where the city of Leander has the right to inform the City of Jonestown that it needs to find another source of water and wastewater service. This is done in case Leander becomes unable to provide these services anymore due to fiscal reasons or lack of available water. If Leander decides to terminate these services, Jonestown has 2 years to find an alternative way to receive these services. On the contrary, if Jonestown decides to terminate the contract at any time, then they must give 12 months' notice.

The second agreement is with the city of Lago Vista on the southern edge of Jonestown. The service is for the area called "The Hollows," located southwest of Study Area B (Figure 2) from the study area map. This agreement outlined a phased plan for providing wastewater service to the Hollows: temporary service would be provided through a "pump and haul system" until a permanent effluent line was established. Lago Vista will provide 400 LUE's of wastewater treatment. As with the Leander agreement, the city must provide all the necessary pipes to deliver the wastewater to the point of delivery. These agreements familiarizes the design teams with some wastewater treatment system practices (quality and quantity of wastewater treated in a certain area) and gives an example layout of a wastewater collection pipeline with lift stations.

In the past the city of Jonestown commissioned a different student group to make a wastewater master plan. The study was named "Phase 1" and consisted of 6 waste water treatment plants (WWTPs) along with: a gravity collection line along FM 1431, a slope map of the study area, a map of future development and wastewater flow projections (capacity of gpd per plant is shown on map) and an approximate 100 year FEMA floodplain. The Phase 1 study was not taken any further due to the residents of Jonestown and the City Council not wanting 6 WWTPs to be built. The residents thought 6 WWTPs was "excessive" and made a point of telling the design team that 6 WWTPs was something they did not want to see in the final design. See Figure 1 below for an image of this previous solution.

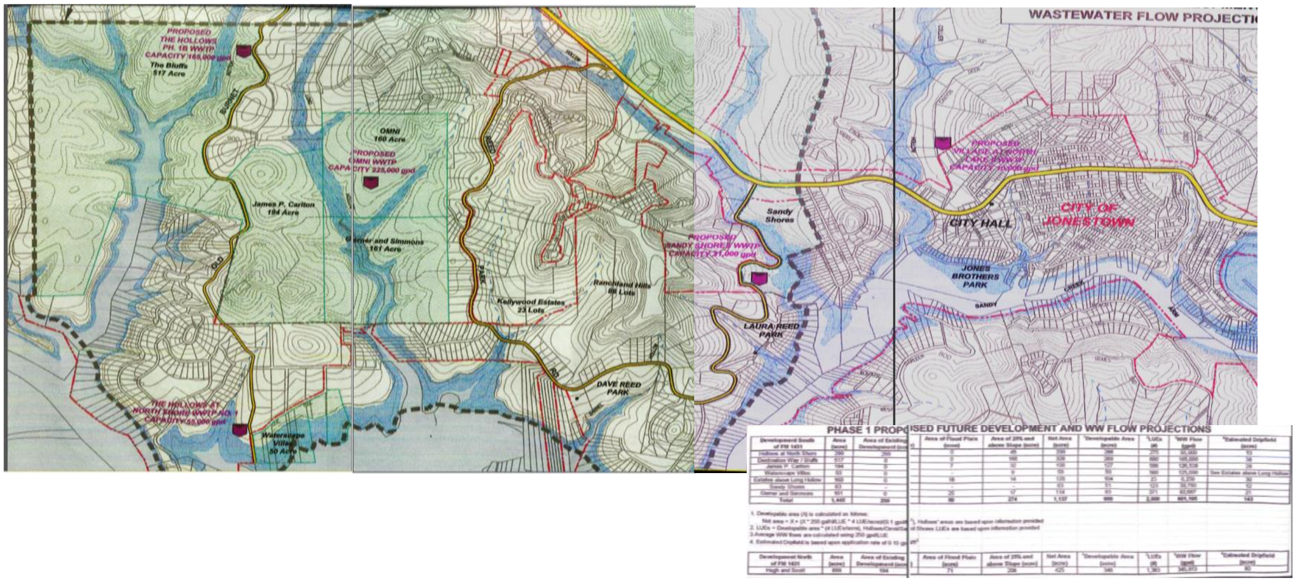


Figure 1. Previous design solution that was rejected by the city due to excessive wastewater treatment facilities.

9. Design

9.1. Alternatives Considered

Factors affecting design alternatives were differing study area priorities (primary and secondary) and study horizons (short-term and long-term).

9.1.1. Primary and Secondary Study Areas

The study areas were divided into primary and secondary priority based on the order of importance placed on future development of the city. This information was collected during the public meetings held in Jonestown, during which the public opinion of the study areas were discussed. Based on a consensus, the commercial corridor (Figure 2, labelled CC) was decided to be the primary study area, so that the city could grow in its commercial sector. A planned subdivision called the “Jonestown 300” was also decided to be a primary study area, since it is expected to achieve an aggressive growth in the near future. This study area is labeled “Study Area C” in the study boundaries map. See Figure 2 below for study area labeling reference.

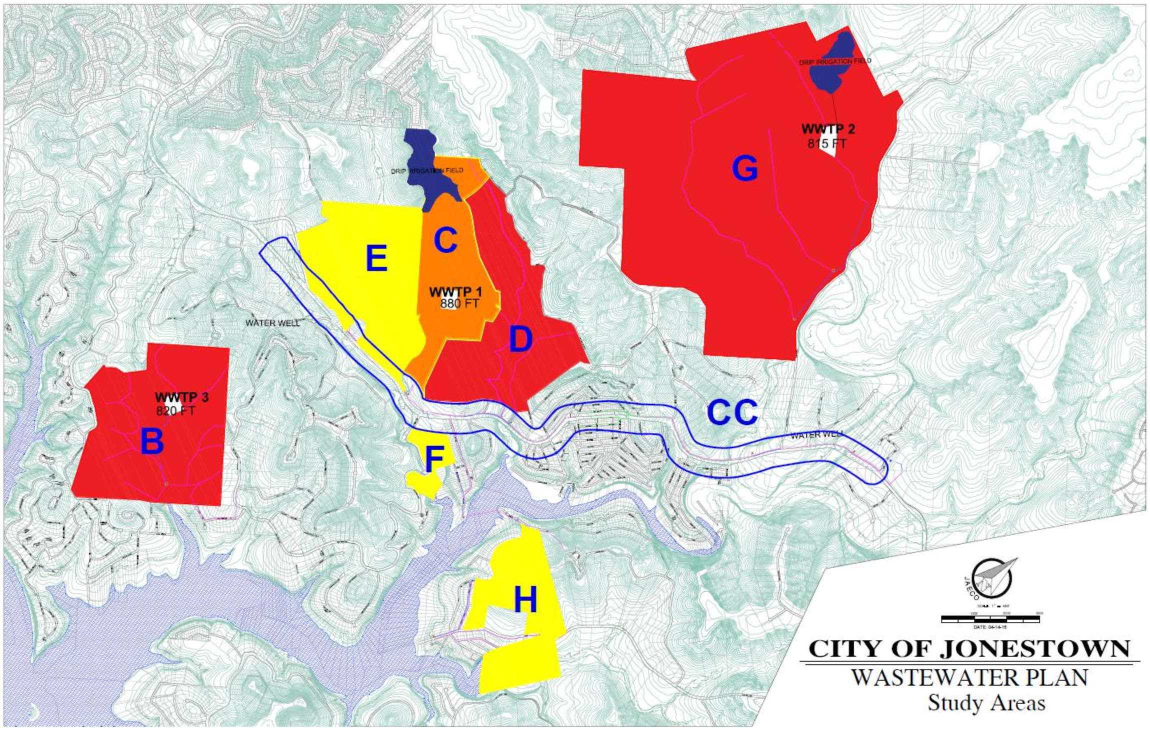


Figure 2. Map of Study Areas

All other study areas (B, D, E, F, G, and H) were classified as secondary study areas due to the assumption that their projected growth will not being as aggressive as in the commercial corridor and Study Area C. These secondary study areas were considered in the final design, but with more generalized piping layouts and cost estimates than the primary study areas.

9.1.2. Short-Term Study Horizon

The short-term study horizon for this project is 10 years. The 10 year horizon is planned to be implemented in Phase A of the phasing plan for the primary study area commercial corridor.

9.1.3. Long-Term Study Horizon

The long-term study horizon is the ultimate horizon. This is the assumed full population and land development within the study boundaries. This portion of the phasing plan is called "Phase B." It was decided that the best option was to base the final design for the secondary study areas and the Jonestown 300 primary study area on the ultimate horizon so that the piping would not have to be upgraded in size as the population surpassed the 10-year projections. This also ensured that the wastewater treatment plants will be equipped to handle the ultimate horizon wastewater capacity flow without having to be redesigned and rebuilt.

9.2 Analysis of Alternatives

9.2.1. Study Boundary Definition

All relevant design parameters, constraints, and population data were considered only for areas within the primary and secondary study areas. Although there is a possibility that in practice, the designed sewer system and corresponding wastewater treatment plants will service areas outside of these set boundaries, the final design is based solely on data relevant to the areas within the study boundaries.

9.2.2. Population Projections

Over the next 20 years, Jonestown is expecting increased growth due to new developments. With these new developments, the city of Jonestown is looking to introduce a wastewater treatment system to transport, treat and dispose of the new waste stream the city is expecting to generate. This paper aims to develop a population projection curve for the city over the next 20 years, to have a better idea of how much wastewater the city will create to size the wastewater system. Conservative, moderate, and aggressive population projections were evaluated to determine the best population projection scenario.

From 1990 to 2010, Jonestown, Texas experienced a growth of just under 600 people based on the census data. Over the next 20 years, Jonestown is expecting increased growth due to new developments. With these new developments, the city of Jonestown is looking to introduce a wastewater treatment system to transport, treat and dispose of the new waste stream the city is expecting to generate. This paper aims to develop a population projection curve for the city over the next 20 years, to have a better idea of how much wastewater the city will create to size the wastewater system. Conservative, moderate, and aggressive population projections were evaluated to determine the best population projection scenario.

Assumptions

- Residential areas in study areas: 2.23 persons/LUE (Census.gov)
- Homes (LUE): 1.23LUE/acre (Moderate & Aggressive)
- Homes (LUE): 2 LUE/acre (Study areas B, D, E, F, and G in Aggressive model)
- Only areas with 0-25% slope change will be inhabited (city ordinance)

9.2.2.1 Conservative Population Projection

In the conservative scenario, historical population data was collected from Census.gov and The Jonestown City Comprehensive Plan. The population data showed a slow increment of growth in the past two decades that has averaged 46.7% for Jonestown, which is approximately 2.34% per year. This is lower than but still comparable to Travis County, which has averaged 3.8% growth per year over the past two decades. A contributing factor to this lag in growth is that the city does not have much existing utility, infrastructure, or wastewater treatment. With the new plan for a wastewater system, the city will be able to compete with the surrounding areas in Travis County, and population will increase at a rate similar to the Travis County averages. There are other factors that affect the population growth in Jonestown, but once the wastewater system is updated, infrastructure will cease to be a limiting factor.

With exclusions of all other uncertainties and probabilities that might potentially affect the growth rate in the future, the average growth rate in the conservative scenario was based fully on the historical growth rate in the past twenty years. In this estimation, the population would increase nearly one thousand people in the next 20 years. However, this estimation would be the most inaccurate projection of population growth because this model does not take new developments into account. Given the current development climate and the planned development of a sewer system and convenient utility supply, a higher growth rate than the historical average is highly probable. The conservative model could be used as a possible projection of growth if a “do nothing” alternative is selected. The population projection for the conservative model is shown in Table 2 and a graphical representation in Figure 3.

Table 2: Census data projections for a conservative model

Year	Population Estimates
1990	1250
2000	1681
2010	1834
2015	2045
2020	2194
2025	2344
2030	2492
2035	2641

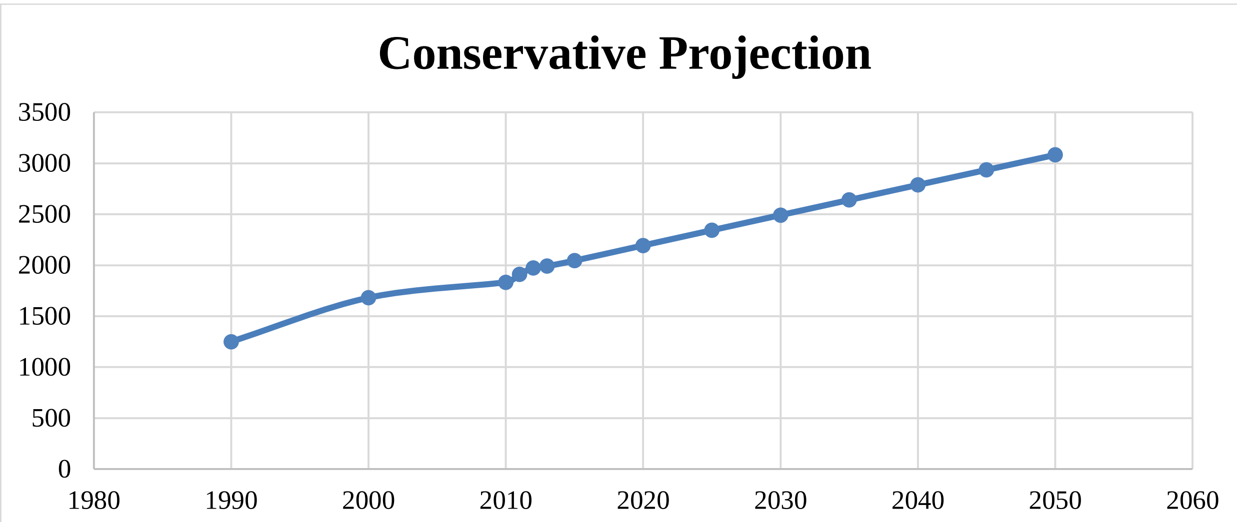


Figure 3: Conservative model for population projections

9.2.2.2 Moderate Population Projection

In the moderate scenario, the team evaluated the topography within the study areas and excluded the areas that were too steep or might be too costly to build units on. After reviewing the city ordinances for Jonestown, we found that the city does not allow development on land steeper than 35%, so areas included in the projection ranged from 0-25% slope change.

As a baseline growth, it was assumed that the starting population for the moderate projection would be the current population for 2015. Assuming that Jonestown will follow the same population density trend as Travis County, there will be approximately 2.53 person/LUE, according to Census.gov. If Jonestown continues the same development patterns seen in current development plans and housing seen in the Hollows, a subdivision in Jonestown that developed recently as a residential area, it will have approximately 1.23 LUE/acre. This number is a good assumption for the rest of the study area due to the similar topography. Areas were calculated using the measuring tool in AutoCAD.

In order to determine the number of LUE's per area, the following equation was used:

$$LUE = A * \frac{1.23}{acre} \quad (1)$$

Where:

LUE= Total number of homes in each study area

A= Total area of each study area, acre

The population in each of the study areas was determined as follows:

$$Population = \#\ of LUE * 2.53\frac{person}{LUE} \quad (2)$$

Where:

Population= Total number of people within each study area

LUE= Living Unit Equivalent

By applying equations 1 and 2 to each study area, the total population and the number of LUEs (homes) was determined. In order to project this population, since these areas are not yet developed, a growth assumption was made. For the moderate model, the assumption is that growth for each area will vary over 8-13 years. Within this time frame, all these areas will have been fully developed. Percentage growth over these years varies from 2%-14% of each total study area.

