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Simplified Sewerage Systems and Potential Application to Rural Louisiana Communities

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SIMPLIFIED SEWERAGE SYSTEMS AND POTENTIAL APPLICATION TO RURAL LOUISIANA COMMUNITIES

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Abstract

Today's rapid growth in population on sub-urban areas has caused an increase in fecal coliforms to be discharged to down-streams and lakes. The unaffordability of conventional sewerage systems makes it inaccessible to big part of the population of rural communities to receive proper sanitation. Since population growth is only projected to increase during the next few years, action is necessary to change the course of the future of rural communities, more specifically described on this report, Louisiana's communities. One option to offer affordable sewerage systems to the Louisiana region is that we utilize of the Simplified Sewerage technology. Simplified Sewerage has been tested on several municipalities around the globe, especially in Brazil. The results of its application have proven to be conclusive and beneficial to communities.

Keywords: Condominial Systems, Cost Efficient, Sanitation, Simplified Sewerage, St. Tammany Parish.

Introduction

The condominial sewerage system is an approach to expand sewerage networks to sub-urban and peri-urban areas that was first developed in Brazil in the 1980's. This model has proven itself very effective on the cities it has been implemented and the intention of this study is to analyze past implementations of these systems and possibly provide an alternative for use of these methods on rural Louisiana communities (Mara, 2001).

The collection systems account for 80-90 percent of the total cost of the conventional sewerage (Kreissl1987). That being so, design criteria and standards can be carefully modified to achieve cost savings from the use of shallower depths, smaller pipe diameters, fewer appurtenances, and other system simplifications.

This report gives the background of simplified sewerage and compares the processes of collection and transportation of wastewater using the conventional sewerage methods and the more modern simplified method. The objective of this study is to expand the local knowledge on Simplified Sewerage and the consideration of implementing such systems in Louisiana.

Body of Thesis

Theory Related to Design

Simplified sewerage is an off-site sanitation technology that removes all wastewater from the household environment. Conceptually it is the same as conventional sewerage, but with conscious efforts made to eliminate unnecessarily conservative design features and to match design standards to the local situation.

The basic idea of a simplified sewerage system is that households are to be grouped into a neighborhood unit, also called a “condominium.” This condominium works in a similar way to how an apartment building does. Instead of every house having their own unique sewerage system, this unit of houses reduces the amount of pipe length in about half because the public network does not need to run through every plot of land or be present on every street.

The basics of a wastewater collecting system are: Water goes from the smaller diameter piper called collectors, flows to “trunk lines”, and finally into a massive system of pipes and pump stations.

Some of the criteria and standards for the design of the sewers, which are later mentioned in more detail on this report’s guidelines, are:

- Minimum velocity in pipe
- Minimum slopes of pipes
- Minimum pipe diameter
- Design peak flow factor

For any sewerage system, to identify which system is to be implemented, it is necessary to know if the wastewater is settled or non-settled (Akinyemi, 2001). The conventional systems have different design criteria depending on its geographic location, water capacity, among other factors. In certain parishes in Louisiana, for instance, standard sanitary systems use a minimum flow velocity when the sewer pipe is running full of 0.6 m/s to ensure self-cleansing and a minimum pipe diameter of 200 mm (8 inn), whereas most conventional systems around the country use a minimum given slope for the minimum flow (Jefferson Parish Public Improvements Standards Manual, 2008).

The Models:

A major convenience of condominial branches is that the sewer system connections can be in the most convenient part of the block (unit). This design also permits the networks to be adapted to the local topographic conditions and soil patterns. The different alternatives of simplified sewer connection, listed from the one that generates the most savings to the least, are the following:

- **Backyard:** this type of pipeline placement has the advantage of being significantly cheaper to build due to shorter lengths and shallower depths of excavation. However, the potential disadvantage is that they may be inaccessible to utility company staff, assigning the responsibility of maintenance to the household members.
- **Front yard:** Similarly, this is another economical type that can be widely used in condominial branches. The costs are slightly higher than backyard connections but have a more strategic placement of pipes that facilitates maintenance.
- **Sidewalk:** commonly used in condominium branches, will usually generate less design cost savings since they may have a single-entry point for each housing provided. The benefit of this pipe placement is on the easy access for public maintenance (Melo, 2005).

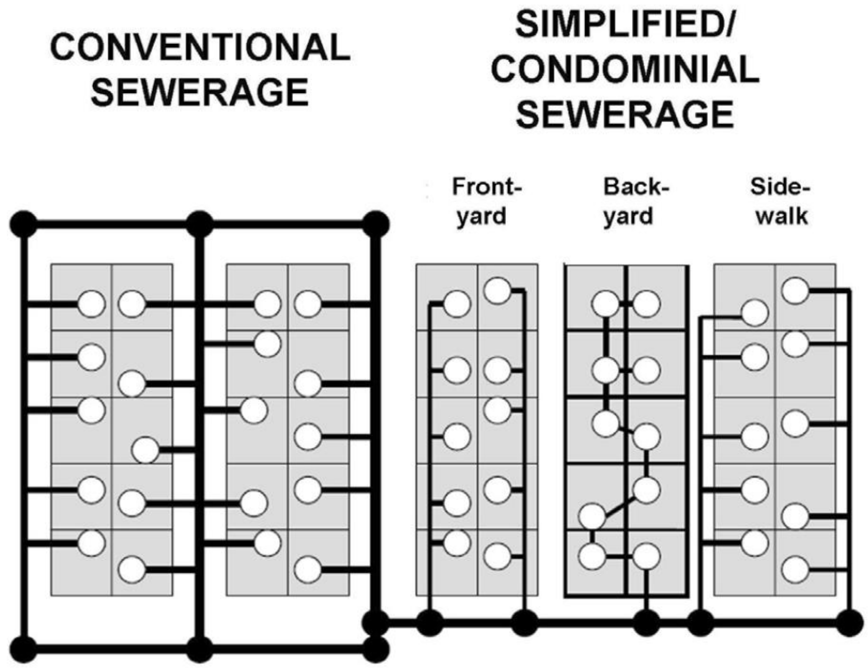

Figure 1: Conventional and Simplified sewer models' pipelines side by side (Mara, 2001)

The figure above illustrates the three models of simplified sewerage systems and places it side-by-side to the conventional system. From the figure it is possible to see that the length of pipes is much shorter for the simplified models and can also be smaller in diameter as later demonstrated on this report.

To ensure self-cleansing, instead of the minimum velocity criterion of 0.6 meters per second as in conventional sewer design, simplified sewer design is based on maintaining a boundary shear stress of 1 Pa, which is sufficient o resuspend a 1-mmp particle of sand. Many authors (Machado1985; Paintal 1977; Yao 1974, 1976) have proposed the use of critical shear stress for determining the minimum slope of sewers as an economical alternative to the minimum velocity approach. For a minimum shear stress of 1 Pa, pipes smaller than 1,050-mm can be made flatter than when designed according to the minimum-velocity approach, and pipes larger than 1,050-mm should be made steeper to maintain self-cleansing.

Based on the following assumptions: critical unit tractive force = 1 Pa; relative depth of flow, y/D = 0.2; Manning's n = 0.013. In Brazil, for design of simplified sewers, the following equation is used:

$$I = 0.0055\ Q^{-0.47} \text{ (Equation 1.1)}$$

Where I, is the minimum slope of the sewer, and Q is the initial flow in liters per second (current flow). (Bakalian, 1994)

Sewerage Model Types

Among the many models implemented around the globe, it is possible to find a wide range of modifications that result on improvements and cost savings of sewerage systems. Some of the most popularly used sewerage models can be classified by the following categories:

- **Conventional Gravity Sewerage:** For larger diameter sewers the minimum slope is inversely proportional to diameter. The main disadvantage is the slope requirements, which to be satisfied on virtually plane areas such as Louisiana, require lift stations, significantly increasing costs.

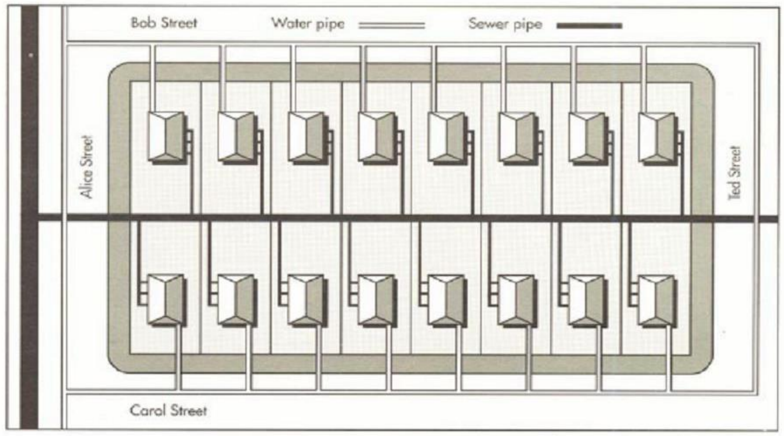

Figure 2: Layout of a Typical Conventional Gravity Sewer (Source: Dr. Eddy Akinyemi -ppt)

- **Simplified Sewerage:** Consists of shallowly-buried plastic pipes, low-cost cleanouts instead of frequent/costly manholes, and a minimum number (if any) of lift stations. Management requirements are equal or lower than conventional gravity sewers (depends on number of lift stations).