Table 3: Study Area Map areas used to estimate moderate population

Area	Acre	Houses	Population
B	380	468	1183
C	202	248	629
D	121	149	377
E	260	320	809
F	33	41	103
G	1241	1527	3863
H	192	237	599
Total	2430	2990	7561

Table 4: Population projection data for a moderate scenario

Calendar Year	Year of Study	Population
2015	0	2045
2020	5	2124
2025	10	4355
2030	15	7742
2035	20	9606

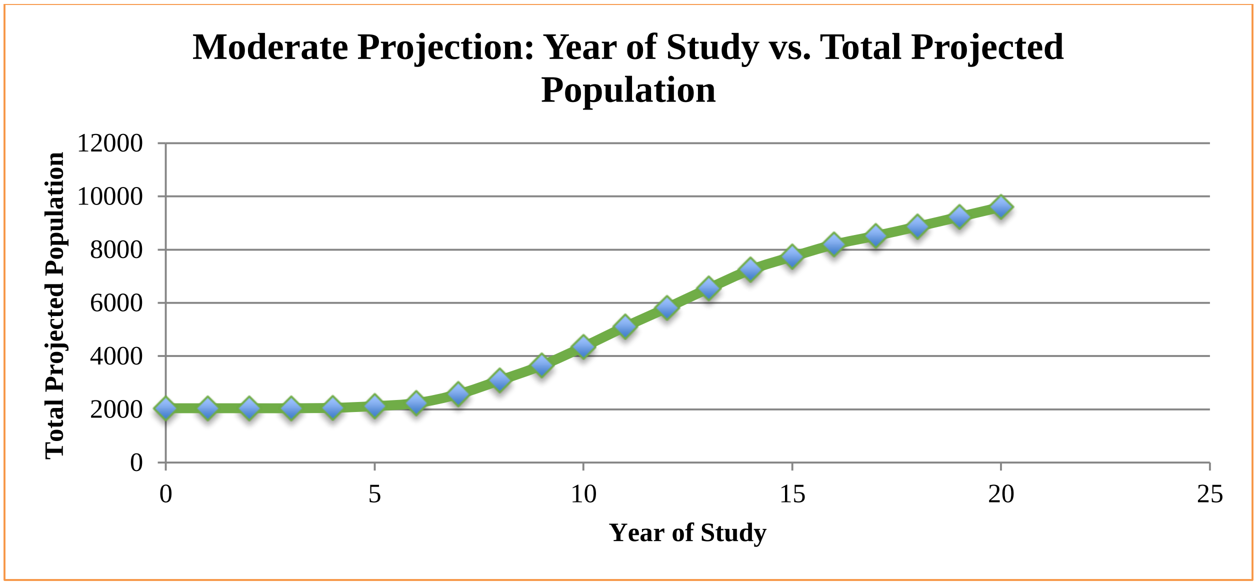


Figure 4: Moderate model for population projections

9.2.2.3 Aggressive Population Projection

In the aggressive scenario, assumptions remain the same except for the amount of LUEs/acre. Sections B, D, E, F, and G were assumed to have 2 houses/acre in order to portray an aggressive development compared to the moderate projection. The growth rate of each different section was spread over a time frame of about 4 to 10 years depending on size. Within this time frame, all these areas will have been fully developed. Percentage growth over these years varies from 10%-30% of each total study area.

Table 5: Study Area Map areas used to estimate aggressive population

Area	Acre	Houses	Population
B	380	760	1924
C	202	248	629
D	121	242	612
E	260	520	1315
F	33	66	167
G	1241	2483	6281
H	192	237	599
Total	2430	4556	13571

Table 6: Population projection data for an aggressive scenario

Calendar Year	Year of Study	Population
2015	0	2045
2020	5	2675
2025	10	5049
2030	15	9457
2035	20	13571

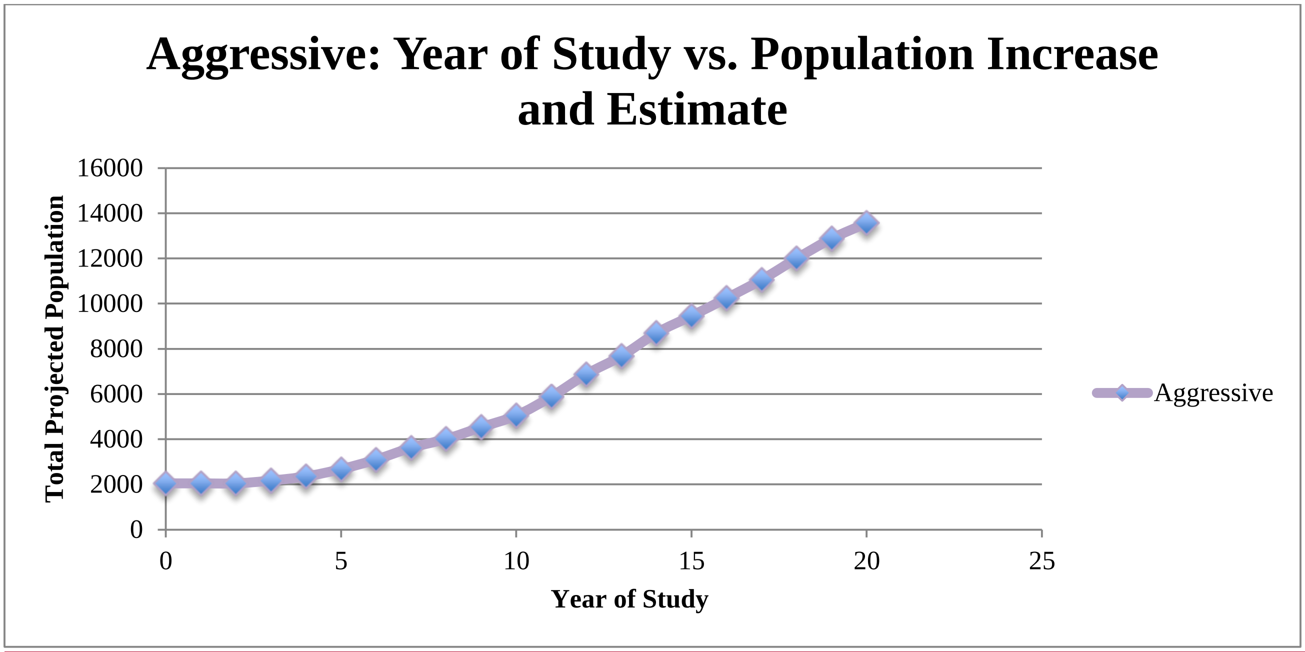


Figure 5: Aggressive model for population projections

Comparison

In Figure 5, a comparison of all three projections has been made to depict differences in population. Most importantly, the rate at which growth occurs is seen throughout all three projections. For calculations, the aggressive model was used because it was the best estimation that supported the need for a wastewater collection system. Also, it was a good estimate for a preliminary wastewater collection design.

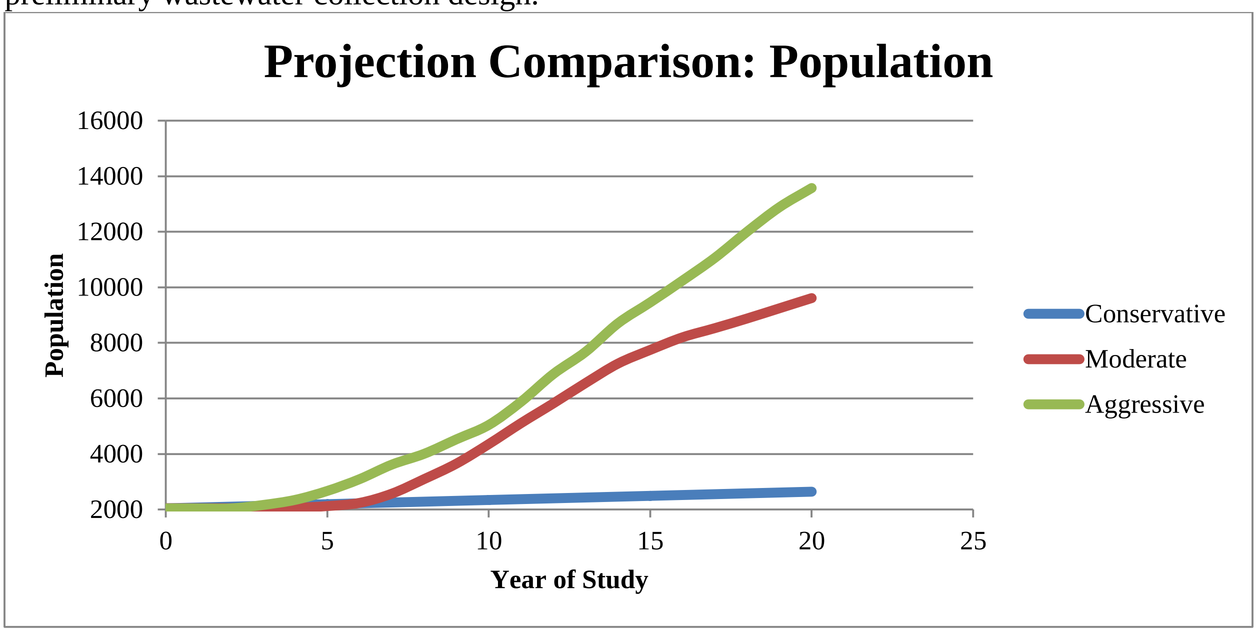


Figure 6: Projection comparison for population in all study areas

9.2.3. Wastewater Capacity Calculations

9.2.3.1. Relevant Assumptions

The general assumptions made, under the guidance of engineering lead Frank Phelan, for the ultimate time horizon were as follows:

- 4 LUE/acre for residential land use in the commercial corridor,
- 6 LUE/acre for commercial land use in the commercial corridor,
- 1.23 LUE/acre for residential land use in Study Areas B, C, D, E, F, G, and H,
- Developable land excludes areas greater than 25-35% slope, and
- Developable land excludes areas within the 100 year floodplain and the adjoining 25 foot LCRA buffer zone.

The assumptions made for the 10 year time horizon were as follows:

- Wastewater generation of 75 gallons/day/person,
- Wastewater generation of 200 gallons/day/LUE, and
- 6 LUE/acre for commercial land use.

An LUE, or living unit equivalent, is a unit of measurement used to define the typical flow produced by a single family residence.

9.2.3.2. Developable Land Estimates

The amount of developable land was estimated in order to determine the ultimate horizon wastewater capacity generation, in which all available land is developed. Areas within the study boundaries greater than 25-35% slopes were deemed too steep for land development, and so were excluded from developable land estimates. Areas within the 100 year floodplain and the adjoining 25 foot LCRA buffer zone were also excluded from estimates, under the guidance of Frank Phelan (LCRA, 2007).

In order to properly size the piping and WWTPs, it is important to know how many people will be using the land and for what kind of use. Looking at the developable land in each area gives an idea of how many people will be living in each study area. Since Jonestown is zoned to be predominantly residential with a small commercial corridor, the WWTP will only need to be able to handle waste from residential and commercial uses.

9.2.3.3. Capacity Calculations

To determine the size of the pipes, the number of LUEs and the total developable land area were used. The projected number of LUE's for each individual pipe was calculated by estimating the developable land that would feed into each specific pipe. Once the total amount of LUE's was determined for each pipe, a sizing spreadsheet was utilized to determine the pipe diameter based on total flow through the pipe. For pipes that collect from multiple sources, the LUE's from each pipe that feeds into the main pipe were added up and used to determine the size for that pipe.

9.2.4. Design Spreadsheets

In order to keep calculations consistent with the industry and our engineering lead, Frank Phelan, two spreadsheets with integrated equations for pipe sizing and lift station design were utilized. These spreadsheets were provided by Jay Engineering Company, Inc., an engineering firm from Leander, TX. The required information needed to apply these spreadsheets was determined from measurements done on the AutoCAD map of Jonestown, population projections, and TCEQ regulations. See Appendix B at the end of this report for samples of the following design spreadsheets.

9.2.4.1. The Wastewater Capacity Modeler Spreadsheet

The Wastewater Capacity Modeler spreadsheet was created to determine the pipe diameter for gravity lines in wastewater collection networks. The inputs for the Modeler included the number of LUEs, magnitude of area served, actual pipe diameter, Manning's roughness coefficient (n), and minimum pipe slope. Manning's roughness coefficient refers to the roughness of open channels and is assumed to be 0.013 according to the minimum requirements of the TCEQ §217.53 standards. The minimum pipe slope varies for different pipe sizes and can be found in TCEQ §217.53 Table C.1. The pipe diameter was chosen based on criteria for, flow velocity, full-flow capacity, and design flows.

The flows and peaking factor section on the spreadsheet refer to the full flow velocity, average dry weather flow and the peaking factor. The average dry weather flow is the sewage flow measured following a 7-day rainless week. This is an important aspect of the design because if flow is too slow due to the lack of water infiltration, sewage will have a longer sitting time, which causes sedimentation of solid particles and undesirable biological reactions. The average dry weather flow was calculated as follows (Jay Engineering Company, 2015):

$$Avg.\,Dry\ Weather\ Flow\ (ADWF) = LUE * \frac{200gpd}{LUE} * \frac{1\ day}{1440\ minutes} \tag{3}$$

Where:

ADWF = wastewater flow (gpm)
LUE = Living Unit Equivalent

The full flow velocity was calculated as follows (Davis, 2010):

$$V = \frac{1.486}{n} * R^{\frac{2}{3}} * S^{\frac{1}{2}} \tag{4}$$

Where:

V = Velocity of the pipe (fps)
n = Manning's roughness coefficient (unitless)
R = Hydraulic radius (ft)
S = Minimum pipe slope (ft/ft)

The peaking factor is used to determine the peak flow of the proposed design capacity. It was calculated as follows (Jay Engineering Company, 2015):

$$PF = \frac{18 + (0.0206 * \mathrm{ADWF})^{0.5}}{4 + (0.0206 * \mathrm{ADWF})^{0.5}} \tag{5}$$

Where:

PF= Peaking Factor
ADWF= wastewater flow (gpm)

Capacity calculations for the pipes include the flow capacity of the pipes (Q), the peak dry weather flow (PDWF), and the peak wet weather flow (PWWF). The flow capacity of the pipes was determined as follows (Jay Engineering Company, 2015):

$$Q = V * A * \frac{7.485\ gal}{ft^3} * \frac{60\ sec}{min} \tag{6}$$

Where:

Q = Flow rate (gpm)
V = Velocity of the pipe (fps)
A = Cross-sectional area of the pipe (ft^2)

PDWF and PWWF were calculated as follows (Jay Engineering Company, 2015):

$$PDWF = Q_{65\%} = 0.65 * Q \tag{7}$$

$$PWWF = Q_{85\%} = 0.85 * Q \tag{8}$$

PDWF is assumed to be 65% of the flow capacity of the pipe and occurs when there is no infiltration into the pipe. PWWF is assumed to be 85% of the flow capacity in the pipe and occurs when there is infiltration. The design flow refers to the peak dry weather flow and peak wet weather flow in relation to the areas served. This differs from the previously calculated PDWF and PWWF because those are in terms of pipe theoretical capacity versus actual wastewater flow being produced. The design flow peak dry weather, inflow & infiltration, and peak wet weather flow were calculated as follows (Jay Engineering Company, 2015):

$$PDWF_{design\ flow} = Avg.Dry\ Weather\ Flow * Peaking\ factor \tag{9}$$

$$Inflow\ and\ Infiltration\ (gpm) = \#of\ acres\ served * \frac{750\,\frac{gpd}{acre}}{1440\,\frac{min}{day}} \tag{10}$$

$$PWWF_{design\ flow} = PDWF + Inflow\ and\ Infiltration \tag{11}$$

The PDWF and PWWF of the pipes had to be greater than the design flow PDWF and PWWF because this indicated that the pipe size would be sufficient to hold the expected flow production. Pipe diameters were chosen according to whether the pipes could sustain the expected capacity.

9.2.4.2. The Lift Station Design Spreadsheet

The *Lift Station Design* spreadsheet was used to design the lift station specifications, including lift station capacity, wet well dimensions, force main diameter, and pump specifications. Flow development of the lift stations was based on the peak wet weather flow. This determined what capacity the lift stations and pumps had to be in order to handle the expected incoming flow. The peak wet weather flow was calculated as follows (Jay Engineering Company, 2015):

$$PPWF = PDWF + INFLOW\ AND\ INFILTRATION \tag{12}$$

The PDWF and PWWF for lift station design is the accumulation of flows from incoming gravity pipes. Lift station and pump capacity is chosen based on the PWWF. The capacity must be higher than the calculated PWWF.

The next step was to determine the depth of the wet well required for the lift station. The purpose of a wet well is to store wastewater and to provide sufficient submergence to the pump suction inlet to maintain suction and prevent pump cavitation (Davis, 2010). 6 and 8-foot diameter wet wells were used because they provided the best detention times and could sustain the minimum wet well working volume. The minimum well working volume (WWV) was calculated as follows (Jay Engineering Company, 2015):

$$WWV_{min} = \frac{P * 6}{4} \tag{13}$$

Where:

WWV_{min} = minimum well working volume (gal)
P = Pump capacity (gpm)
6 = minimum cycle time (min)

The total static head was determined from the difference in elevation between the force main outflow and the force main inflow. Based on the Q-H system curve needed to transport the wastewater, an optimal pump system could be determined based on published pump performance curves.

9.2.4.3. Force Mains Spreadsheet

Since the topography in Jonestown has areas that are not favorable to gravity sewage systems, the use of force mains became an alternative to overcome high slope changes. Since TCEQ §217.62 mandates flow velocity in force mains to be in the range of 3-7 fps, the diameter of the force main was determined based on this required velocity and the expected flow production from the serviced LUEs. The following equation was used to find the pipe flow velocity for determining the desired pipe diameter (Jay Engineering Company, 2015):

$$V = \frac{Q}{A} \tag{14}$$

Where:

V = Velocity of the force main (fps)
Q = Flow rate in the force main (ft^3/s)
A = Cross sectional area of the force main pipe (ft^2)

By inputting the known quantities, the diameter was interpolated for the required velocity, 3-7 fps.

9.2.5. Vulnerability Mapping

9.2.5.1. DRASTIC Methodology

The DRASTIC model method consists of several different types of data and information, including: depth to water (D), net recharge (R), aquifer media (A), soil media (S), topography (T), impact of the vadose media zone (I), hydraulic conductivity of the aquifer (C.) The DRASTIC model equation is as follows:

$$DRASTIC\ INDEX(DI) = D_R D_W + R_R R_W + A_R A_W + S_R S_W + T_R T_W + I_R I_W + C_R C_W \tag{15}$$

Where: R = rating (Refer to Tables 4,5,6,7,8,9,10 – EPA DRASTIC Handbook)
W = weight (Table 2 – EPA DRASTIC Handbook)

The higher the DRASTIC INDEX (DI) the more susceptible the area is to contamination. The use of the DRASTIC model for the city of Jonestown risk map used data from: United States Geological Survey (USGS), the Web Soil Survey (WSS), Environmental Protection Agency (EPA), and the Jonestown comprehensive plan and city ordinances.

9.2.5.2. DRASTIC Data Collection and Assumptions

Table 1 from the EPA DRASTIC handbook contains information on where the data for the DRASTIC parameters were found for Tables 2-11 (USEPA, 1987.)

The Jonestown Comprehensive plan clearly states that the city of Jonestown area lies on a karst limestone area with no aquifer area directly beneath (Espey Consultants Inc, 2006). With this information, a safe assumption was made for the “R” value of the DRASTIC model; the value can be zero or eliminated from the equation for this specific project.

Since there is no aquifer water to consider in the area, eliminating the “C” value (the hydraulic conductivity of the aquifer water) was a valid assumption.

Because the Jonestown Comprehensive plan states the city is on a karst limestone area aquifer media, the “A” value for aquifer media was assumed constant for the entire City of Jonestown and its ETJ area.

The Jonestown comprehensive plan didn’t specifically state information regarding a water table, and it was decided not to eliminate the “D” value (depth to water table) from the DRASTIC model. Data for the depth to water table was taken from the United States Geologic Survey (USGS) website and was found to be 200 feet. It was assumed that this value was constant for the entire city of Jonestown and its ETJ area.

The impact of the vadose zone was assumed to be equal in length to the depth to water table as the vadose zone takes into account the soil and geologic layer. Therefore, the “I” value was assumed to be equal to the “D” value.

Soil maps and topography maps were provided by the United States Department of Agriculture (USDA) and Natural Resource Conservation Service (NRCS) via the Web Soil Survey (WSS.) Each individual study area was mapped on the WSS.

9.2.5.3. DRASTIC Vulnerability Calculations

Excel was used to compute the data from the DRASTIC INDEX equation and corresponding EPA tables (USEPA, 1987.) A table for risk values of the Jonestown vulnerability map was generated and the results can be found in Table 7 below.

Table 7. Vulnerability Mapping table

Study Area	Soil Type	Slope	Rating	DI	Color code
B	W- Water				
	TdF- Tarrant-Rock outcrop complex	18-50%	1	117	Rose
	TaD- Tarrant soils	5-18%	5	121	Rose3
	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2
	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
C	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
	TcA- Tarrant and speck soils	0-2%	10	126	Red
	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2
D	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2
	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
	VoD- Volente silty clay loam	1-8%	8	124	Red
	TaD- Tarrant soils	5-18%	5	121	Rose3
E	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2
	TaD- Tarrant soils	5-18%	5	121	Rose3
	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
	VoD- Volente silty clay loam	1-8%	8	124	Red
	TcA- Tarrant and speck soils	0-2%	10	126	Red
F	VoD- Volente silty clay loam	1-8%	8	124	Red
	TaD- Tarrant soils	5-18%	5	121	Rose3
	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2
G	TcA- Tarrant and speck soils	0-2%	10	126	Red
	Md- Mixed alluvial land (frequently flooded)	0-1%	10	126	Red
	LcB- Lewsville silty clay	1-2%	10	126	Red
	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2
	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
	AgC2- Altoga silty clay	3-6%	9	125	Red
	TaD- Tarrant soils	5-18%	5	121	Rose3
	VoD- Volente silty clay loam	1-8%	8	124	Red
H	GP- pits, gravel	1-90%	1	117	Rose
	VoD- Volente silty clay loam	1-8%	8	124	Red
	BoF- Brackett-Rock outcrop-real complex	8-30%	3	119	Rose2

	BiD- Brackett-Rock outcrop- complex	1-12%	8	124	Red
	W – Water				
	TaD- Tarrant soils	5-18%	5	121	Rose3

According to the excel data, the majority of Jonestown has very high levels of contamination risk, primarily due to the topography of the areas. Since all areas initially rated "high – very high", the "very low – very high" scale was revalued to include all ratings within the original risk evaluation. Refer to Table 8 below for the general and revalued scale.

Table 8. General DRASTIC scale and revalued scale used for Jonestown.

General Scale	DI Value		Revalued Scale	DI Value
Very Low	19		Very Low	115
Low	45		Low	117.75
Moderate	70		Moderate	120.5
High	115		High	123.25
Very High	160		Very High	126

After downloading the ArcGIS information from the WSS, and reviewing the information from the excel tables, a "color system" was devised marking which areas were: very low to low, low to moderate, moderate to high and high to very high susceptibility to contamination based on the DRASTIC INDEX (DI) value for each area. Refer to the Appendix for the Vulnerability Maps for the study areas B,C,D,E,F,G,H.

10. Final Recommendation

10.1. WWTP Locations and Specifications

Since the study areas are separated, the final recommendation for the WWTPs is that 3 separate WWTPs are to be built. WWTP1 will serve Study Areas C, D, E, F, and H. WWTP2 will service Study Area G. WWTP3 will service Study Area B. The three zones that the study areas have been divided into will allow each WWTP to service the surrounding study areas.

10.1.1. WWTP1 Location

Study Areas C, D, E, F, and H, along with portions of the commercial corridor will feed into a treatment plant located in the center of Study Area C, labeled WWTP 1. Area C was chosen because it is the most centralized of the study areas, and so will be the most cost efficient location which to convey the wastewater. See Figure 7 for the location of the treatment plant located in the center of Study Area C.

The suggested placement of WWTP 1 within Study Area C was based on the grade of the land, the sizing of the WWTP, the elevation, and the proximity to the 100-year floodplain. It would be difficult and costly to build in an area where the slope is too great because it would be necessary to excavate large amounts of land. Therefore, an area with a low slope was chosen.

A disadvantage is that the chosen location is at an elevation that is slightly higher than a good portion of the serviced study areas. This means that there will have to be a force main connecting the plant to a lift station collection point downhill to WWTP1, located in the commercial corridor.

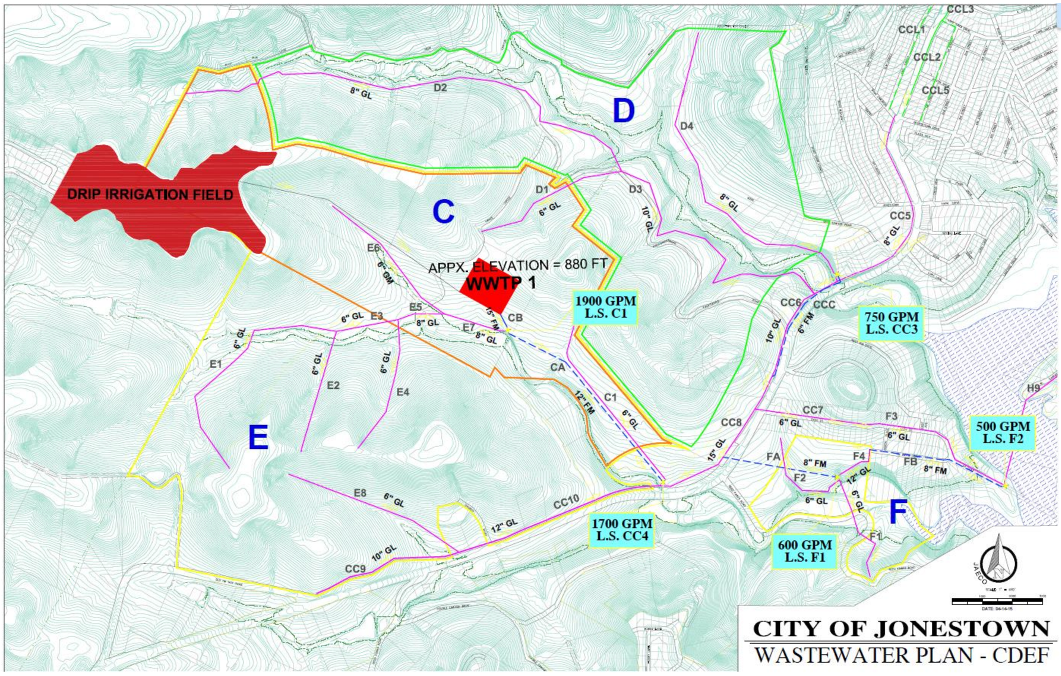


Figure 7. Location of WWTP 1 and Drip Irrigation Field in Study Area C

10.1.2. WWTP2 Location

The second wastewater treatment plant will be placed within Study Area G. The plant can be seen in Figure 8, labeled WWTP 2. This plant will only provide service for Study Area G. This may change in the future as more land is developed in the surrounding areas since this study area is isolated, but the scope of this project focuses only within the study area boundaries. The criteria for placement of this WWTP is the same as the first: topography, size of plant, elevation and proximity to 100-year floodplain. There was also a water well located within the study area, and so the plant had to be placed at least 500 feet away. The suggested location has the most level grade within Study Area G. The elevation is roughly average of the surrounding study area, meaning that both gravity lines and force mains will be required. Similar to WWTP 1, gravity lines flow into low points and lift stations pump the wastewater uphill to WWTP2.

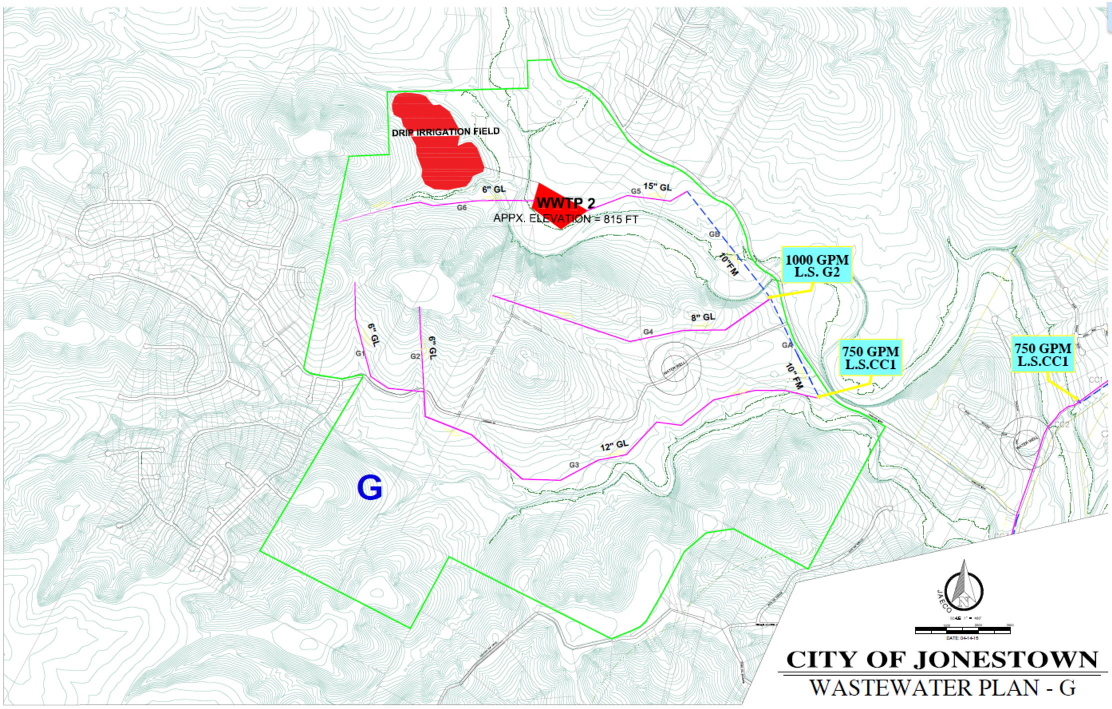


Figure 8. Location of WWTP 2 and Drip Irrigation Field in Study Area G

10.1.3. WWTP3 Location

The suggested WWTP in Study Area B can be seen in Figure 9, labeled WWTP3. This treatment plant will service only Study Area B. "The Hollows," suburb adjacent to Study Area B has a wastewater agreement with Lago Vista and so was removed from the scope of this project. The location criteria for WWTP3 are the same as for the other two plants.