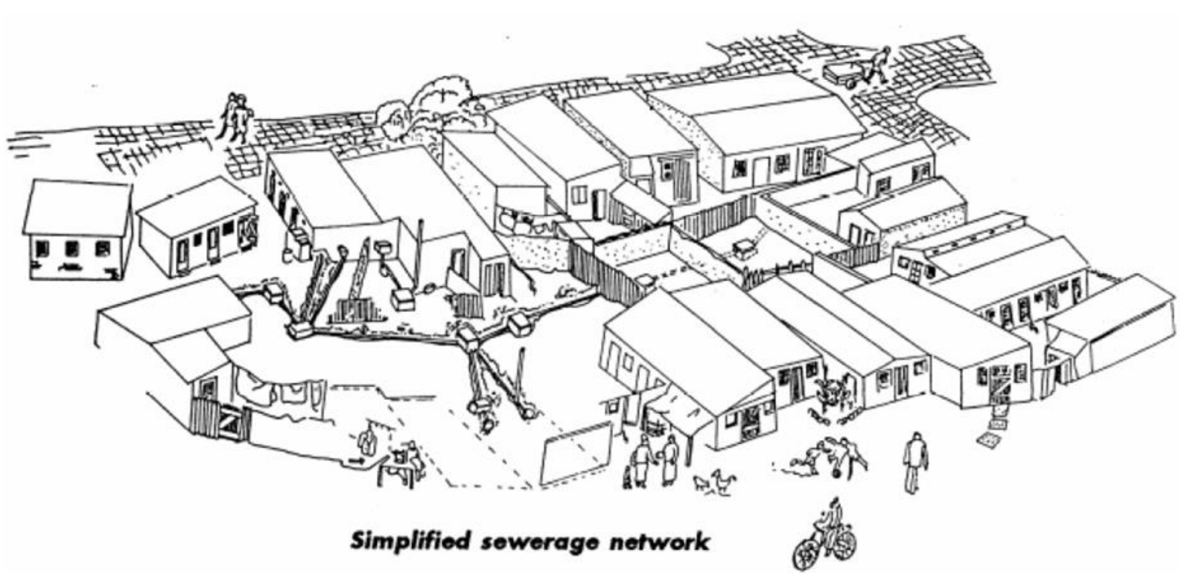

Figure 3: A Simplified Sewer Network (EAWAG/SANDEC, 2008)

On the Simplified Sewerage Network illustrated above it is possible to see the many cleanouts, which are small inspection boxes where maintenance is done. Notice that sewers are laid within property boundaries rather than beneath central roads.

- **Small-Bore Gravity Sewerage (EPA manual EPA/625/I-91/024):** It has many similarities to the simplified sewerage with the main difference being that the wastewater is first settled in a septic tank at household level.

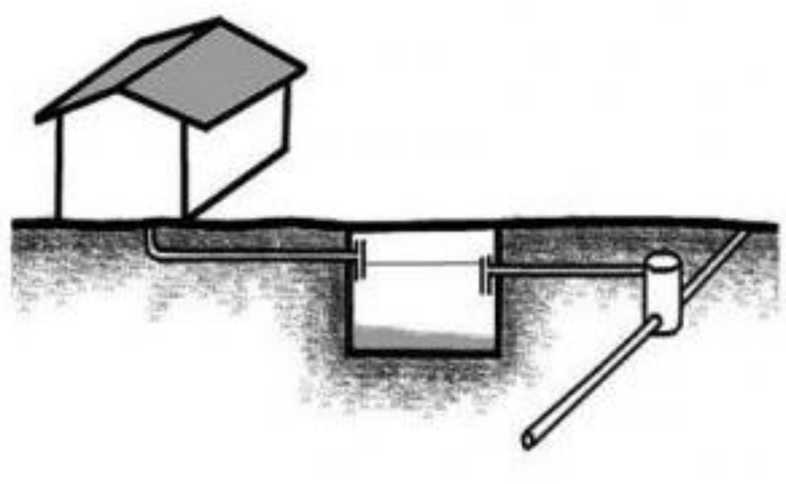

Figure 4: Septic Tank Between Household and Remaining Pipelines of System (EAWAG, 2014)

- **Vacuum:** The vacuum or draw within the system is created at a vacuum station. Vacuum stations are small buildings that house a large storage tank and a system of vacuum pumps.
- **Pressurized Systems:** Wherever space constraints prevented a traditional gravity-based collector from being installed, pressurization of the sewerage system was used as an alternative. It employs cleanouts instead of manholes as access points for cleaning and monitoring the lines. Cleanout points, as it will be discussed later, are very economical.
- **Vertical Drop Pipes:** There was a case of an area whose natural outlet consisted of a ravine approximately 20 meters deep, the bottom of which was occupied by an informal settlement.

The solution to this problem was to use a vertical drop pipe consisting of a 150-millimeter cast iron pipe fixed to the wall of the ravine with clamps, protected by concrete casing and ending in a pressure-break box at the base, which also served as an initial manhole.

- **Aerial:** An extreme case of the approach had to be used in cases where urban density was very high and water runoff was not adequately covered or channeled. In these cases, flooding is usually prevented by placing the lower floors of the houses above the runoff level. This normally enables the installation of network branches bordering the houses, above the water runoff channels, either by using pipes attached to the house walls or supported on pillars.

Construction Procedures

GUIDELINES

Note: These recommendations are made from the point of view of enabling further improvements at the time of planning, designing and operations of the simplified sewerage systems. The recommendations are by no means comprehensive but are expected to improve success rate and overall performance of the systems (Ramagundam, 2017).

1. Layout Basics:

The networks of sewerage pipelines are split into separate smaller systems. Where feasible, each determined area is defined by individual drainage basins, each with its own collectors and treatment plant.

To minimize excavation, and thus the cost of pavement restoration, sewers are located away from the traffic loads, generally under the sidewalks rather than down the center of the street. To save even more on costs, on many cases the sewers extend only to the last upstream connection rather than the end of the block (Bakalian, 1994).

2. Hydraulics:

Another approach to sewerage systems that can bring major benefits to the project is to reduce the design period of the sewerage system. Conventional sewerage systems have long design periods of 50 years, usually done this way considering the saturation population of the area. Planning for the long run is important when designing a city but the design can be made for separate short-term periods. A great advantage of using shorter term periods is that it avoids uncertainties of population growth and reduces the high costs of maintenance of large sewer systems with low flow. Other benefits of the reduced design period, are that it can also facilitate financing and achieve greater coverage with the same investment.

With shorter design periods and construction phases, starting from upstream ends, the effects of errors in forecasting population growth and their water consumption can be minimized and corrected. For these reasons, simplified sewerage employs design periods of 20 years or less. In this regard, it is noteworthy that the USEPA limits the design period to 10-15 years (ASCE 1982).

When it comes to the design flow, it is important to collect records of pumpage for each day and fluctuations during the day. Keep in mind that the design flow capacity of sewers is less than the water supply design since water is lost on the process through leaking, garden watering, house cleaning, among other uses.

The peak factor, often conservatively estimated between 2.0 and 3.3, has been modified to 1.8 on simplified sewerages implemented in Brazil and Colombia (Bakalian, 1994).

Sewer pipe diameters of 100-mm have been used in Brazil for a maximum length of 400m. The use of this pipe diameter on residential areas generates a greater depth of flow and improves self-cleansing. Past implementations have show that, where the pipe diameter on the street sewers was reduced from 200-mm (used as a minimum diameter on conventional systems on some Louisiana parishes) to 150-mm, do not present additional maintenance problems compared to conventional sewers (Bakalian, 1994).

3. Service Connection:

In the simplified design, a 60-cm connection (or inspection) box is placed between the building and the service line. All the sewers or drains from the house or building enter the box. This box is usually located under the sidewalk in the public right of way.

4. Depth of Sewers:

For any type of sewer connection, it is standard to have a minimum depth in which the pipes are laid should be sufficient to make house connections and have a layer of soil over the crown to protect the pipe against structural damage from external loads and frost.

On simplified connections, the minimum sewer depths are usually much shallower than the conventional systems. Being as shallow as 0.65-m below sidewalks, 0.95-1.50-m below residential streets (depending on distance from the centerline of street), and 2.5-m below heavily traveled streets.

5. Manholes and Other Appurtenances:

One of the most important differences between conventional and simplified sewer systems is that the former utilizes many manholes, whereas the latter type avoids its use as much as possible. The conservative criteria for manhole use contributes to the high cost of sewerage. The use of shallower depths is one way of reducing these costs.

In conventional systems, manholes are generally located at:

- the upper ends of all laterals
- changes in direction and slope
- pipe junctions, except building connections

- at intervals not greater than 100m for pipes up to 600mm diameter, and at less than 120m for sewers between 700mm and 1,200mm diameter.

In the United Kingdom the distance between manholes has been changed from 110m to 180m (Escritt and Haworth 1984); however, for the Cairo sewerage project in the late 1970s as little as 35m between manholes was proposed for sewers less than 250mm (Taylor & Sons, Binnie and Partners 1977).

Where possible, conventional manholes are replaced with “simplified” manholes, or buried boxes, and manholes are used only at major junctions. The need for maintenance personnel to enter the manholes is eliminated by the shallower depths and the availability of modern hydraulic cleaning equipment; making it possible to reduce the diameter of the manholes to 0.6-0.9m instead of the conventional 1.5 meter.

At changes of direction or slope, the manholes are entirely replaced by simple underground boxes or chambers.

House connections are adjusted to serve as inspection devices as well; a small box is built under the walkway and connected to the sewer with a curve of 45 degrees and a "Y" (the cleaning rod is introduced through this box).

These guidelines for the design of manholes considerably lower the costs of the system, especially since up to 90 percent of manholes are never opened. In 1881 Waring wrote, "It seems to me decidedly advantageous to use inspection pipes, or even lamp holes on 6-inch and 8-inch sewers, rather than build manholes and inspection chambers" (USEPA 1986).

There are situations, however, where manholes should not be eliminated:

- very deep sewers (more than 3 m)

- slopes smaller than required
- sewers with drops
- points of connections from certain commercial and industrial establishments, (i.e., points of sampling and flow measurements)

6. Project Specific Design:

Although the basics of simplified sewerage are similarly applied everywhere, some factors are going to differ from a location to another. For instance, estimates of water use are dependent of the return coefficient, which varies according to each location's humidity in the air.