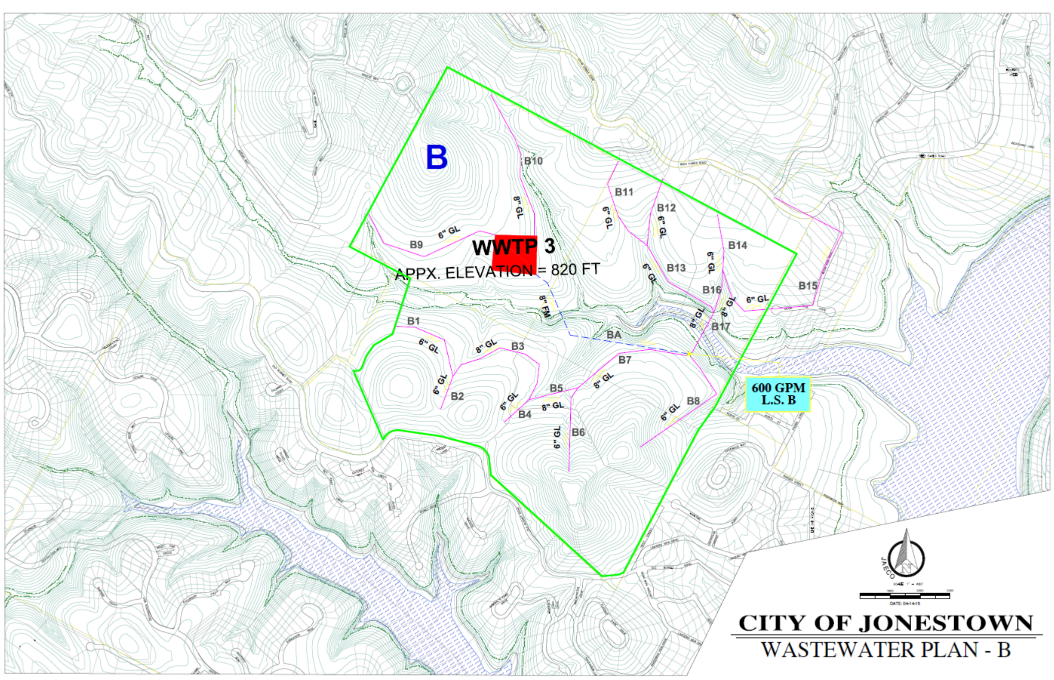


Figure 9. Location of WWTP 3 in Study Area

10.1.4. WWTP Sizes and Specifications

According to Frank Phelan, P.E. (Jay Engineering Company, Inc., personal correspondence, 21 April 2015), the wastewater treatment facilities should have 20:20 (TSS:BOD) effluent characteristics. Furthermore, WWTP 1 should be able to process both residential and commercial wastewater since it will be servicing the commercial corridor in addition to residential developments. Only residential wastewater will be processed by WWTPs 2 and 3. These characteristics will be important to consider when completing the final cost analysis of the project. The WWTPs' capacities are calculated from the flow from the collection network:

$$Capacity\ (MGD) = \frac{Pipe\ flow\ (gpm) * 1440\,\frac{min}{day}}{Peaking\ Factor} \tag{16}$$

Each of the capacities of the WWTPs are shown in Table 9 and reflect the ultimate horizon and inclusion of all study areas.

Table 9. WWTP Capacity for ultimate development.

WWTP	**Location (Study Area)**	**Study Areas Serviced**	**Capacity (MGD)**
1	C	C, D, E, F, H, CC	**0.86**
2	G	G	**0.40**
3	B	B	**0.22**

10.2. Effluent Discharge

10.2.1. Drip-field Locations

The recommended location for the drip fields associated with WWTP1 and WWTP2 can be seen in Figure 7 and Figure 8, respectively. WWTP3 does not currently have a projected drip field or recommended location currently selected.

For WWTP1, the drip field location was chosen to be the best fit between current land uses, land zoning, and topographical slope. This location has the lowest slope rise across the whole of the drip field, and is placed in an area that has not been developed. According to TAC §222.81, the drip field cannot be within 500 feet of public water wells, 150 feet of private water wells, or 100 feet of surface water. Precise drip field sizing is out of the scope of this project.

The drip field for WWTP2 was selected with the same process as the first.. This location has low slope relative to the rest of the surrounding area, and it is set in an undeveloped plot of land. It meets TAC criteria, and the sizing is handled similarly as the first drip field, with specifications not required for the purpose of this project.

10.2.2. Re-use Plan

Drip fields will be utilized to disperse the treated wastewater from the WWTP. This type of drip line is the recommended design since it can be used on the widest range of slope options and is easily built and maintained.

The physical process of the effluent reuse will start at each WWTP, and the effluent is then pumped through force mains to each dispersion site location. The drip field details such as layout of the drip field and sizing of the individual drip field pipes are beyond the scope of this project, but can be developed from the work shown here for this project on WWTP sizing, total effluent flow, and force main sizing.

10.3. Collection Network Layout and Specifications

A well-laid out and properly sized wastewater collection network was fundamental to the overall project objectives. This network included a system of pipes and lift stations. In order that the City of Jonestown would be able to fund this project effectively and sustainably, a phased design of incremental project implementation is recommended.

10.3.1. Pipes

The pipe layouts for each study area are shown below in Figures 10-14. The mains and laterals were laid out in such a way to meet the following criteria:

- Provide wastewater collection service access to all parts of the study areas,
- Minimize pipe length
- Follow the topography, usually running along the valleys to allow wastewater to be collected from all hills by gravity, and
- Follow right-of-ways (ROWs) for easy, organized infrastructure construction.

For all gravity mains and force mains, the chosen pipe material is PVC. This is also the preferred material for pressure systems (Davis, 2010). The benefits of PVC include corrosion resistance, low density per unit length, and low installation cost. For the discharge pipes within the lift stations, ductile iron is chosen. This is a common material to choose for pressure pipes (Davis, 2010).

The final pipe sizes are shown in Table 10 below. The sizes were chosen to provide adequate wastewater collection capacity for ultimate development of all study areas. Therefore, with regards to capacity, no system upgrades will be needed, even as new pipes are added in phases.

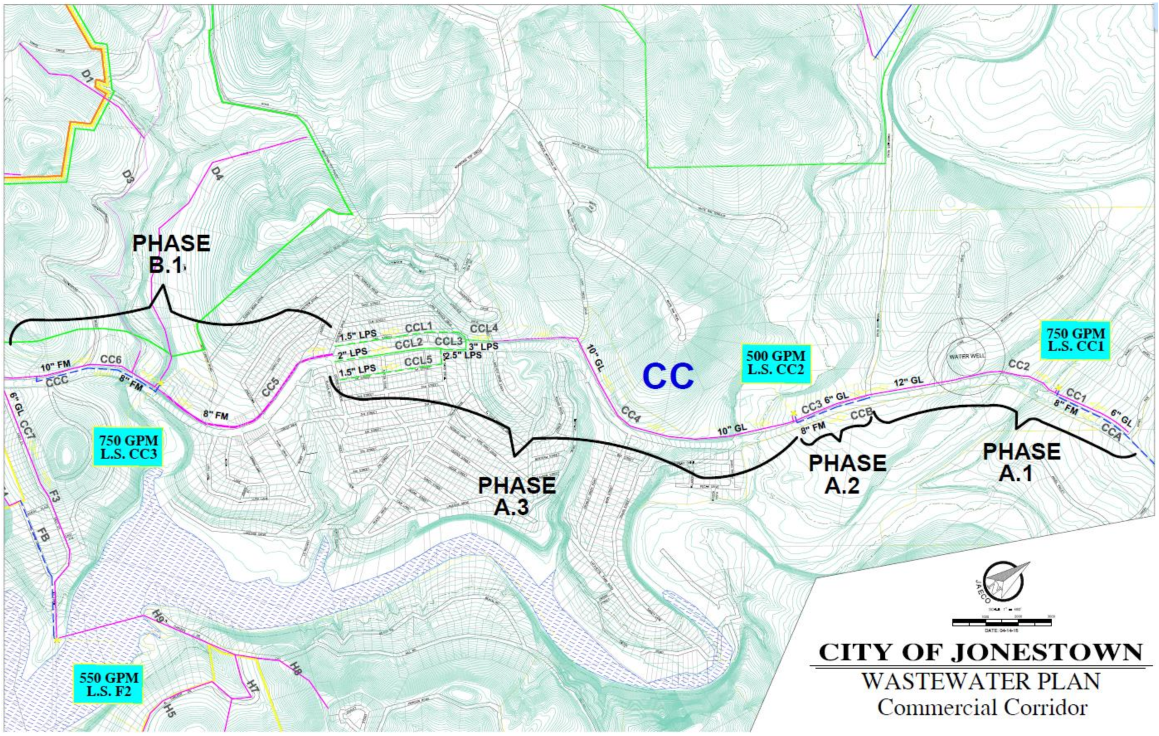


Figure 10. Pipe layout for the commercial corridor.

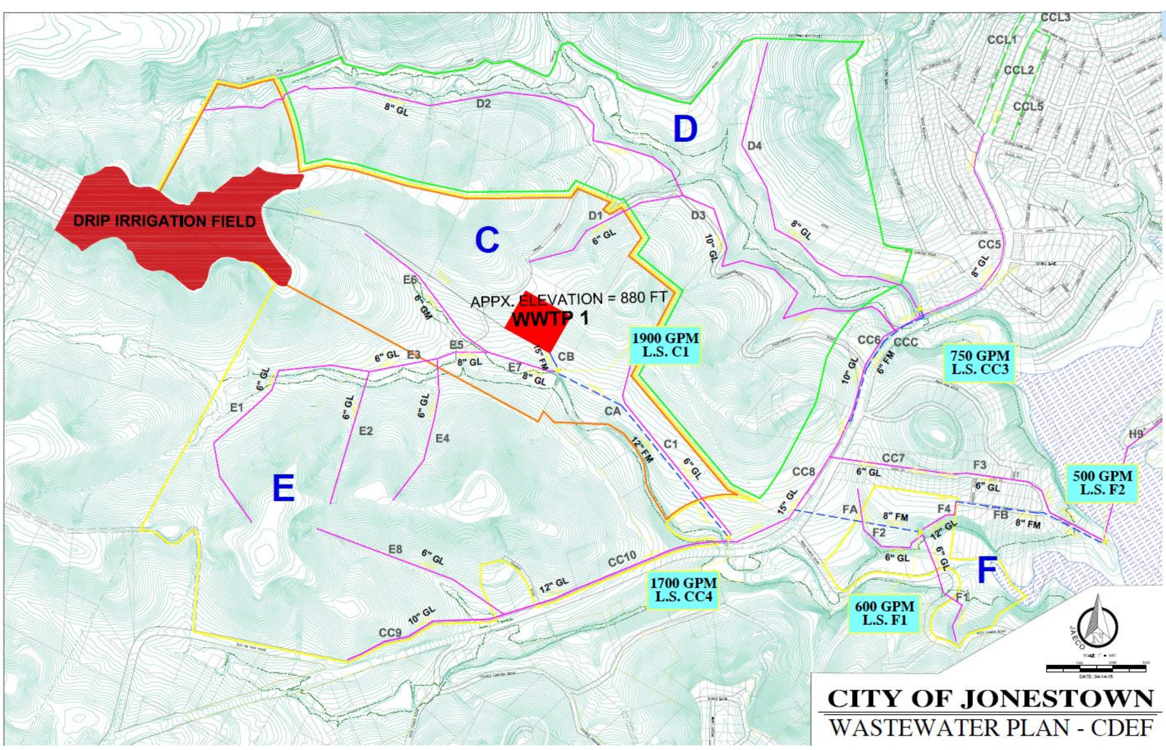


Figure 11. Pipe layout for Study Areas C, D, E, and F.

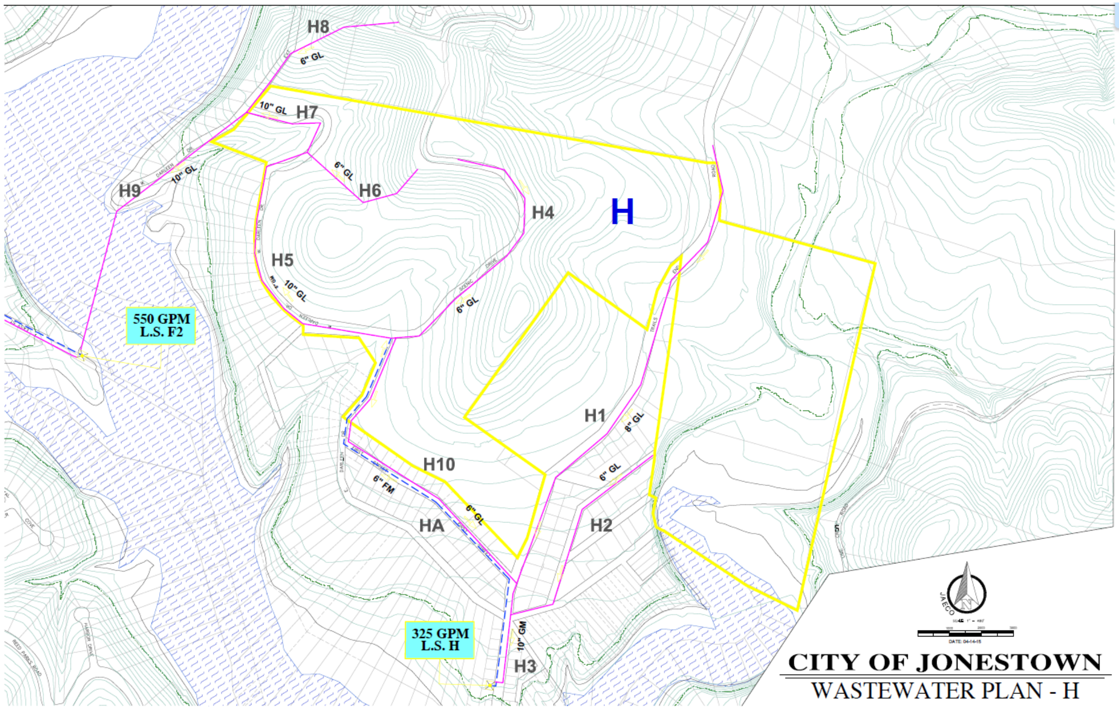


Figure 12. Pipe layout for Study Area H.

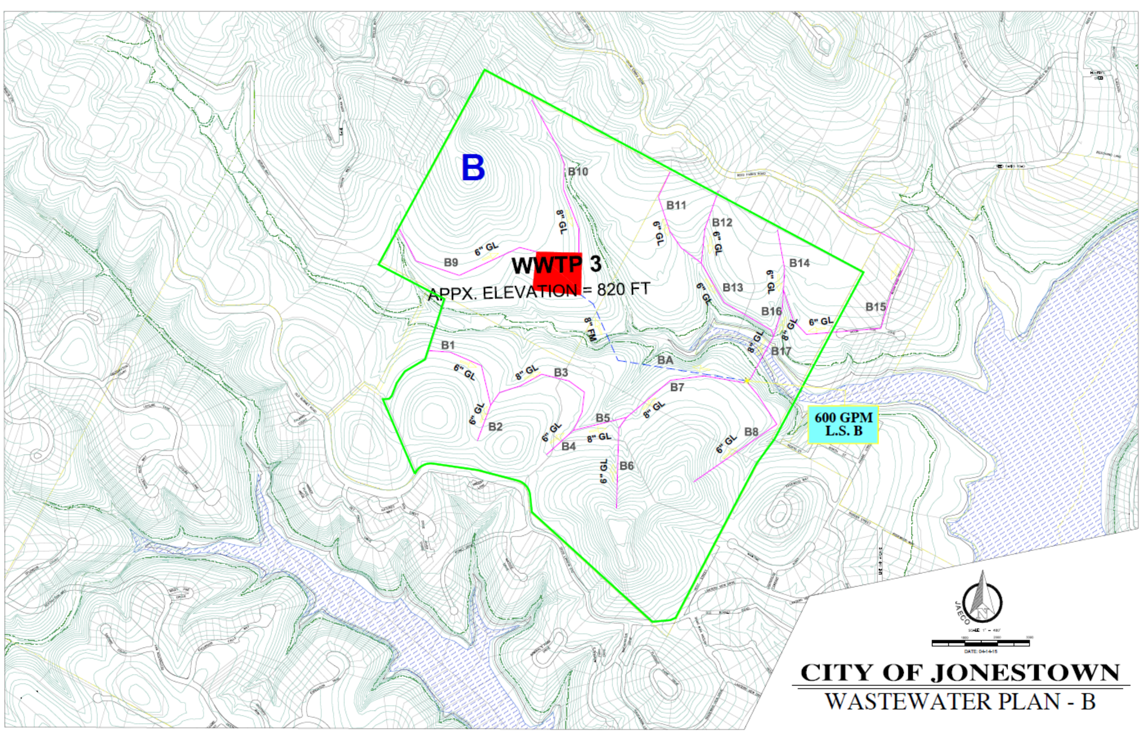


Figure 13. Pipe layout for Study Area B.

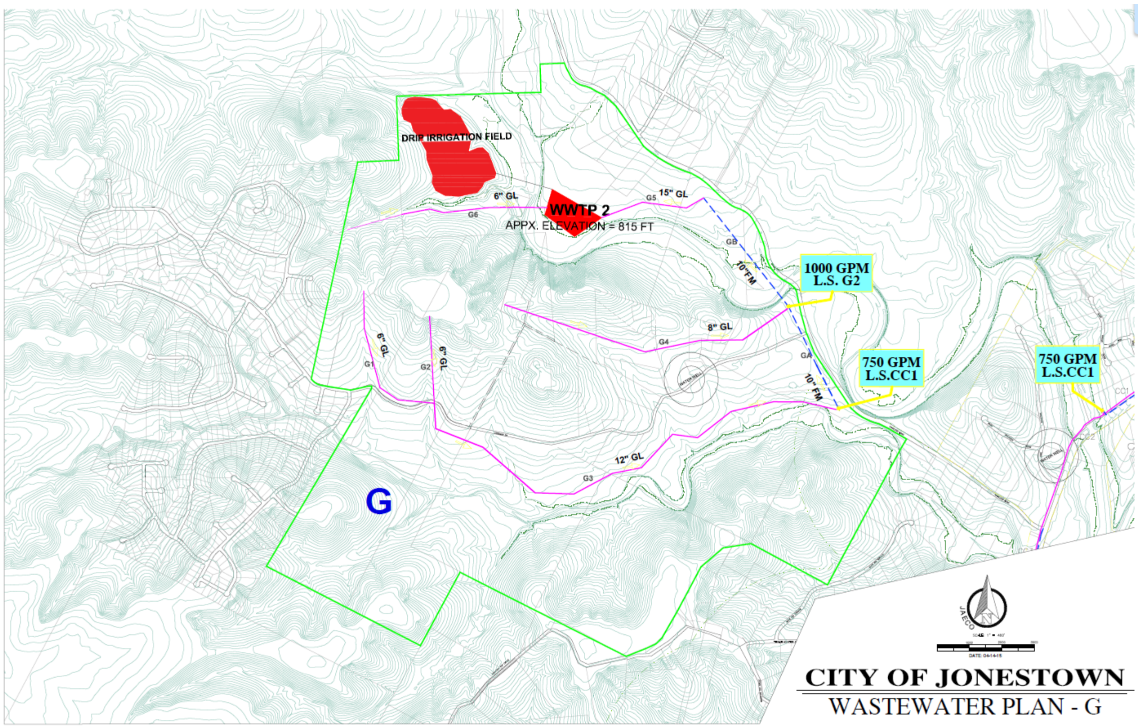


Figure 14. Pipe layout for Study Area G.

Table 10. Final pipe sizes.

Study Area	Pipe ID	Pipe Type	Material	Pipe size (in.)	Length (ft)
B	B1	Gravity	PVC	6	2,153
	B2	Gravity	PVC	6	468
	B3	Gravity	PVC	8	1,838
	B4	Gravity	PVC	6	448
	B5	Gravity	PVC	8	675
	B6	Gravity	PVC	6	1,150
	B7	Gravity	PVC	8	1,671
	B8	Gravity	PVC	6	1,853
	B9	Gravity	PVC	8	3,245
	B10	Gravity	PVC	8	2,209
	B11	Gravity	PVC	6	1,490
	B12	Gravity	PVC	6	923
	B13	Gravity	PVC	6	1,314
	B14	Gravity	PVC	6	746
	B15	Gravity	PVC	6	3,635
	B16	Gravity	PVC	8	584
	B17	Gravity	PVC	8	583
	BA	Force Main	PVC	8	2,559

C	C1	Gravity	PVC	6	3,017
	CA	Force Main	PVC	12	2,895
	CB	Force Main	PVC	15	190
CC	CC1	Gravity	PVC	6	1,080
	CC2	Gravity	PVC	12	2,569
	CC3	Gravity	PVC	6	1,016
	CC4	Gravity	PVC	10	4,635
	CC5	Gravity	PVC	8	2,705
	CC6	Gravity	PVC	10	1,749
	CC7	Gravity	PVC	6	1,232
	CC10	Gravity	PVC	12	2,819
	CC8	Gravity	PVC	15	1,713
	CC9	Gravity	PVC	10	2,012
	CCA	Force Main	PVC	8	2,251
	CCB	Force Main	PVC	8	1,069
	CCC	Force Main	PVC	8	1,678
	CCL1	LPS	PVC	1.5	1,753
	CCL2	LPS	PVC	2	1,735
	CCL3	LPS	PVC	2.5	1,834
	CCL4	LPS	PVC	3	362
	CCL5	LPS	PVC	1.5	1,535
D	D1	Gravity	PVC	6	2,090
	D2	Gravity	PVC	8	6,518
	D4	Gravity	PVC	8	4,433
	D3	Gravity	PVC	10	4,051
E	E1	Gravity	PVC	6	3,388
	E2	Gravity	PVC	6	1,674
	E3	Gravity	PVC	6	833
	E4	Gravity	PVC	6	1,855
	E5	Gravity	PVC	8	627
	E6	Gravity	PVC	6	2,083
	E7	Gravity	PVC	8	711
	E8	Gravity	PVC	6	2,504
F	F1	Gravity	PVC	6	1,487
	F2	Gravity	PVC	6	1,401
	F3	Gravity	PVC	6	2,414
	F4	Gravity	PVC	12	593
	FB	Force Main	PVC	8	1,858
	FA	Force Main	PVC	8	1,535
G	G1	Gravity	PVC	6	2,905
	G2	Gravity	PVC	6	1,610
	G3	Gravity	PVC	12	8,389

	G4	Gravity	PVC	8	5,584
	G5	Gravity	PVC	15	1,994
	G6	Gravity	PVC	6	3,762
	GA	Force Main	PVC	10	2,101
	GB	Force Main	PVC	10	2,692
H	H1	Gravity	PVC	8	4,199
	H2	Gravity	PVC	6	1,824
	H3	Gravity	PVC	10	679
	H4	Gravity	PVC	6	2,306
	H5	Gravity	PVC	10	2,424
	H6	Gravity	PVC	6	1,117
	H7	Gravity	PVC	10	850
	H8	Gravity	PVC	6	1,479
	H9	Gravity	PVC	10	2,450
	H10	Gravity	PVC	6	2,333
	HA	Force Main	PVC	6	3,501

10.3.2. Lift Stations

Lift stations were located within each study area in order to maximize the efficiency of wastewater collection and conveyance under the constraint of the topography. In the final design, lift stations were located at the lowest points within the study areas so that wastewater could drain to them through gravity lines. In the final design, the team sought to minimize the number of lift stations by finding valleys that are central to the study areas so that large portions of the study areas could be serviced by each lift station.

The size of each lift station is determined by the total peak wet weather wastewater flow through a given lift station. This is determined by the amount of LUEs and within the lift station's service area. In the final design, the lift stations capacities are chosen such that each service area are capable of being serviced indefinitely, through ultimate development. Within each lift station, pumps are chosen to achieve the minimum cost and maximum efficiency. Pump and wet well characteristics for each lift station are shown in Table 11 below.

Table 11. Lift station pump and wet well specifications. Lift stations are designed to handle wastewater production from ultimate development.

Lift Station ID	Study Area	LUEs	Peak wet weather flow (gpm)	Pump Capacity (gpm)	Pump Max Static Head (ft)	Wet Well Diameter (ft)	Wet Well Depth (ft)
LSB	B	589	547	600	30.5	8	13.5
LSH	H	321	306	325	70.5	6	11.5
LSF1	F	626	580	600	84	8	14
LSF2	F	556	519	550	137	8	14
LSG1	G	836	763	800	25.5	8	14.5

LSG2	G	1060	955	1000	29	8	16
LSCC1	CC	1248	736	750	116	8	13
LSCC2	CC	766	480	500	43	8	13
LSCC3	CC	910	716	750	65	8	14
LSCC4	CC	2533	1644	1700	79.5	10	17.5
LSC1	C	2771	1832	1900	16	10	18

10.3.3. Phasing Plan

As shown in Figure 15 below, the wastewater collection system is divided into several phases and sub-phases. In this way, the wastewater collection system will grow along with the City of Jonestown. Jonestown will have the option to begin construction with partial funding and will not have excess infrastructure.

Phase A represents the northern half of the commercial corridor, including the downtown area. This Phase is further divided into three sub-phases, A.1, A.2, A.3, and B.1. This will incrementally provide the commercial corridor with wastewater collection service within the next five years. Since the commercial corridor is the most essential study area and an Interlocal Wastewater Collection Agreement with the City of Leander is already in place, this system can begin development.

Within the 10-year study horizon, Phase B.1 will be developed. Phase B.1 will incrementally provide wastewater service to (1) the “Jonestown 300” development of Study Area C, (2) the southern half of the corridor, and (3) the developments of Study Areas F and H.

Further beyond the 10-year study horizon will be the development of wastewater collection systems in Study Areas B, D, E, and G for the ultimate horizon. Since these developments will provide wastewater collection service independent of the other study areas, they can be developed any time, independent of progress in the other developments.

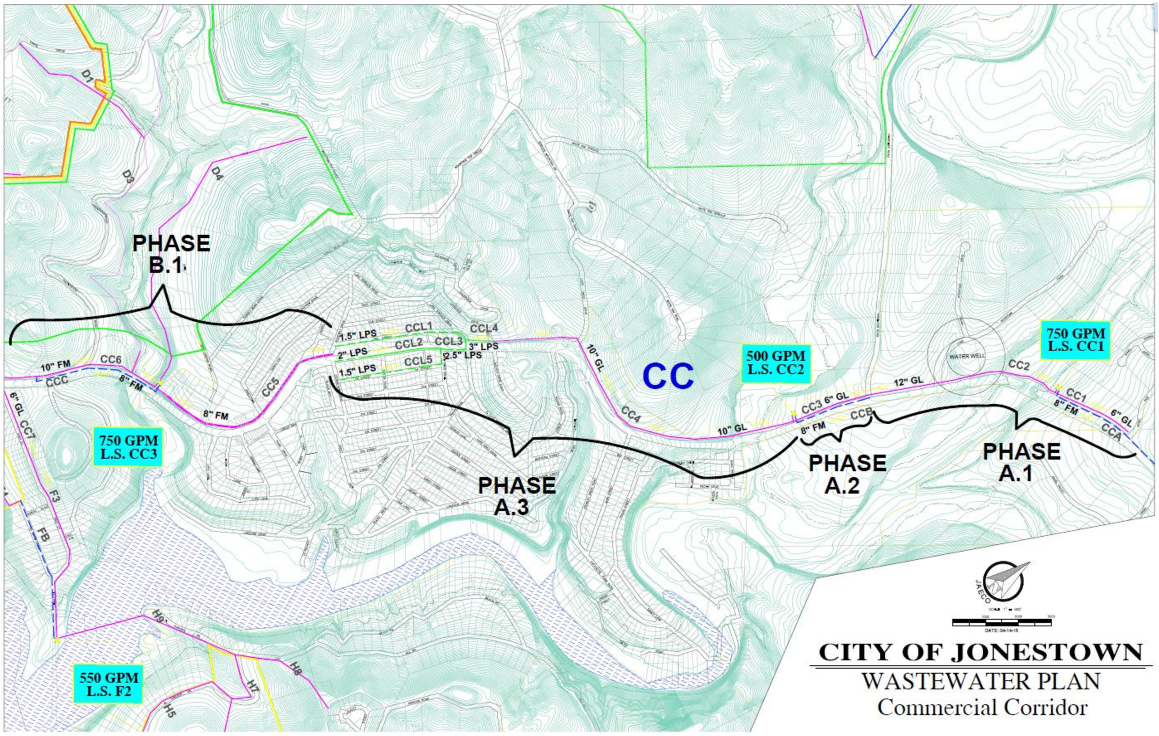


Figure 15. Overall map of wastewater collection system with phases and study areas delineated.