Successful Projects

Although the Simplified Sewerage technology was only developed a couple decades ago, it has proven itself to be very efficient. Literature on this topic demonstrates that projects are often successful. The following locations have implemented this technology and are listed along with some information about its projects:

Juiz de Fora, Minas Gerais - Brazil:

The use of simplified sewerage generated huge savings for the city. To treat its waste water, the city of Juiz de Fora (population 400,000) in the state of Minas Gerais plans to build 57 communal septic tanks with anaerobic filters and 17 up-flow anaerobic sludge blanket systems at a total estimated cost of $18 million. The cost of a central conventional treatment plant and the necessary interceptors was estimated at $75 million (Bakalian, 1994).

Brasilia, Federal District – Brazil:

CAESB, the company responsible for the water supply and sewerage for Brasilia, a 2-million people city, capital of Brazil, aims to deliver basic sanitation to 100% of its urban population.

The procedure now adopted by CAESB is the result of more than 8 years of using the condominial system in Brasilia, not as a special project, but as the normal solution employed by the company. During this time the system was continually developing, allowing the city to reach the highest rates of sewage collection service in the whole country.

The methodology proved to be capable of meeting the objectives of the company, allowing it to service a project population of 1,134,574 (with about 121,000 homes already linked to the system), through the installation of 1,328,498m of condominial branches and 667,485m of public networks. The average per capita cost was approximately US $27 (Exchange rate R$1,20 = US1,00). These costs, along with the use of appropriate technology for sewage treatment, have allowed the installation of complete sewerage systems at a cost of about US $65 per inhabitant serviced, which probably represents the lowest cost likely to be found in a public works project. The whole condominial program, along its 8-years life time, was financed by several national or international agencies, using the normal available financing lines which exist for conventional sewerage systems. This task was made easier due to the low costs involved. One must remember that CAESB has got a fast-partial return of the investments cost's, due to the user's payment of the connecting fee, which represents around 25-30% of the cost of the system. This fee is normally paid divided in 10 monthly quotas and allows new investments in the system.

A positive aspect verified during maintenance of the condominial system is that the cost of interventions effected has been lower than that in the conventional system. This is because of the way the condominial system is constructed, where interventions are made more easily, and because of the maintenance technology itself. The experience obtained in Brasilia overturns the general view that condominial systems need more maintenance than conventional ones: rather, they need the same or less. (Nedler, 1998)

Salvador, Bahia – Brazil:

Salvador, the capital of the State of Bahia, is the largest city in northeastern Brazil, with a population of approximately 2.5 million. As Brazil's first capital, Salvador is also one of the country's oldest cities, with an exceptionally rich cultural and historical heritage. Nevertheless, over half of the population lives in unsanitary urban slums.

For Salvador, the use of pressurized sewerage networks, pipes that "cut across" residences, and even large vertical drop pipes, were introduced to the sewerage systems to be the most economical and efficient possible while providing sanitation to the most people. The implementation of simplified sewerage and its ramifications was expected to have more difficulties on this city than other smaller and less developed cities.

The case of Salvador de Bahia is probably the largest single application of the condominial model. It is also noteworthy for the enormous engineering challenges posed by the city and the extraordinary ingenuity with which they were overcome. More details on the process can be found on the report written by Melo (2005).

A key problem in implementation has been the difficulty in convincing local-residents to assume maintenance responsibilities associated with the networks. The main difficulty observed has been the low rate of connection to the sewerage network, undermining the original rationale for the program. However, this problem is not specifically related to the use of condominial technology, but rather is indicative of a wider challenge affecting the expansion of sewerage networks.

Parauapebas, Pará – Brazil:

Located in the State of Pará, 650 kilometers from the state capital, Belém. Established less than 20 years ago, the city owes its history and rapid growth to the development of rich iron ore deposits by the Companhia do Vale do Rio Doce (CVRD). Since its formal incorporation as a municipality in 1988, the population has grown from 20,000 to current levels approaching 100,000.

When CVRD was established in Parauapebas in the late 1970s, the company invested in a water and sewerage system for residents. There was no charge for the systems. Nevertheless, the rapid growth of the city quickly rendered these initial systems obsolete. Thus, by the early 1990s, they were deteriorating and served only a small area of the city. The sewerage network served only 6,400 residents in the downtown area, whose untreated effluents were returned directly to the river system. Elsewhere, residents developed their own on-site sanitation solutions, ranging from insanitary pits to septic tanks, which often overflowed onto streets and public areas of the city. The precarious conditions of both services contributed to the proliferation of diseases, particularly gastrointestinal infections, despite major efforts in public health education.

In 1993, a collaboration agreement was signed between CVRD and the Municipal Prefecture of Parauapebas (PMP) with a view to securing a World Bank loan to finance the expansion of water and sewerage services in the city. A loan of US$7.8 million (later increased to US$14.5 million) was disbursed to enable CVRD to finance the necessary works.

The case of Parauapebas is particularly noteworthy, for it illustrates the challenges involved in constructing a city-wide system virtually from scratch and achieving a fourfold increase in coverage in the space of six years. It is one of the most ambitious examples of community mobilization with 100% of residents both financing the materials for the condominial branches and contributing the labor to excavate the trenches.

Petrolina, Pernambuco – Brazil:

Before the implementation of simplified sewers, this city was under very precarious conditions – lots of waste water could be seen running down the unpaved streets, and waterborne diseases were frequent.

Studies on this area show that users did not mention any negative impacts. The most common positive impacts they referred to, were a decrease in the number of mosquitoes, an increase in property values, the cessation of disputes with neighbors concerning waste water disposal and the decrease or elimination of visible waste water from the street environment.

Ramagundam – India:

Small town in Karimnagar district of Andhra Pradesh in the southern part of India. It is about 240 km north of the state capital of Hyderabad and is located on right bank of Godavari River. It is the only town in India with long-term experience of using small-bore sewers. There is abundance of coal and water resources and which caused it to witness rapid industrialization during 1981-91 - mainly mining and thermal power plants.

An interesting off-shoot of the intervention towards improvement in off-site sanitation in the selected settlements is the changed behavior of the communities towards domestic solid waste management. It is noted that households in almost all settlements have now adopted the desirable practice of storing domestic waste at source and handing it out only to a separate private service provider who comes on daily collection rounds. It is understood that anchoring and evolution of this system has been facilitated by the ULB in consultation with the local communities.

Because of this practice the settlements are found to be very clean and free of any litter or trash. An added advantage of this is the reduced risk of blockage of the small-bore sewer lines and thereby improved operations (PSEAU-Ramagundam, 2017).

Louisiana Wastewater Systems Background

A significant part of Louisiana's current wastewater conveyance and treatment system is outdated and inefficient (Irion, 2016). When the Federal Water Pollution Act (later Clean Water Act of 1977) was enacted on October 18, 1972, a grant program to finance the creation of sanitary wastewater treatment plants throughout the United States was founded. This grant program was financed through 1990 and resulted in the construction of community wastewater treatment plants (WWTPs) throughout the country with corresponding substantial reduction of fecal loading to the waters of United States. Louisiana was one of the states that let the program lapse as federal founding diminished (Irion, 2016).

Up until the 1990s Louisiana had been mainly a rural state, but rapid suburban growth occurred along the newly constructed interstate corridors. This explosion of subdivision growth along the interstate corridors, where the marshy clay soil is unsuitable for sceptic systems, led to the requirement for individual **Aerated Package Treatment plants (ATUs)** for homeowners (Irion, 2016).

The practice of allowing ATU installations for treatment of household sewerage was intended to apply to isolated homes in rural areas of widely-separated houses, where a community sewerage system is economically or physically untenable. However, in Louisiana the onsite systems rule, which dates back to the early 1990s, contained a clause that exempted a development from providing community sewerage. This clause permitted the use of ATUs in subdivisions comprised of less than 125 lots.

The clause was appealing to many politicians and developers, eager for growth and development of their areas. Growth escalated in the 1990s, and now more than 322,000 systems have been installed in Louisiana to date. This means that nearly 19% of the households are currently served by individual sewerage treatment systems (Irion, The US Census Bureau, 2016).

Many parishes and incorporated communities provide community drinking water but have little or no infrastructure for provision of community sewerage.

On top of the inadequate maintenance of ATUs, that was often only charged a small fine, studies have indicated that even ATUs with good maintenance history exceeded the Total Suspended Solids (TSS) and Biochemical Oxygen Demand at day five (BOD5) limits (Pipeline, 2005).

The inefficiency of the current systems calls for changes. Simplified Sewerage systems are a great option and can be designed to be more sustainable than conventional systems.

Application to Rural Areas in Louisiana

From the studies on this topic it is visible that most if not all locations where the simplified systems were implemented have had success. On that note, it is possible to predict cost savings in comparison to the conventional systems, while providing basic sewerage to small rural communities.

For the purposes of this study, **St. Tammany** parish was chosen as an example for the application of Simplified Sewerage due to the parish's fitting profile as a location that has an increasing population on rural areas, residences that are far-spaced apart, and a high percentage (estimated to exceed 80%) of unsewered subdivisions.

Figure 5: Louisiana Parishes' Limits Map: Saint Tammany Parish highlighted. (Any Place America)

One area of St. Tammany Parish contained unsewered subdivisions of more than 3,500 homes. On top of the unsewered subdivisions, there are subdivisions which fail to comply with the regulations and chlorination requirements. The non-compliance with the standard requirements is typical, reaching more than 50% of homes in some areas. Multiply that number by the number of fecal colonies that are discharged at a rate of 20,000 fecal coliforms per 100 milliliters of discharge to waterways, streams, and lakes of that region. The contamination projection can reach a point to be irreversible.

St. Tammany Parish, Louisiana covers an area of approximately 1,120 square miles with a geographic center of **30.38773340°(N), -89.99761327°(W)**.