10.2.4. Opinion of Probable Cost/Cost Estimate

The cost estimate has been broken down by phases and study areas to show how Jonestown will need to fund the construction of the wastewater collection system over time (Tables 12 and 13). As this is a basic cost estimate, it includes only the cost of the pipes, lift station, and wastewater treatment plants. Soft costs (e.g., right-of-way acquisition and surveying fees) and contingency costs are not included in this analysis. Inherent to the pipe cost estimates, though, are construction costs and manhole costs. Inherent to the lift station cost estimates are the cost of generator sets, wet wells, and pumps. Also, the time value of money is not taken into consideration. These cost estimates are based on values given or interpolated from correspondence with Frank Phelan, P.E. (Jay Engineering Company, Inc., email, 21 April 2015).

Table 12. Total cost estimates for each study area.

Study Area	Total Cost Estimate
CC (Commercial Corridor)	$ 3,429,100.00
Phase A	**$ 1,747,800.00**
A.1	$ 750,500.00
A.2	$ 396,300.00
A.3	$ 601,000.00
Phase B	$ 1,681,300.00
B	$ 2,342,600.00
C	$ 826,200.00

D	$	1,464,000.00
E	$	1,018,300.00
F	$	1,154,800.00
G	$	3,091,300.00
H	$	1,998,900.00
	$	**15,325,200.00**

Table 13. Cost estimates broken down for each study area.

Study Area	Item Description		Units	Quantity (LF)	Unit Price		Cost	
CC – Phase A.1	6" Gravity Line	LF	1	1,080	$	73.40	$	79,300.00
	12" Gravity Line	LF	1	2,569	$	106.40	$	273,300.00
	8" Forced Main	LF	1	2,251	$	39.05	$	87,900.00
	Lift Station - 750 GPM	LS	1		$	310,000.00	$	310,000.00
							$	**750,500.00**

Study Area	Item Description		Units	Quantity (LF)	Unit Price		Cost	
CC – Phase A.2	6" Gravity Line	LF	1	1,016	$	73.40	$	74,600.00
	8" Forced Main	LF	1	1,069	$	39.05	$	41,700.00
	Lift Station - 500 GPM	LS	1		$	280,000.00	$	280,000.00
							$	**396,300.00**

Study Area	Item Description		Units	Quantity (LF)	Unit Price		Cost	
CC – Phase A.3	1.5" Pressure Sewer	LF	1	1,753	$	18.00	$	31,600.00
	1.5" Pressure Sewer	LF	1	1,535	$	18.00	$	27,600.00
	2" Pressure Sewer	LF	1	1,735	$	24.05	$	41,700.00
	2.5" Pressure Sewer	LF	1	1,834	$	26.05	$	47,800.00
	3" Pressure Sewer	LF	1	362	$	28.05	$	10,200.00
	10" Gravity Line	LF	1	4,635	$	95.40	$	442,200.00
							$	**601,000.00**

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
CC – Phase B	6" Gravity Line	LF	1	1,232	$ 73.40	$ 904,400.00
	8" Gravity Line	LF	1	2,705	$ 84.40	$ 228,300.00
	10" Gravity Line	LF	2	3,762	$ 95.40	$ 358,800.00
	12" Gravity Line	LF	1	2,819	$ 106.40	$ 300,000.00
	15" Gravity Line	LF	1	1,713	$ 127.40	$ 218,200.00
	8" Forced Main	LF	1	1,678	$ 39.05	$ 65,500.00
	Lift Station - 1700 GPM	LS	1		$ 420,000.00	$ 420,000.00
						$ 1,681,300.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area B	6" Gravity Line	LF	10	14,180	$ 73.40	$ 1,040,800.00
	8" Gravity Line	LF	7	10,805	$ 84.40	$ 911,900.00
	8" Forced Main	LF	1	2,559	$ 39.05	$ 99,900.00
	Lift Station - 600 GPM	LS	1		$ 290,000.00	$ 290,000.00
						$ 2,342,600.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area C	6" Gravity Line	LF	1	3,017	$ 73.40	$ 221,400.00
	12" Forced Main	LF	1	2,895	$ 51.05	$ 147,800.00
	15" Forced Main	LF	1	190	$ 89.05	$ 17,000.00
	Lift Station - 1900 GPM	LS	1		$ 440,000.00	$ 440,000.00
						$ 826,200.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area D	6" Gravity Line	LF	1	2,090	$ 73.40	$ 153,400.00
	8" Gravity Line	LF	2	10,951	$ 84.40	$ 924,200.00
	10" Gravity Line	LF	1	4,051	$ 95.40	$ 386,400.00
						$ 1,464,000.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area E	6" Gravity Main	LF	7	12,336	$ 73.40	$ 905,400.00
	8"Gravity Main	LF	2	1,338	$ 84.40	$ 112,900.00
						$ 1,018,300.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area F	6" Gravity Line	LF	3	5,302	$ 73.40	$ 389,200.00
	12" Gravity Line	Lf	1	593	$ 106.40	$ 63,100.00
	8" Forced Main	LF	2	3,393	$ 39.05	$ 132,500.00
	Lift Station - 550 GPM	LS	1		$ 280,000.00	$ 280,000.00
	Lift Station - 600 GPM	LS	1		$ 290,000.00	$ 290,000.00
						$ 1,154,800.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area G	6" Gravity Line	LF	3	8,277	$ 73.40	$ 607,500.00
	8" Gravity Line	LF	1	5,584	$ 84.40	$ 471,300.00
	12" Gravity Line	LF	1	8,389	$ 106.40	$ 892,600.00
	15" Gravity Line	LF	1	1,994	$ 127.40	$ 254,000.00
	10" Forced Main	LF	2	4,793	$ 45.05	$ 215,900.00
	Lift Station - 800 GPM		1		$ 310,000.00	$ 310,000.00
	Lift Station - 1000 GPM		1		$ 340,000.00	$ 340,000.00
						$ 3,091,300.00

Study Area	Item Description		Units	Quantity (LF)	Unit Price	Cost
Study Area H	6" Gravity Line	LF	5	9060	$ 73.40	$ 665,000.00
	8" Gravity Line	LF	1	4199	$ 84.40	$ 354,400.00
	10" Gravity Line	LF	4	6403	$ 95.40	$ 610,800.00
	6" Forced Main	LF	1	3501	$ 31.05	$ 108,700.00
	Lift Station - 325 GPM	LS	1		$ 260,000.00	$ 260,000.00
						$ 1,998,900.00

11. References

City of Jonestown. 2014. Code of Ordinances. Jonestown, Tx.; City of Jonestown, Tx. Available at: http://z2.franklinlegal.net/franklin/Z2Browser2.html?showset=jonestownset. Accessed 6 March 2015.

Davis, M. L. 2010. *Water and Wastewater Engineering: Design Principles and Practice*. New York, N.Y.: McGraw-Hill Education.

Edwards, J et al. 2007. *Handbook for Vulnerability Mapping: EU Asia Pro Eco Project*. Sweden, EU. Available at: http://www.unep.fr/shared/publications/pdf/ANNEXES/3.2.4%20Risk%20assessment%20and%20vulnerability%20maps/Handbook%20for%20Vulnerability%20Mapping.pdf. Accessed 23 February 2015.

Espey Consultants, Inc. 2006. City of Jonestown Comprehensive Plan Update, 2005-2015. EC Project No. 5023. Austin, Tx: Espey Consultants, Inc.

Goldscheider, N., Klute, M., Sturm, S., & Hötzl, H. (2000). The PI method–a GIS-based approach to mapping groundwater vulnerability with special consideration of karst aquifers. *Zangew Geol*, *46*(3), 157-166.

Interlocal Cooperation Agreement for Wholesale Water and Wastewater Utility Services by and between the City of Leander and the City of Jonestown. 2014.

Interlocal Wholesale Wastewater Agreement by and between the City of Lago Vista and the City of Jonestown. 21 June 2013.

Lacey, Linda. "Lesson 4: Use of Census and Related Population Information." Lesson 4: Use of Census and Related Population Information. M&E Learning Center, USAID, n.d. Web. 25 Apr. 2015.

Lacey, Linda. "Lesson 8: The Cohort Component Population Projection Method." Lesson 4: Use of Census and Related Population Information. M&E Learning Center, USAID, n.d. Web. 25 Apr. 2015.

LCRA. 2007. Highland Lakes Watershed Ordinance. Water Quality Management Technical Manual. 5th ed. Austin, Tx.: Lower Colorado River Authority. Available at: www.lcra.org. Accessed 6 March 2015.

LCRA. 2014. Parks Land and Water Use Regulations. Austin, Tx.: Lower Colorado River Authority. Available at: www.lcra.org. Accessed 15 April 2015.

Rahman, Atiqur. 2008. A GIS based DRASTIC model for assessing groundwater vulnerability in shallowing aquifer in Aligarh, India. *Applied Geography.* Vol 28, Issue 1: 32-53.

TCEQ. 2008. Chapter 217: Design Criteria for Domestic Wastewater Systems. Effective 28 August 2008. Austin, Tx.; Texas Commission on Environmental Quality. Available at: http://www.tceq.texas.gov/rules/indxpdf.html/#217. Accessed 6 March 2015.

TCEQ. 2009. Chapter 309: Domestic Wastewater Effluent Limitation and Plant Siting. Most recent edition effective 26 November 2009. Austin, Tx.; Texas Commission on Environmental Quality. Available at: http://www.tceq.texas.gov/rules/indxpdf.html/#309. Accessed 6 March 2015.

TCEQ. 2015. Water Well Report Viewer. Texas Commission on Environmental Quality. Available at: http://www.tceq.texas.gov/gis/waterwellview.html. Accessed 6 March 2015.

USEPA. 1987. DRASTIC: A Standardized System for Evaluating Groundwater Pollution Potential Using Hydrogeologic Settings. Ada, Ok. Available at: http://www.epa.gov/region5/waste/clintonlandfill/PDFClintonLFChemicalWaste_USEPAApplication/cl_130.pdf. Accessed 23 February 2015.

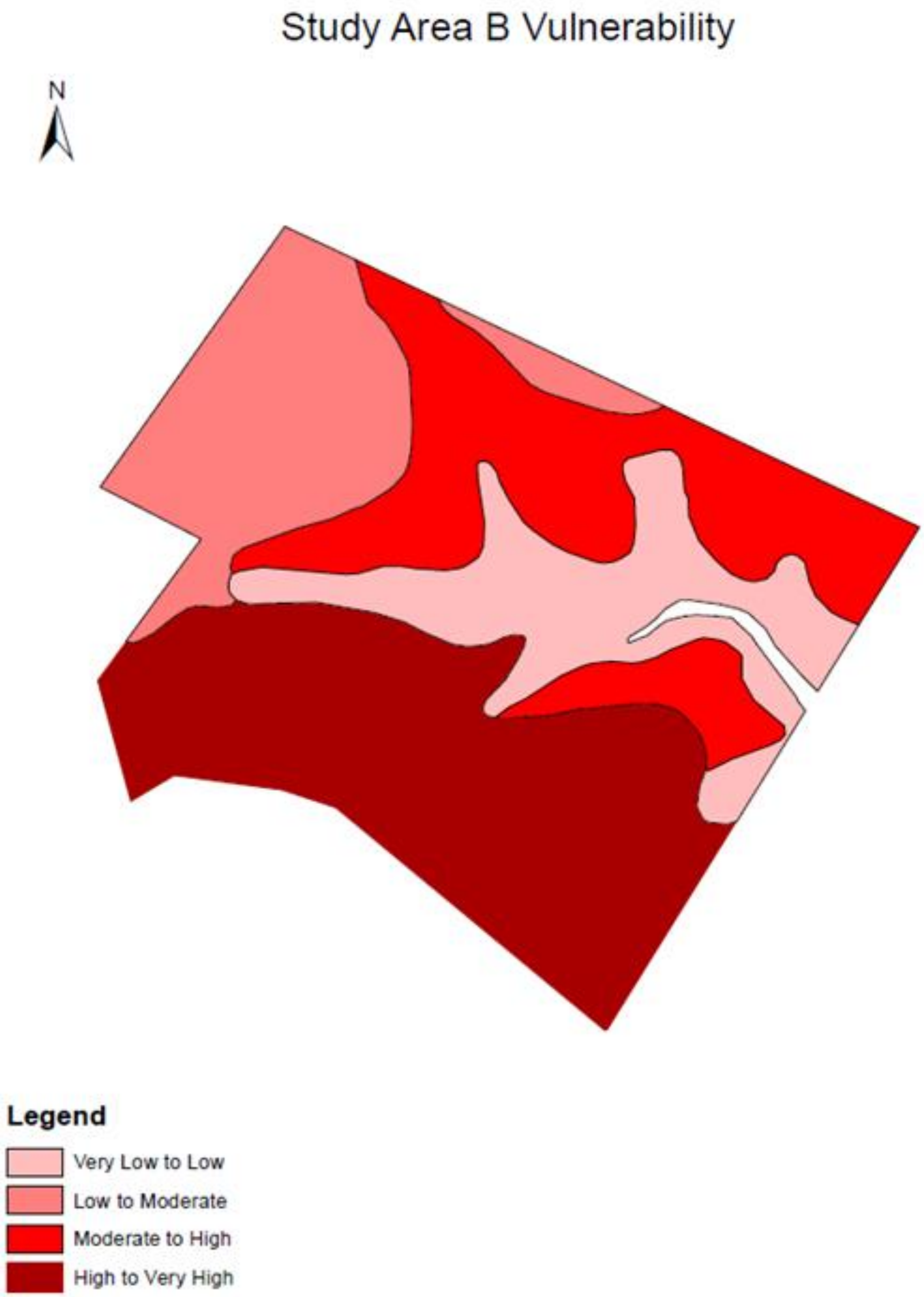
USEPA. 1999. Collection Systems Technology Fact Sheet: Sewers, Pressure. USEPA – Office of Water. Available at: http://water.epa.gov/scitech/wastetech/upload/2002_10_15_mtb_presewer.pdf