These are the far north, south, east, and west coordinates of St. Tammany Parish, Louisiana, comprising a rectangle that encapsulates it. Please note that not all coordinates inside this rectangle will fall inside of St. Tammany Parish, Louisiana, as it (most likely) doesn't fill up the entire rectangle.

North: **30.71240234°(N)**	East: **-89.52432251°(W)**
South: **30.14905167°(N)**	West: **-90.25668335°(W)**

According to data gathered by the Shuttle Radar Topography Mission. The lowest elevation in St. Tammany Parish is **-37 meters** (-121 feet). This ranks St. Tammany Parish **13th** in terms of lowest elevations when compared to a total of 64 counties in Louisiana.

St. Tammany Parish's highest elevation is **90 meters** (295 feet) which ranks it **27th** in terms of highest elevations when compared to a total of 64 counties in Louisiana (AnyPlaceAmerica, Access: 2018).

On a general perspective, the existing topography is appropriate to generate a sewer system going from the high point of the analyzed neighborhood to the lowest point and further on to the treatment plant.

Small Changes, Big Improvements

From experience, ATUs have proven to be ineffective and problematic to the general health of a community. Their use started as a rapid and affordable solution to the fast development of rural regions, for which the price of this technique is being paid for in the present, in the long-run.

Simplified Sewerage Systems come in handy as a financially viable solution for changing the direction for which the existing (current) sewerage systems, mainly constituted of ATU units, have taken.

The main goal of a sewerage system is to provide the most affordable, that requires the least amount of maintenance, and designed to have a life-cycle that makes it a sustainable, sewerage for a community. Simplified Sewerage is a perfect alternative.

Example of Design Project

Below there is a sample comparison of conventional sewers and simplified sewerage, designed for a fictitious community which is to be created. To facilitate the design and from lack of resources, the topography map used was from a location near, but not on St. Tammany parish. The community designed is expected to have rapid development and population fill. Some of the characteristics of the chosen sample community are the following:

Demographics:

For the calculations of sewer flow on the pipelines necessary either of sewer models to be compared, the number of households is neglected. The sewer flow is closely estimated by distributing the population to the planned areas.

Based on the average household size in Louisiana of 2.6 people per household, the assumed population for this region is of 3000 people per square kilometer.

The community is expected to have a future growth in population of one-hundred percent. For the calculation of sewer flow of the future population, 6000 people per square kilometer is assumed.

Average wastewater production for suburban residential areas in the united states is estimated to be 66gal/capita/day (sixty-six gallons per person per day) (Davis 2010).

$$66\frac{gal}{capita.day}x\ 2.6people = 172\frac{gal}{day} = 0.0075\frac{L}{s}$$

Topography:

Not many steep elevation changes, typical of Louisiana;

Change in elevation within the community: Highest point: 222ft

Lowest point: 180ft (Runoff stream)

The total area to be developed is approximately 1.556 km^2.

Due to the topography, no pumping (lifting) stations are going to be designed for the purpose of this study. The design will use of existing slopes to facilitate wastewater runoff, this is an important factor that generates huge cost savings.

The town used for this example is within the following limits:

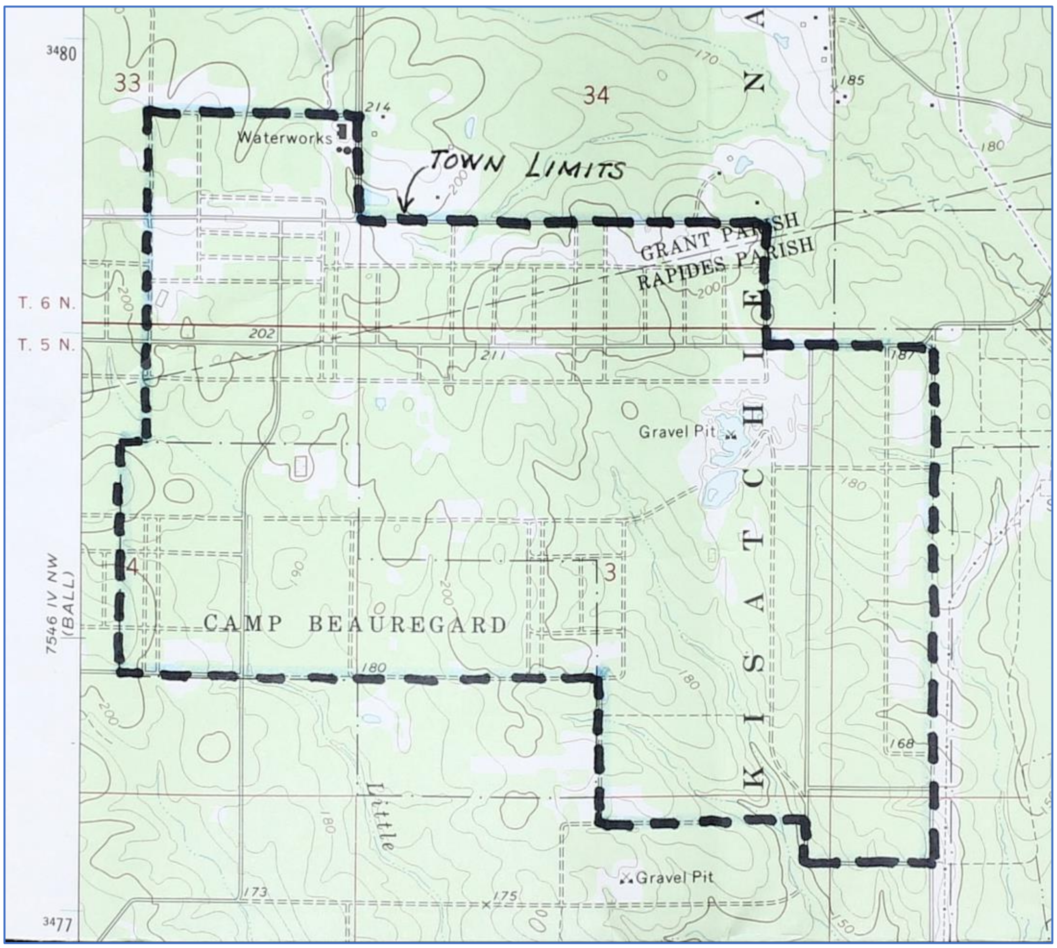

Figure 6: Camp Beauregard Limits to be Used as an Example of Community Soil Elevations.

The Camp Beauregard town's area is large. To facilitate the design, only the left (West) portion of it is going to be used. The remaining area is labeled as "Future Development Area" (Figure 12).

For a large area such as the one of this sample community, the sewer that flows from each of the four areas connects to the main at different points. At each one of these connections, the sewerage flow will be connected, and thus add to the main trunk line. The total sewer flow is the sum of the four area flows. The resulting sewer flow continues downstream to the treatment plant where the treatment process will begin. The designed points for this example are shown below:

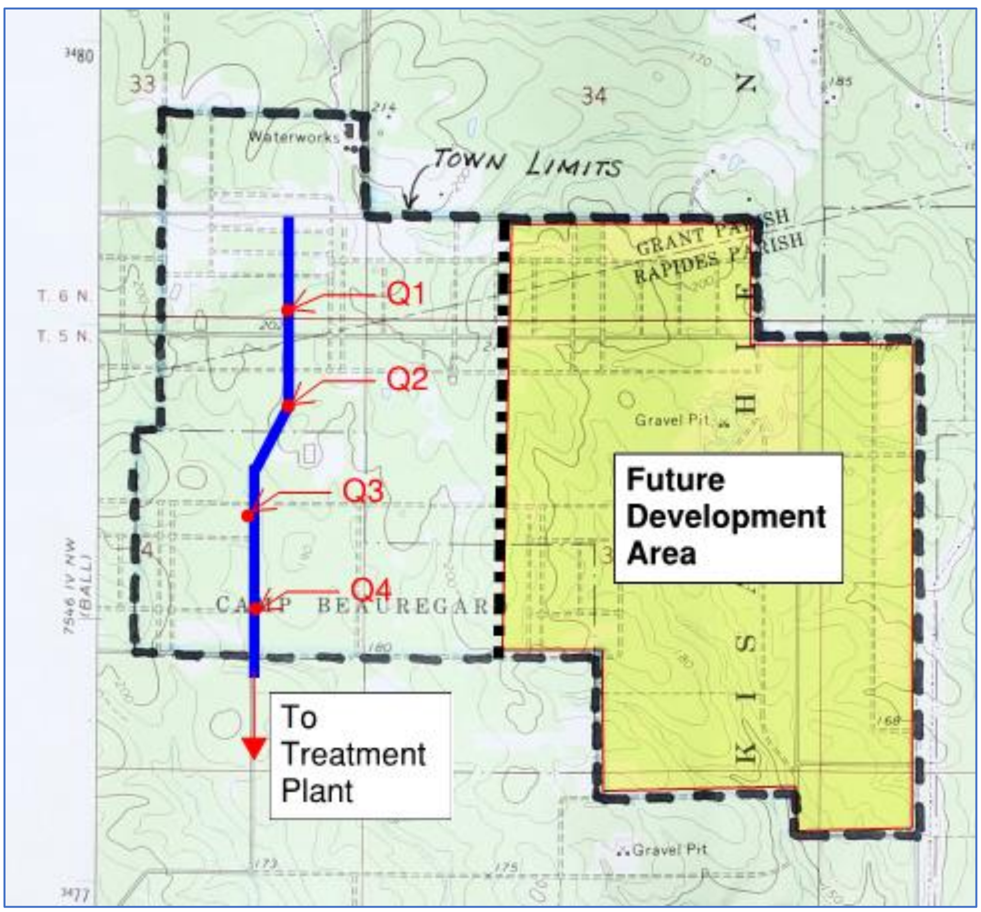

Figure 7: Overall Town View with Points of Connection to the Main.

The west side of the camp has all its sewage, infiltration, and inflow directed to the main trunk sewer line. Furthermore, the west side of the proposed subdivision was divided into four smaller areas which will have individual connections to the main. The main sewer line will direct the sewer downstream to a local treatment plant. The following design and calculations will show a more detailed explanation of Area-1 and will proceed to add the sewer flow resulting from both Area-1 and the other three areas to the main trunk sewer.

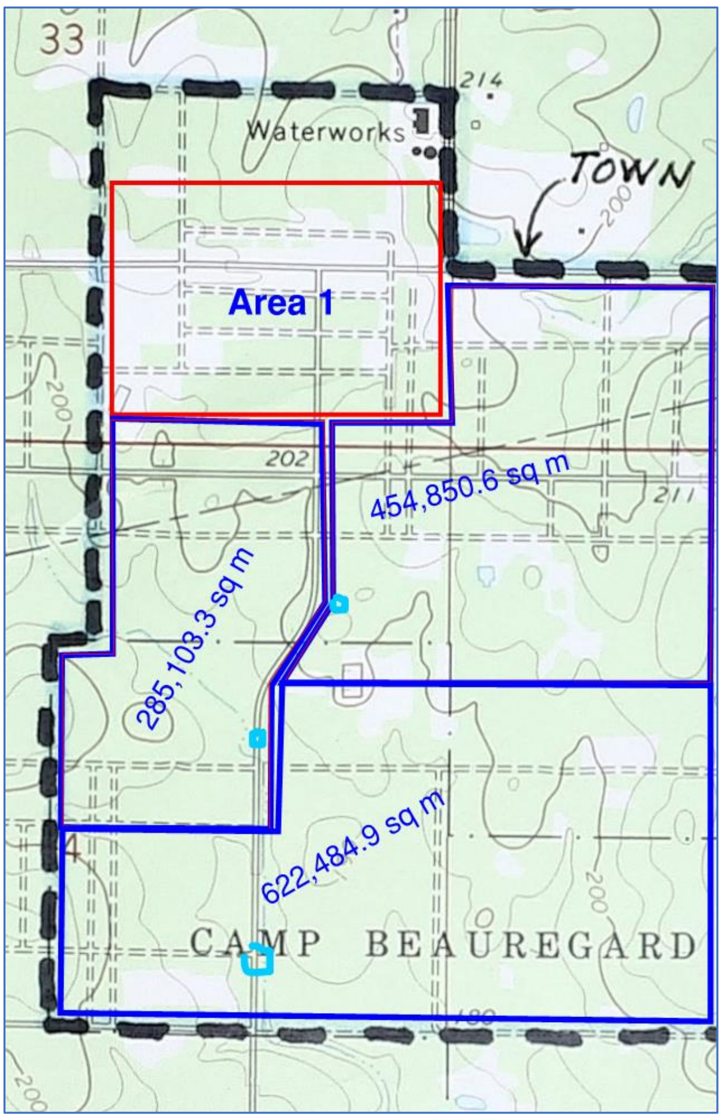

Figure 8: Left Side of Camp and Subdivided Areas of Community to be Created

Closing-up on Area-1: From the contour lines it is possible to determine the soil elevation at the position of each of the required manholes on the main avenues. Note that the elevations in feet are later converted to meters for the calculations:

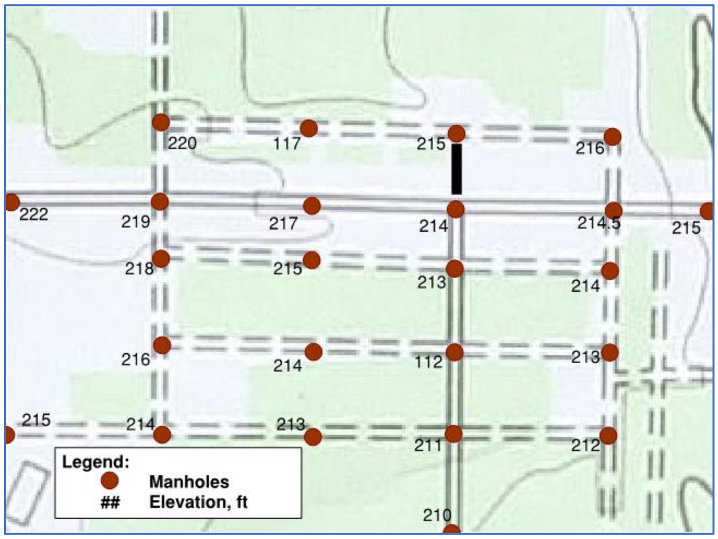

Figure 9: Manhole Elevations in feet on Area-1

The sewer that will go to each pipe is divided by sub-areas of collection from where it is produced and linked to a Main sewer pipe:

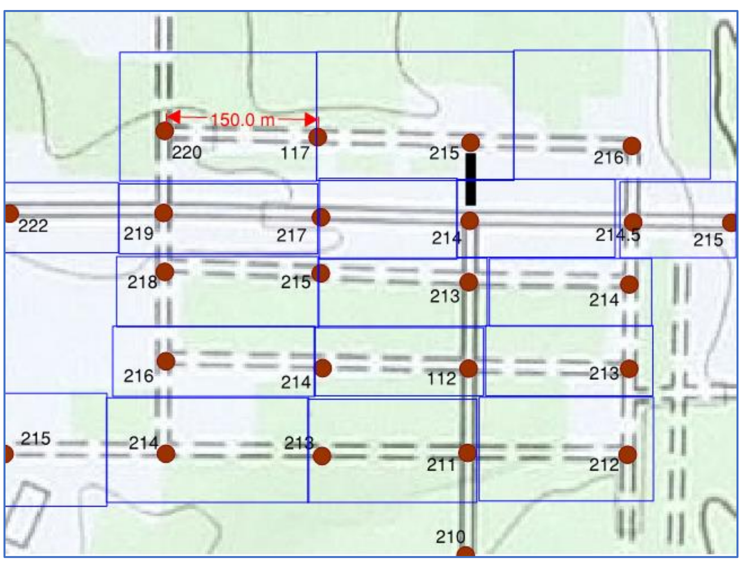

Figure 10: Sub-areas within "Area-1"

Pipe connections:

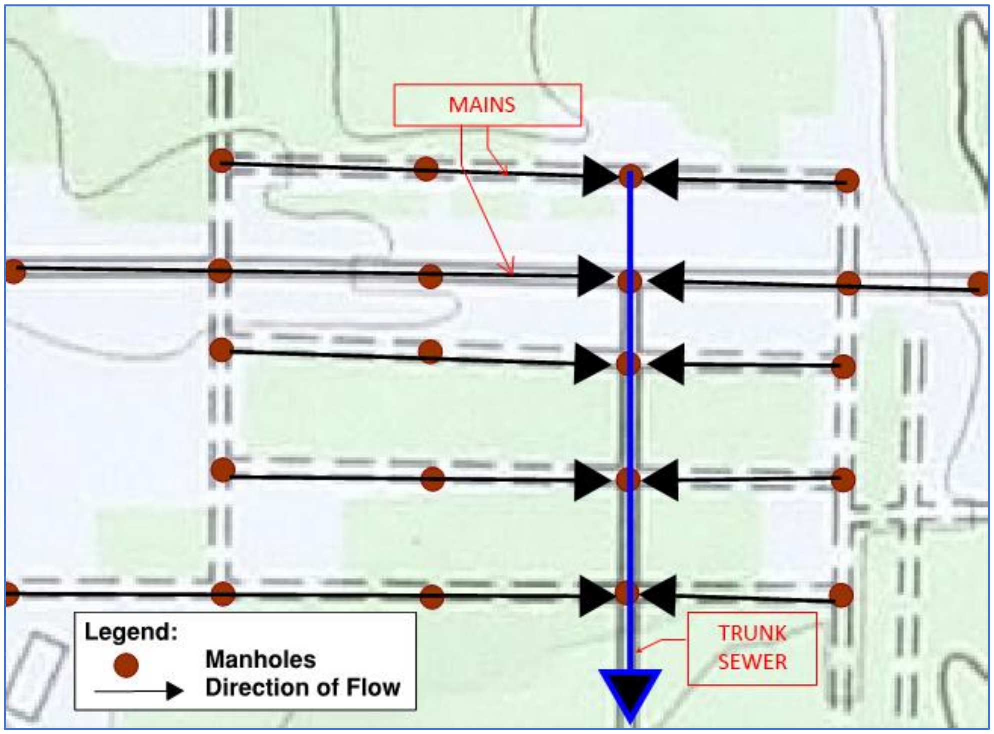

Figure 11: Required Sewer Pipe connections in "Area-1"

The following table shows the main's and trunk sewer's flow rates for the identified population using the sample pipe connections illustrated previously on Figure 11:

Table 1: Computation of Sewage Quantity on the Principal Mains and Trunk Sewer

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
		Manhole				**increment of population**		**Total population**		**Approx. total Infiltration (L/s)**	**Average Wastewater flow (L/s)**		**Surface elevation (m)**		
no.	**Location**	**from**	**to**	**Length of line (m)**	**area drained (km2)**	**present'**	**ultimate''**	**present**	**ultimate**		**present'''**	**ultimate**	**upper manhole**	**lower manhole**	**Street slope**
1	Avenue A	1	2	150	0.018	54	108	54	108	0.154	0.310	0.466	66.0	65.1	0.006
2	Avenue A	2	3	150	0.018	54	108	108	216	0.309	0.620	0.932	65.1	64.5	0.004
3	Avenue A	4	3	120	0.0144	43	86	43	86	0.123	0.248	0.373	64.8	64.5	0.002
4	Street 3	3	8	60				205	410		0.593	1.185	64.5	64.2	0.005
5	Avenue B	5	6	150	0.009	27	54	27	54	0.077	0.155	0.233	66.6	65.7	0.006
6	Avenue B	6	7	150	0.009	27	54	54	108	0.154	0.310	0.466	65.7	65.1	0.004
7	Avenue B	7	8	150	0.009	27	54	81	162	0.231	0.465	0.699	65.1	64.2	0.006
8	Avenue B	10	9	90	0.0054	16	32	16	32	0.046	0.093	0.140	64.5	64.4	0.002
9	Avenue B	9	8	120	0.0072	22	43	38	76	0.108	0.217	0.326	64.4	64.2	0.001
10	Street 3	8	13	60				421	842		1.216	2.432	64.2	63.9	0.005
11	Avenue C	11	12	150	0.00975	29	59	29	59	0.084	0.168	0.252	65.4	64.5	0.006
12	Avenue C	12	13	150	0.00975	29	59	59	117	0.167	0.336	0.505	64.5	63.9	0.004
13	Avenue C	14	13	120	0.0078	23	47	23	47	0.067	0.134	0.202	64.2	63.9	0.003
14	Street 3	13	17	75				532	1065		1.537	3.074	63.9	63.6	0.004
15	Avenue D	15	16	150	0.01125	34	68	34	68	0.096	0.194	0.291	64.8	64.2	0.004
16	Avenue D	16	17	150	0.01125	34	68	68	135	0.193	0.388	0.583	64.2	63.6	0.004
17	Avenue D	18	17	120	0.009	27	54	27	54	0.077	0.155	0.233	63.9	63.6	0.003
18	Street 3	17	22	75				661	1321		1.907	3.815	63.6	63.3	0.004
19	Avenue E	19	20	150	0.012	36	72	36	72	0.103	0.207	0.311	64.5	64.2	0.002
20	Avenue E	20	21	150	0.012	36	72	72	144	0.206	0.414	0.622	64.2	63.9	0.002
21	Avenue E	21	22	150	0.012	36	72	108	216	0.309	0.620	0.932	63.9	63.3	0.004
22	Avenue E	23	22	120	0.0096	29	58	29	58	0.082	0.165	0.249	63.6	63.3	0.002
23	Street 3	22	Q1	30				905	1811		2.614	5.229	63.3	63.0	0.010
24	Street 3	A2	Q2		0.455	3640	7280	4545	7271	12.987	26.112	33.981	**63**		
25	Street 3	A3	Q3		0.285	2280	4560	6825	10691	19.501	39.209	50.371			
26	Street 3	A4	Q4		0.622	4976	9952	11801	18155	33.718	67.795	86.140		**54**	0.008
				Total Area=	1.556	km2									

*' [Area (km2) (Col. 6) x 3000 people/km2]

*'' [Area (km2) (Col. 6) x 6000 people/km2]

*''' [Area (km2) (Col. 9) x rate (66gal/person.day) [1m3/1000L][1day/24.60.60][3.78L/gal]

For model comparison purposes, the north portion of Area-1 is to be used for the following calculations.

The households used for comparison are illustrated as green rectangles, and the added black lines are where the streets adjacent to the houses will be located:

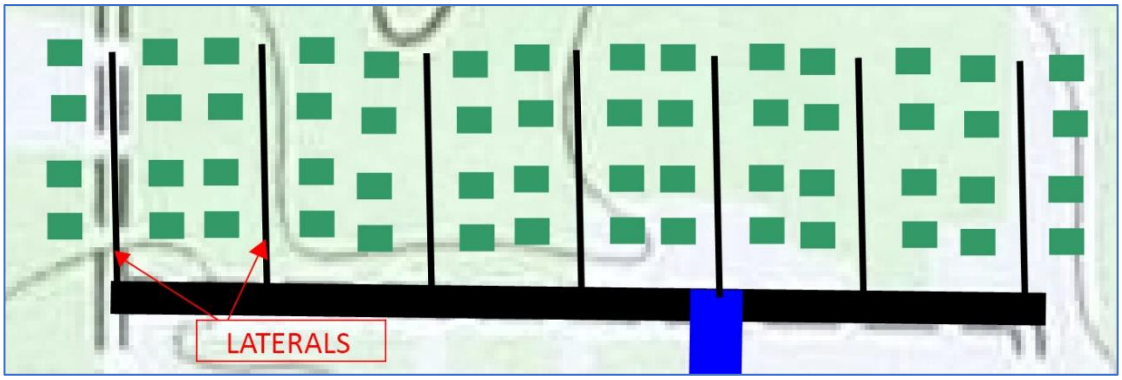

Figure 12: Proposed Subdivision Streets and Houses on the North Portion of Area-1.

Following is the comparison of the conventional sewer system with the simplified sewer system, which has the objective to demonstrate the possible cost savings of using one system over the other:

Design 1 – Conventional Gravity Sewer System

The design of a sewer system generally includes the following steps: preliminary investigations, a detailed survey, the actual design, and preparation of final drawings.

There are two main appurtenances on the conventional gravity sewer: Pumping/lifting stations and manholes. For the local characteristics of this sample, no pumping stations are being designed since the wastewater will mostly flow by gravity downstream. On the other hand, manholes are necessary at every intersection of a neighborhood when using the conventional system. Manholes have some very particular standards for construction and account for a significant part of the system's cost.

Typical manhole spacing for straight runs, some more general design criteria and recommendations for gravity flow sewer systems are given in Appendix B.

As previously mentioned, in Louisiana, the design criteria recommendation for sewers is to maintain a minimum velocity of 0.6m/s when the pipes are running full. Which can be complicated to achieve sometimes, since pipes used on a conventional system are also required to be at least 8-in in nominal diameter. There are two main equations used to the design of sewers: the Hazen-Williams equation (Equation 2.1) and Manning's equation (Equation 2.2).

Jefferson Parish uses the following criteria:

“Sewer mains shall be designed and constructed to provide mean velocities, when flowing full, of not less than 2.0 feet per second, based on Manning’s formula using an “n” value of 0.011. Minimum desired slope for an 8- inch main is 0.40%. slopes slightly less than the desired slope of 0.40% (0.40% to 0.30%) will be permitted to avoid exceeding maximum depth requirement for sewer gravity lines of 15 feet, to enable tying to an existing gravity system, and minimizing the number of lift stations.” (Source: Appendix “A,” Jefferson Parish Department of Engineering, Gravity Sanitary System General Standard Notes, revised 11/18/2014.)

Hazen and Williams formula:

$$Q = 0.2785\, C\, D^{2.63}\, S^{0.54} \text{ (Equation 2.1)}$$

Manning’s formula:

$$Q = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} A \text{ (Equation 2.2)}$$

Where Q = flow rate, m^3/s

C = Hazen-Williams coefficient of roughness

D = diameter of pipe, m

n = coefficient of roughness, unitless

R = hydraulic radius, m

S = slope of energy grade line, m/m

A = cross-sectional area of flow, m^2

The hydraulic radius “R” is the result of the division of the cross-sectional area “A” by the wetted perimeter of the channel (Equation 2.3).

$$R = \frac{Cross - sectional\ area, m^2}{Wetted\ perimeter, m} \text{ (Equation 2.3A)}$$

Thus, when the pipe is running full, the hydraulic radius is defined as:

$$R = \frac{(\pi/4)\ (D)^2}{\pi D} = \frac{D}{4}\ (Equation\ 2.3B)$$

Williams and Hazem (1905) investigated several different types of pipe to determine the now called Hazen-Williams coefficients of roughness, denoted by the letter C. A table of updated "C" values is given in Appendix C.

Typical values of coefficient of roughness, *n*, that are used with the Manning equation are shown on Appendix B.

Equation 2.4 bellow is used to find the central angle of flow inside the pipes. Once the central angle is known, it is possible to determine the hydraulic radius, R, and find if the existing surface slope is steep enough to be used for the sewer. The required slope is the one which will avoid solids settling and assure self-cleansing in the pipe.

$$q = \left[\frac{D^2}{8}(\theta - \sin\theta)\right]\left\{\frac{1}{n}\left[\frac{D}{4}\left(1 - \frac{\sin\theta}{\theta}\right)\right]^{\frac{2}{3}} S^{\frac{1}{2}}\right\} \quad (Equation\ 2.4)$$

Where q = minimum design flow rate, m^3/s

D = pipe internal diameter, m

θ = central angle of flow, radians

S = Existing slope, usually the street slope, unitless

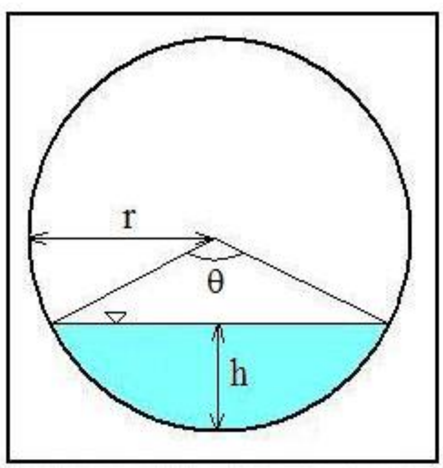

Figure 13: Partially Full Pipe Flow Parameters (Less Than Half Full) (Source: Engineering Excel Spreadsheets, 2011)

If the central angle θ is known, the hydraulic radius R can be calculated by:

$$R = \frac{D}{4}\left(1 - \frac{\sin\theta}{\theta}\right)(Equation\ 2.3C)$$

Then to check if the surface slope will assure self-cleansing, the tractive force developed in the sewer τ_o must be greater than the critical tractive force τ_c, which is usually considered to be 1 Pa (Davis, 2010).

$$\tau_0 = \gamma\ R\ S(Equation\ 2.5)$$

Where γ = Unit weight of water, 9.81kN/m^3

S = surface slope, unitless

Design 1 – Conventional House to Streets Connections:

The conventional system requires a manhole at every intersection of streets and individual house connections to the sewer line under the streets. The following figure shows the manholes, indicated by the red arrows, required to be added to "Avenue A" in case a conventional sewer system is to be used:

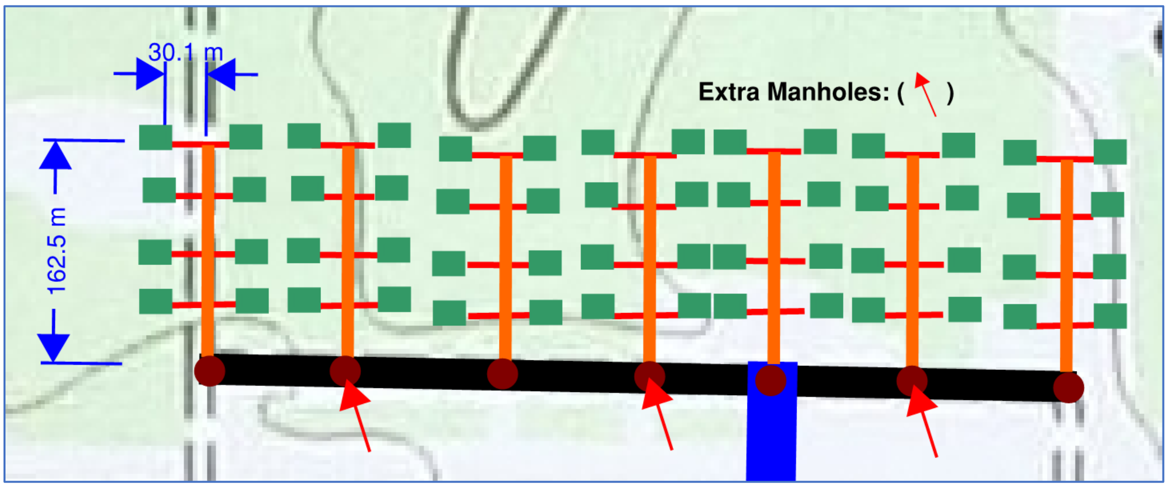

Figure 14: Conventional sewer connections between houses and streets and additional manholes required.

On the figure above, the house to street connections are approximate to be 30 meters long and the sewer line under each of the seven neighborhood streets is approximate to be 162.5 meters long.

A simple addition of length of orange pipes in the illustration shows that for those houses a total of approximately 2,817.5 meters of pipe are required.

Values are expected to have an average of 172 gal/day = 0.0075 L/s on the pipe connections between the house and the streets at the beginning of design period. The beginning of the design period values are the "present" values on the tables that are later introduced at this section.

The recommended Manning’s n used is = 0.013 for pipes running full, the recommendation is available on Table B-4 on Appendix B.

The flow maximum velocity is recommended to be between 2.5 m/s and 3.0 m/s. Thus, for the purposed of this comparison we will use 2.5 m/s.

Using Manning’s Equation (Equation 2.2), it is possible to come up with the best market available pipe diameters for each section of the sewer line.

Infiltration/inflow values are estimated to be 30 L/(d. mm. km.), which is in accordance with ASCE code listed on Appendix-D.

Table 2: Computation of Sewge Quantity and Required Drop on Conventional Sewer Connection

House		Length of line (m)	increment of population		Total population		Wastewater flow (L/s)		Infiltration (L/s)	Total Flow (L/s)		Peak Ultimate Flow (L/s)	Minimum diameter (mm)	Commercial Diameter (mm)	Minimum Slope "I"	Total drop in sewer (m)
from	to		Present	Future	Present	Future	Present	Future		Present	Future					
A1	S1	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
A2	S1	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
S1	S2	40			5.2	10.4	0.015	0.030	0.003	0.018	0.033	1.203	24.75	200	0.027	2.31
B1	S2	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
B2	S2	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
S2	S3	40			10.4	20.8	0.030	0.060	0.003	0.033	0.063	2.406	35.00	200	0.018	3.03
C1	S3	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
C2	S3	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
S3	S4	40			15.6	31.2	0.045	0.090	0.003	0.048	0.093	3.608	42.87	200	0.014	3.59
D1	S4	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
D2	S4	30	2.6	5.2	2.6	5.2	0.008	0.015	0.002	0.010	0.017	0.601	17.50	200	0.041	1.23
S4	S5	42.5			20.8	41.6	0.060	0.120	0.003	0.063	0.123	4.811	49.50	200	0.012	4.10

Although a smaller sewer pipe would be able to carry the present and future design flow requirement for the sewer, the minimum internal diameter requirement dictates that 8-in (200 mm) pipes are to be used. On this sample neighborhood the houses are far-spaced, the design population is relatively small for the area, and the design flow comes out to be very small. All the house connections north and on Avenue A main require pipes no bigger than 200 mm. Thus, the net amount of materials required comes out to be:

The total length of 200 mm pipes = (2817.5 + 450) = 3267.5 meters

The cost of this system can be broken down as follows:

Cost of pipes:	3267.5 meters (U$56/meter)	=	U$182,980
Cost of manholes:	7 manholes (U$4000/manhole)	=	U$28,000
Total Costs (includes additional 25% for labor cost)		=	U$268,725

Design 2 – Simplified Sewerage System (Backyard and Front yard):

Model designed utilizing backyard and front yard types of simplified sewerage systems.

Following are figures of the north portion of Area-1 on the proposed site:

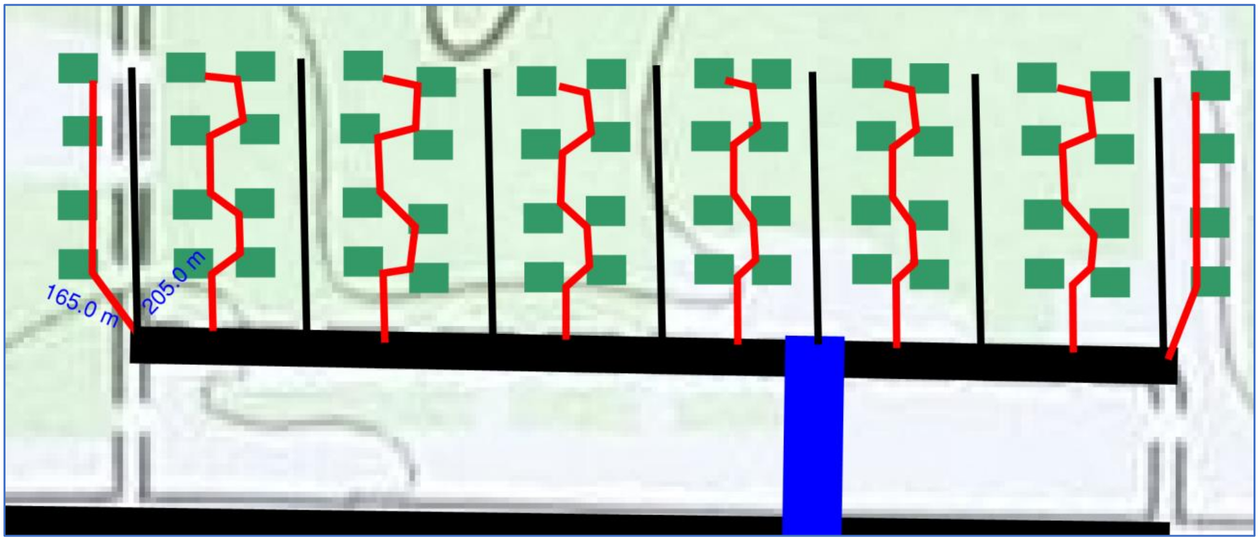

Figure 15: Simplified Sewer Connection Between Houses and Streets. (Backyard and Front Yard Models)

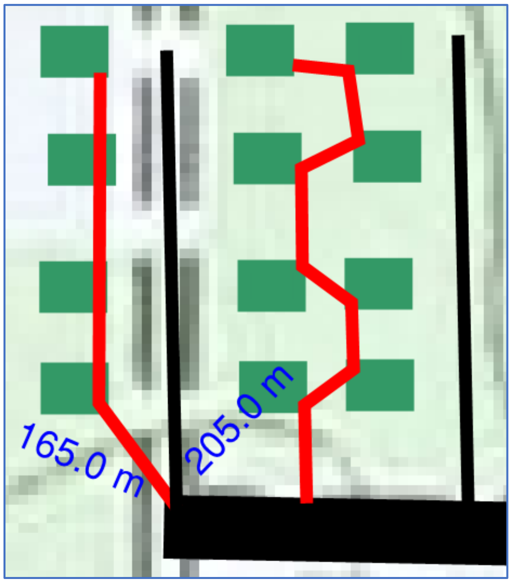

Figure 16: Closeup of Aerial View of the Proposed Simplified Connections.

On the figures above, the house to street connections use of two models: The outside ones use of the front-yard connection type and are approximately 165 meters long. The remaining six inside connections use of the backyard connections and are approximately 205 meters long.

A simple addition of length of pipes shows that for those houses a total of approximately 1,560 meters of pipe marked in orange are required. A total of forty-five percent less pipes than the conventional system.