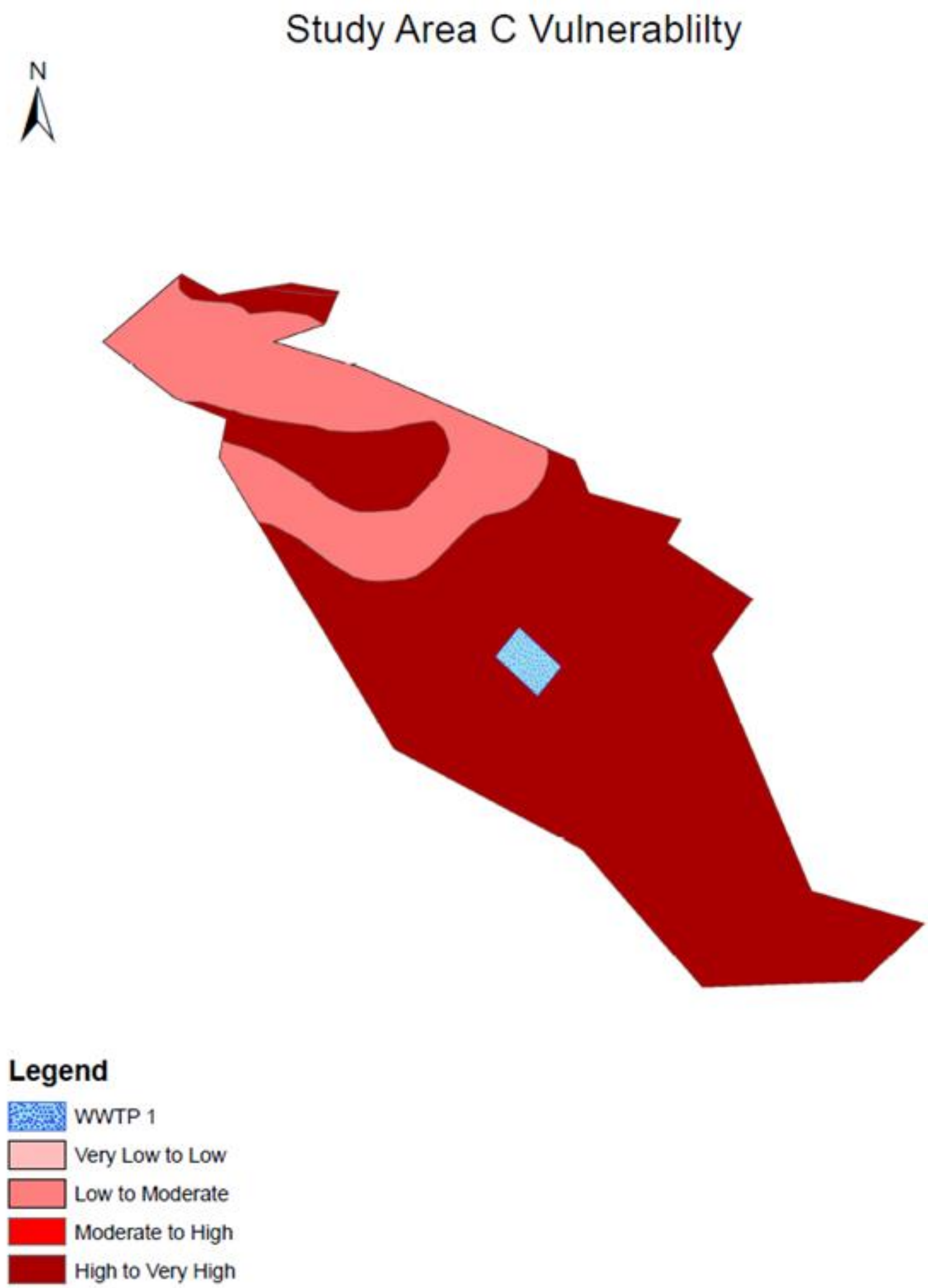
USEPA. 2000. Collection Systems Technology Fact Sheet: Sewers, Force Main. USEPA – Office of Water. Available at: http://water.epa.gov/scitech/wastetech/upload/2002_06_28_mtb_force_main_sewers.pdf

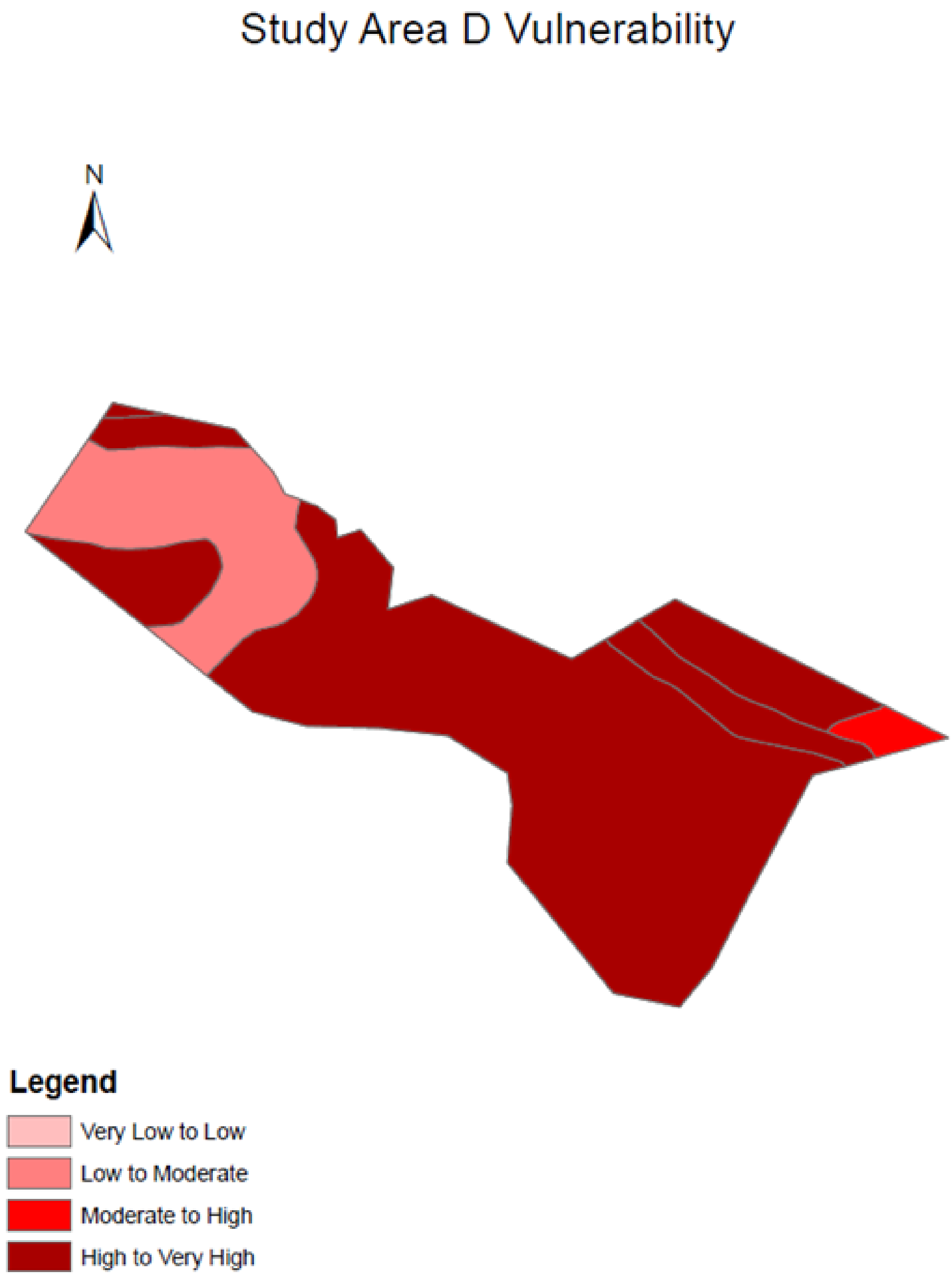
USEPA. 2002. Collection Systems Technology Fact Sheet: Sewers, Conventional Gravity. USEPA – Office of Water. Available at: http://water.epa.gov/scitech/wastetech/upload/2002_10_15_mtb_congrasew.pdf

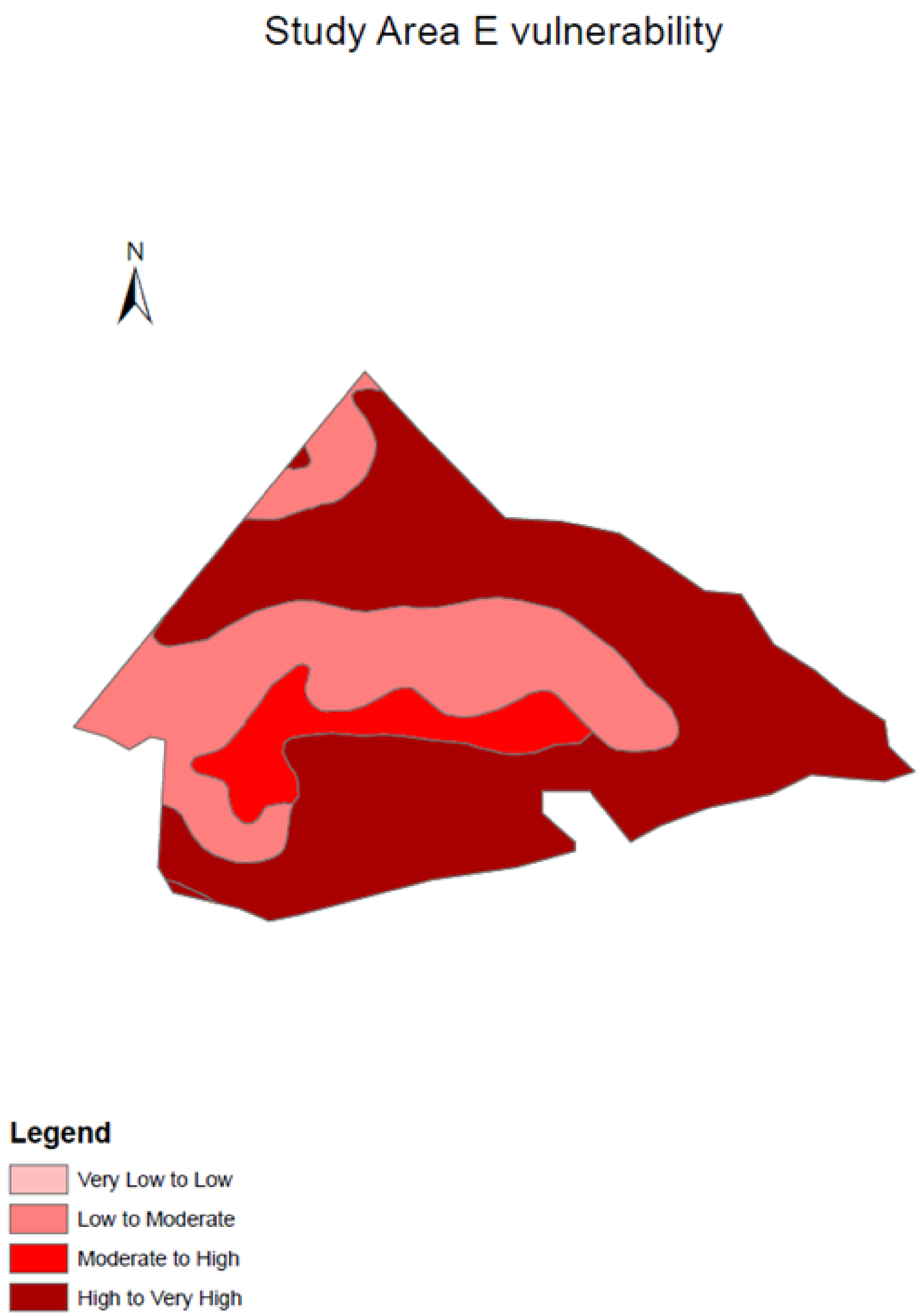
12. Appendices

A. Vulnerability Maps

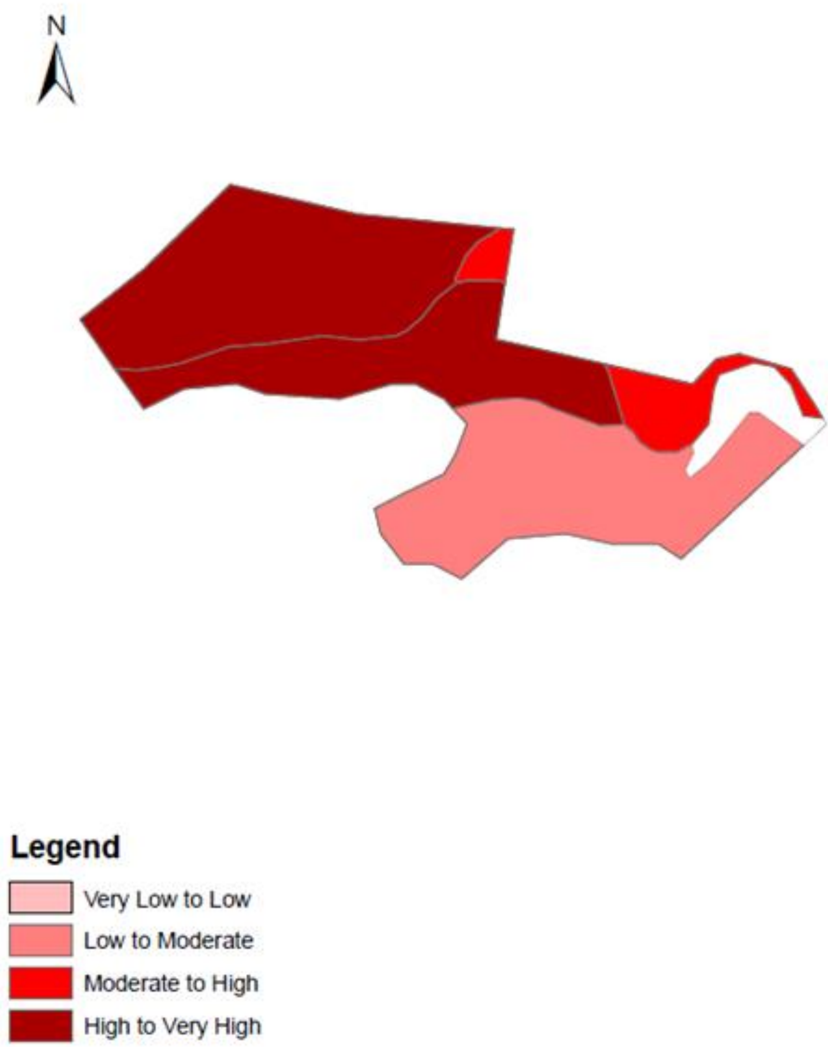


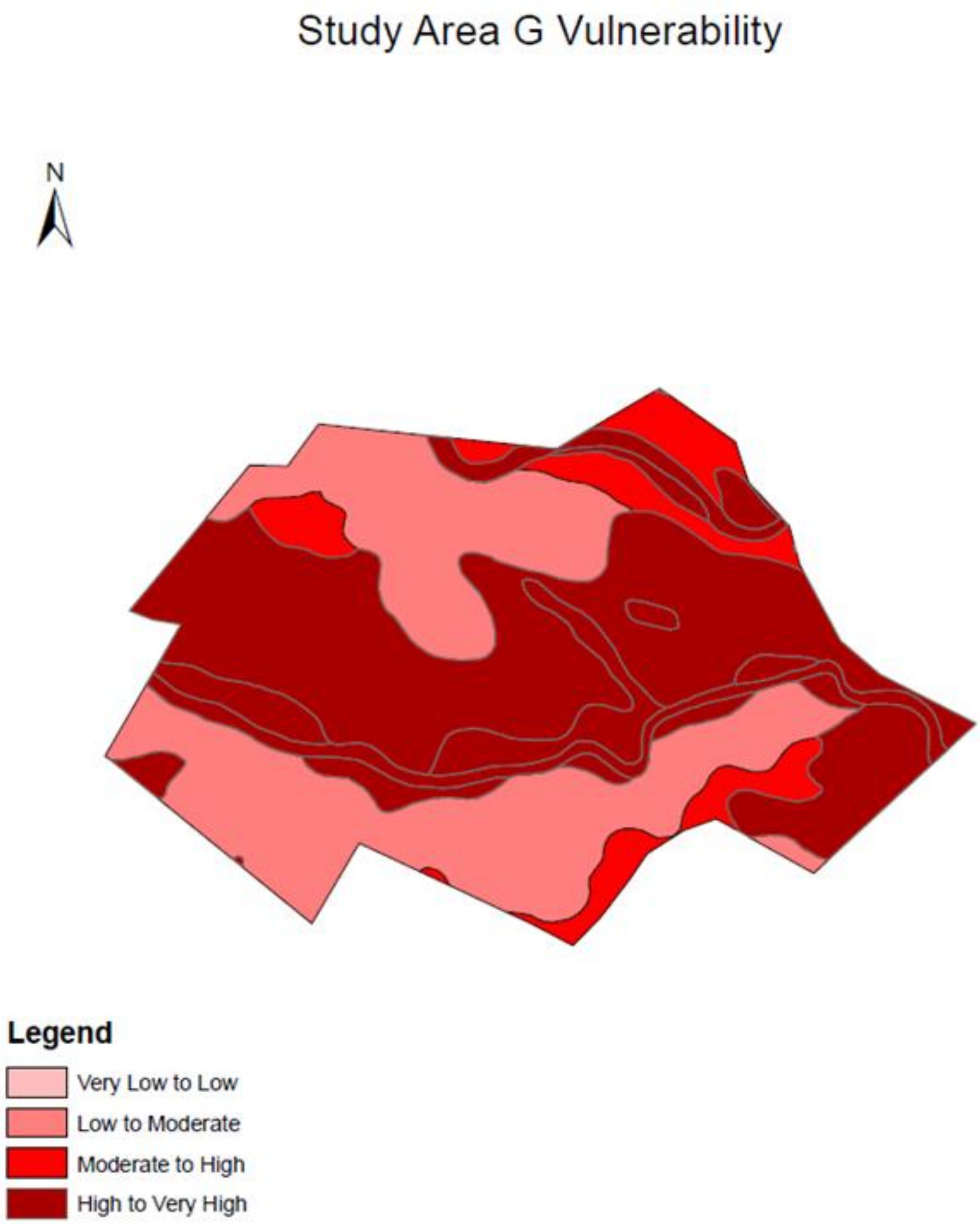

Study Area C Vulnerablilty
N
Legend
WWTP 1
Very Low to Low
Low to Moderate
Moderate to High
High to Very High

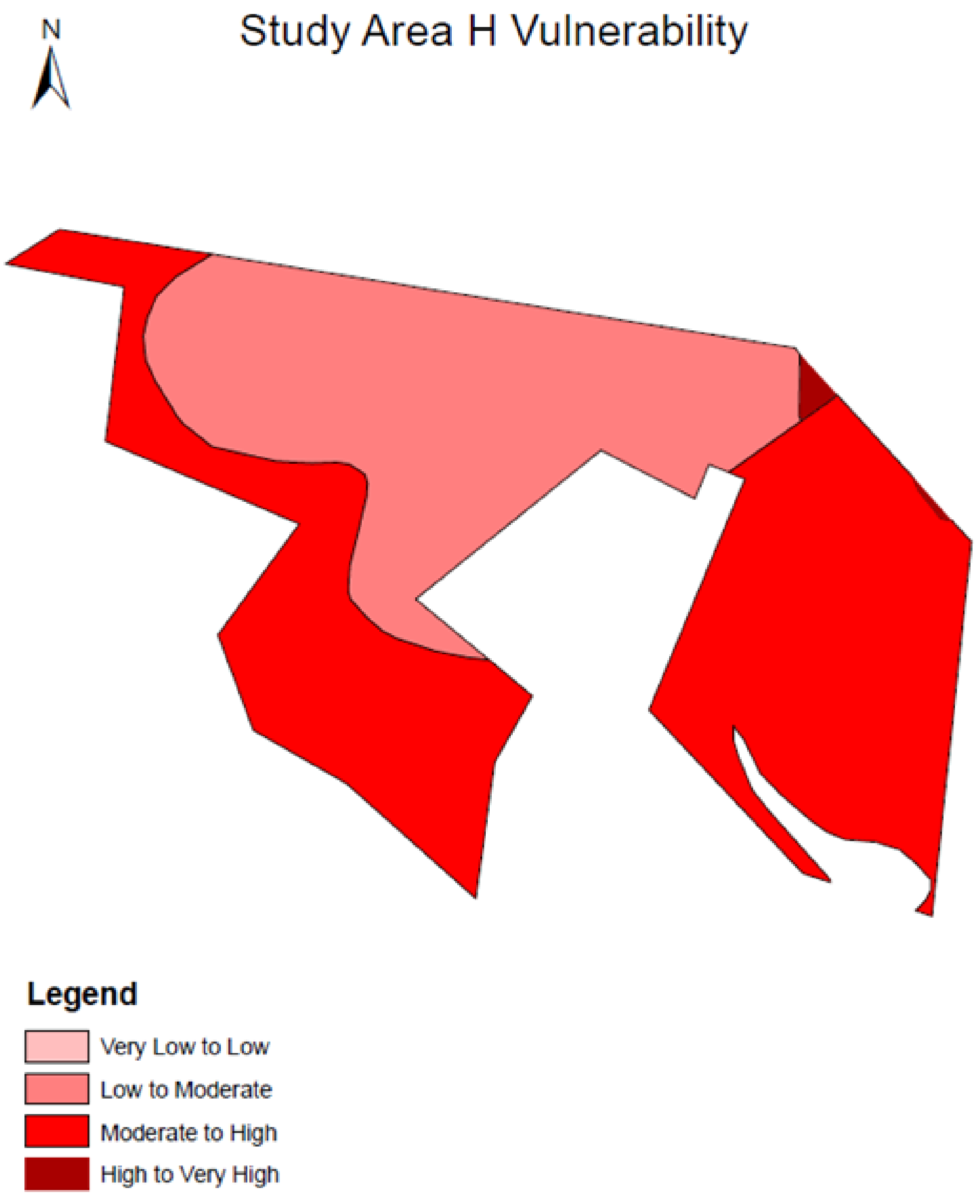

Study Area D Vulnerability
N
Legend
Very Low to Low
Low to Moderate
Moderate to High
High to Very High


Study Area E vulnerability
N
Legend
Very Low to Low
Low to Moderate
Moderate to High
High to Very High

Study Area F Vulnerability




Study Area G Vulnerability
N
Legend
Very Low to Low
Low to Moderate
Moderate to High
High to Very High


Study Area H Vulnerability
N
Legend
Very Low to Low
Low to Moderate
Moderate to High
High to Very High

B. Design Spreadsheets

Wastewater Capacity Modeler

Wastewater Line Description		Wastewater Line Data						
Line Name	Line No.	No. of LUE's Served	No. of Acres Served	Pipe Dia. (in)	Mann-ing's n	Min. Pipe Slope (ft/ft)	Pipe Area (sq.ft.)	Hyd. Radius (ft)
Maximum LUE Capacity								
6" Gravity Line		323	73.0	5.5	0.013	0.0060	0.16	0.11
8" Gravity Line		323	73.0	7.75	0.013	0.0050	0.33	0.16
10" Gravity Line		323	73.0	9.5	0.013	0.0050	0.49	0.20
12" Gravity Line		323	73.0	11.54	0.013	0.0050	0.73	0.24
15" Gravity Line		323	73.0	14.12	0.013	0.0050	1.09	0.29
18" Gravity Line		323	73.0	17.26	0.013	0.0050	1.62	0.36

Flows and Peaking Factor			Capacity Calculations			Design Flows		
Full Flow Vel. (fps)	Avg. Dry Weather Flow (gpm)	Peaking Factor	Max. Cap. Q_{full} (gpm)	PDWF(peak dry weather flow) Cap. $Q65_{\%}$ (gpm)	PWWF(peak wet weather flow) Cap. $Q8_{5\%}$ (gpm)	Peak Dry Weather Flow (gpm)	Inflow and Infiltr-ation (gpm)	Peak Wet Weather Flow (gpm)
2.09	45	3.82	**155**	101	131	172	38	**210**
2.40	45	3.82	**352**	229	300	172	38	**210**
2.75	45	3.82	**606**	394	515	172	38	**210**
3.13	45	3.82	**1,019**	662	866	172	38	**210**
3.58	45	3.82	**1,745**	1,134	1,483	172	38	**210**
4.09	45	3.82	**2,980**	1,937	2,533	172	38	**210**

Lift Station Design

JAY ENGINEERING COMPANY, INC.
P.O. Box 1220 (512) 259-3882
Leander, TX 78646 Fax 259-8016

Phase B-1 (Ult)
Lift Station - 1

JAECO Job No.: 193-042-20

Flow Development :

Average Dry Weather Flow (ADWF):

Gravity LUE's =	**2533**	
Total Number of LUE's =	2533	
Average Dry Weather Flow per LUE =	**200**	gallons/day
Total Average Dry Weather Flow =	**506,600**	**gallons/day**
Total Average Dry Weather Flow =	**352**	**gallons/minute**

Peak Dry Weather Flow (PDWF):

Peaking Factor = $\dfrac{18+(0.0206 \cdot ADWF_{gpm})^{0.5}}{4+(0.0206 \cdot ADWF_{gpm})^{0.5}}$ =	3.09	
Total Peak Dry Weather Flow =	**1,088**	**gallons/minute**

Peak Wet Weather Flow (PWWF):

Inflow and Infiltration =	**750**	gpd/acre
Existing Service Area =	**0**	acres
Service Area =	**1067**	acres
Total Inflow and Infiltration =	556	gallons/minute
Total Peak Wet Weather Flow =	**1,644**	**gallons/minute**

Wet Well Design :

Minimum Wet Well Working Volume

Enter pump capacity =	1700	gallons/minute	Minimum Pump Cycle Times
			6 minutes for less thatn 50 hp
Enter minimum cycle time =	6	if minimum =	10 minutes for 50 to 100 hp
			15 minutes for greater than 100 hp

Minimum Wet Well Working Volume = 2550

Wet Well Detention Time

Pump Capacity,	q =	1700	gallons/minute
Using Average Dry Weather Flow, ADWF	i =	352.00	gallons/minute
	(q - i) =	1348.00	net outflow in gallons per minute
Using Peak Dry Weather Flow, PDWF	i =	1088	gallons/minute
	(q - i) =	612	net outflow in gallons per minute
Using Peak Wet Weather Flow, PWWF	i =	1644	gallons/minute
	(q - i) =	56	net outflow in gallons per minute

10	-foot Dia.	Time to Fill Wet Well, $T_f = v / i$ (min)			Time to Empty Well, $T_e = v / (q-i)$ (min)			Wet Well Detention Time $T_d = T_f + T_e$ (min)		
Wet Well Depth (ft)	Working Volume, v (gal.)	MDW Flow (min.)	PDW Flow (min.)	PWW Flow (min.)	MDW Flow (min.)	PDW Flow (min.)	PWW Flow (min.)	MDW Flow (min.)	PDW Flow (min.)	PWW Flow (min.)
3.5	2,056	6	2	1	2	3	37	8	5	38
4	2,350	7	2	1	2	4	42	9	6	43
4.5	2,644	8	2	2	2	4	47	10	6	49
5	2,937	8	3	2	2	5	52	10	8	54
5.5	3,231	9	3	2	2	5	58	11	8	60
6	3,525	10	3	2	3	6	63	13	9	65

Discharge Piping and Force Main Design:

Headloss Due to Friction and Minor Losses :

Discharge Pipe Material =	DI		Force Main Material =	PVC	
Discharge Pipe Length =	30	feet	Force Main Length =	2,895	feet
Discharge Pipe Diameter =	6	inches	Force Main Diameter =	12	inches
Roughness Coefficient, c =	130		Roughness Coefficient, c =	130	
Flow in Discharge Pipe =	1700	gallons/minute	Flow in Force Main =	1700	gallons/minute
Velocity in Discharge Pipe =	19.29	feet/second	Velocity in Force Main =	4.82	feet/second
f =	19.87	feet/100 ft of pipe	f =	0.68	feet/100 ft of pipe

	#	K	# x K
# of 90 deg. bends =	3	0.34	1.00
# of 45 deg. bends =	0	0.25	0.00
# of 22.5 deg. bends =	0	0.20	0.00
# of tees (branch flow) =	1	1.02	1.00
# of Swing CVs =	1	0.85	0.90
# of Gate Valves =	1	0.14	0.10
		Sum of K's =	3.00

	#	K	# x K
# of 90 deg. bends =	0	0.30	0.00
# of 45 deg. bends =	0	0.22	0.00
# of 22.5 deg. bends =	1	0.18	0.20
# of Exit Losses =	1	1.00	1.00
		Sum of K's =	1.20

Head Loss Curves:

Pump vs. System Curve with C = 100

	Discharge Piping			Force Main					
System Flow (gpm)	Velocity (fps)	Friction Losses (ft)	Minor Losses (ft)	Velocity (fps)	Friction Losses (ft)	Minor Losses (ft)	Max. Static Head (ft)	Total Discharge Head (ft)	Selected Pump Discharge Head (ft)
900	10.21	2.99	4.86	2.55	9.88	0.12	79.50	97.35	90.2
1100	12.48	4.33	7.26	3.12	14.33	0.18	79.50	105.59	89.6
1300	14.75	5.90	10.14	3.69	19.51	0.25	79.50	115.30	89
1500	17.02	7.68	13.50	4.26	25.43	0.34	79.50	126.44	88.5
1700	19.29	9.68	17.34	4.82	32.05	0.43	79.50	139.01	87.9
1900	21.56	11.90	21.66	5.39	39.38	0.54	79.50	152.97	87.3
2100	23.83	14.32	26.46	5.96	47.38	0.66	79.50	168.32	86.7
2300	26.10	16.94	31.73	6.53	56.07	0.79	79.50	185.04	86.1
2500	28.37	19.77	37.49	7.09	65.42	0.94	79.50	203.12	85.6
2700	30.64	22.79	43.73	7.66	75.43	1.09	79.50	222.55	84.9

Pump vs. System Curve with C = 130

	Discharge Piping			Force Main					
System Flow (gpm)	Velocity (fps)	Friction Losses (ft)	Minor Losses (ft)	Velocity (fps)	Friction Losses (ft)	Minor Losses (ft)	Max. Static Head (ft)	Total Discharge Head (ft)	Selected Pump Discharge Head (ft)
900	10.21	1.84	4.86	2.55	6.08	0.12	79.50	92.40	90.2
1100	12.48	2.66	7.26	3.12	8.82	0.18	79.50	98.42	89.6
1300	14.75	3.63	10.14	3.69	12.01	0.25	79.50	105.53	89
1500	17.02	4.73	13.50	4.26	15.65	0.34	79.50	113.71	88.5
1700	19.29	5.96	17.34	4.82	19.73	0.43	79.50	122.96	87.9
1900	21.56	7.32	21.66	5.39	24.23	0.54	79.50	133.25	87.3
2100	23.83	8.81	26.46	5.96	29.16	0.66	79.50	144.59	86.7
2300	26.10	10.43	31.73	6.53	34.51	0.79	79.50	156.96	86.1
2500	28.37	12.16	37.49	7.09	40.26	0.94	79.50	170.36	85.6
2700	30.64	14.03	43.73	7.66	46.43	1.09	79.50	184.78	84.9