Table 3: Computation of Sewage Quantity and Required Drop on Simplified Sewer Connection

House		Length of line (m)	increment of population		Total population		Wastewater flow (L/s)		Infiltration (L/s)	Total Flow (L/s)		Peak Ultimate Flow (L/s)	Minimum diameter (mm)	Commercial Diameter (mm)	Minimum Slope "I"	Total drop in sewer (m)
from	to		Present	Future	Present	Future	Present	Future		Present	Future					
a	b	40	2.6	5.2	2.6	5.2	0.008	0.015	0.001	0.009	0.016	0.588	17.30	100	0.051	2.02
b	c	40	2.6	5.2	5.2	10.4	0.015	0.030	0.001	0.016	0.031	0.846	20.75	100	0.038	3.54
c	d	40	2.6	5.2	7.8	15.6	0.023	0.045	0.001	0.024	0.046	1.433	27.02	100	0.032	4.81
d	S5	45	2.6	5.2	10.4	20.8	0.030	0.060	0.002	0.032	0.062	1.233	25.06	100	0.028	6.07
e	f	15	2.6	5.2	2.6	5.2	0.008	0.015	0.001	0.008	0.016	0.570	17.04	100	0.053	0.80
f	g	35	2.6	5.2	5.2	10.4	0.015	0.030	0.001	0.016	0.031	0.843	20.72	100	0.038	2.13
g	h	15	2.6	5.2	7.8	15.6	0.023	0.034	0.001	0.023	0.034	0.888	21.27	100	0.032	2.62
h	i	35	2.6	5.2	10.4	20.8	0.030	0.045	0.001	0.031	0.046	1.050	23.13	100	0.028	3.60
i	j	15	2.6	5.2	13	26	0.038	0.056	0.001	0.038	0.057	1.178	24.50	100	0.026	3.98
j	k	35	2.6	5.2	15.6	31.2	0.045	0.068	0.001	0.046	0.069	1.311	25.84	100	0.023	4.80
k	l	15	2.6	5.2	18.2	36.4	0.053	0.079	0.001	0.053	0.079	1.421	26.90	100	0.022	5.13
l	S5	40	2.6	5.2	20.8	41.6	0.060	0.120	0.001	0.061	0.122	1.803	30.31	100	0.020	5.94

The table above calculates the minimum slope required on each of the pipes using the previously introduced Formula-1, $I = 0.0055\ Q^{-0.47}$, used in Brazil (Brazilian Standards NBR 9469/86 – NBR 9649 Projeto de redes coletoras de esgoto sanitario). The table shows that the pipelines connecting these houses to a sewer main under the Avenue-A need a significantly less drop from the first house on the layout to the point of connection to the pipelines under the main street (Avenue A in this case). The downside is, that the two drops of 6.07m and 5.94m are significantly more than the 4.10m drop required for the conventional system. This is a downside of using Simplified Sewerage on low density areas.

From the sample design, it is possible to determine that the best market available pipe diameter for all the sewer connections using the simplified model are 100 mm in nominal diameter.

The total length of 100 mm pipes = (1560 + 450) = 2010 meters

As mentioned previously, the proposed simplified sewerage design can avoid the use of any manholes and replace them with cleanout boxes. These boxes are much cheaper than manholes and easier to perform maintenance on.

The cost of this system for the specified area can be broken down as follows:

Cost of pipes: 2010 meters (U$24/meter)	=	U$48,240
Cost of cleanout boxes: 7 boxes (U$300 /box)	=	U$2,100
Total Costs (includes a 25% for labor costs)	=	U$62,925

Overall Design and Cost Comparison for the Entire Site:

For the west portion of Camp Beauregard, which is to be developed, it is possible to estimate the cost for the sewer system necessary by assuming pipelines and other appurtenances required per area of neighborhood.

With exception of the trunk sewer which goes from north to south across the entire development area and is designed to convey all the sewage, all the remaining pipes are to be designed to convey a flow rate small enough so that the design diameter does not need to be greater than 100 mm. The following table shows that for "Area-1" the design diameter of the mains does not need to be bigger than the 100 mm allowed for the simplified systems:

Table 4: Computation of Design Diameter for Area 1 and Trunk Sewer							
			Location		Ultimate	Peak flow	Min d
no.	Location	Description	from	to	flow (L/s)	(L/s)	(mm)
1	Avenue A	Main	1	2	0.466	3.83	**44.16**
2	Avenue A	Main	2	3	0.932	5.64	**53.62**
3	Avenue A	Main	4	3	0.373	3.38	**41.48**
4	Street 3	Trunk Sewer	3	8	1.185	6.46	**57.34**
5	Avenue B	Main	5	6	0.233	2.60	**36.37**
6	Avenue B	Main	6	7	0.466	3.83	**44.16**
7	Avenue B	Main	7	8	0.699	4.80	**49.47**
8	Avenue B	Main	10	9	0.140	1.95	**31.52**
9	Avenue B	Main	9	8	0.326	3.14	**39.96**
10	Street 3	Trunk Sewer	8	13	2.432	9.66	**70.13**
11	Avenue C	Main	11	12	0.252	2.72	**37.19**
12	Avenue C	Main	12	13	0.505	4.00	**45.16**
13	Avenue C	Main	14	13	0.202	2.40	**34.94**
14	Street 3	Trunk Sewer	13	17	3.074	11.01	**74.89**
15	Avenue D	Main	15	16	0.291	2.94	**38.71**
16	Avenue D	Main	16	17	0.583	4.34	**47.01**
17	Avenue D	Main	18	17	0.233	2.60	**36.37**
18	Street 3	Trunk Sewer	17	22	3.815	12.17	**78.74**
19	Avenue E	Main	19	20	0.311	3.05	**39.42**
20	Avenue E	Main	20	21	0.622	4.50	**47.86**
21	Avenue E	Main	21	22	0.932	5.64	**53.62**
22	Avenue E	Main	23	22	0.249	2.69	**37.03**
23	Street 3	Trunk Sewer	22	Q1	5.229	16.19	**90.81**
24	Street 3	Trunk Sewer	A2	Q2	33.981	88.09	**211.81**
25	Street 3	Trunk Sewer	A3	Q3	50.371	125.79	**253.10**
26	Street 3	Trunk Sewer	A4	Q4	86.140	204.42	**322.66**

Note that the trunk mains are not included on any of the cost calculations for either of the designs under comparison. Thus, the minimum diameter for all the pipes in analysis is considered to be the same for the mains and laterals.

From previous calculation on the northernmost street within area-1 (Avenue A), it was possible to get the following parameters:

> For an area of 0.0504 km^2, design 1 needed 3267.5 meters of 200 mm pipes and 7 manholes. The estimated cost using this design came out to be U$268,725.
>
> For this same area, using design 2, 2010 meters of 100 mm pipes were required and the estimated cost including 7 cleanout boxes came out to be U$62,925.

The area to be developed on the west portion of Camp Beauregard is 1.556 km^2. Thus, multiplying the cost per area analyzed (0.0504 km^2) with the total area, the costs came out to be the following:

$$Factor = \frac{1.556 km^2}{0.0504\ km^2} = 30.87$$

Design 1: 30.87 x U$268,725 = U$ 8.30 million

Design 2: 30.87 x U$62,925 = U$ 1.95 million

For the same area, population, and number of residences, design 1 comes out to be clearly more affordable for a couple of reasons. The pipes used are smaller in diameter and thus, cheaper. The total length of pipes is reduced on a simplified design, thus, uses less material. Not to forget that replacing expensive manholes with cleanout boxes is clearly another major cost saving benefit of simplified sewerage.

Conclusion

1. Understanding the Process:

This procedure is a community effort. During its years of putting the condominial system into practice on a grand scale, perhaps the most important lesson learned is that its success depends on involving all the areas of those responsible to the task in hand, thus getting a result that truly addresses the objectives of the system. This involvement demands complete understanding of the system by everybody in the project, so that each one has a precise idea of their importance in the system and of the contribution that their work will make to the process as a whole (Neder, 1998).

2. Benefits of Simplified Sewerage:

Among the benefits of simplified sewerage, it is common to find reduced construction cost, maintenance cost, and construction time. The collective process generates savings for the community and government.

Due to the reduced costs, it is also much easier to find financing for projects. Simplified Sewerage projects can be initiated and start to be used before its total completion, whereas the conventional systems cannot, making it more time efficient and relating to actual populational changes instead of only projections.

3. Louisiana Savings Projection:

From the sample design presented, it is possible to conclude that the conventional system is far more expensive at initial installation than the simplified model. The calculations presented show savings projections for multiple factors when using the simplified model.

In Louisiana, the surface soil is often too flat and requires a lot of pumping to convey sewage. With this simple technology in mind, parishes such as St. Tammany can generate huge savings and provide appropriate sanitation for its population quicker than using the conventional system.

The enormous cost of installation of the conventional model, as shown on the comparison case, is assigned to the costs of this unnecessary quantity of pipes, which are larger in diameter and thus, more expensive than the ones required for the simplified model.

The need for additional manholes is another major additional cost of conventional systems. The simplified system can replace the manholes for cleanout boxes that are easier to inspect and perform maintenance than conventional manholes.

4. **Possible Negative Aspects of Simplified Sewerage:**

Apart from the undoubtful benefit of using smaller pipe diameters, there were results that peered against simplified sewerage. The results show that for a neighborhood with a small population density, the required depth of sewer using a minimum slope for self-cleansing can be enormous since the design minimum flow rate is very low. As shown on the calculations, when utilizing the simplified model, it is possible that small density areas will require a steeper pipe slope to similarly assure self-cleansing. The cost savings of having a shallower installation and possibly avoid or at least reduce the need for pumping generates savings, thus, being a setback for Simplified Sewerage on low density areas.

Lastly, the implementation of simplified sewerage systems must face the barrier of having neighbors cooperate with each other. The simplified model can only achieve the same purpose as the conventional model if there are no additional complications to maintain the system working properly. The population using it needs to understand the importance of such system and allow regular maintenance within their property limits. These are some of the reasons why past implementation of the simplified model have mostly happened in high density areas, which required an immediate action on the existing, if any, sewerage system at the time.

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Appendices

APPENDIX A MEETINGS

Meetings were held with the following people:

Dr. La Motta, Engineer, University of New Orleans, New Orleans;

Dr. Skaggs, Water Quality Program Director, Lake Pontchartrain Basin Foundation, New Orleans;

Dr. Rincón, Professor, University of New Orleans, New Orleans;

Ms. Sutherland, Academic Counselor, University of New Orleans, New Orleans;

APPENDIX B GRAVITY SEWERS AND MANHOLE RECOMMENDATIONS

TABLE B-1

Recommended minimum slopes for gravity flow sewers[a]

Nominal diameter, mm	Minimum slope, m/m $n = 0.013$	Minimum slope, m/m $n = 0.010$	Capacity, flowing full, m^3/s
200	0.0033	0.0020	0.019
250	0.0025	0.0015	0.029
300	0.0019	0.0011	0.042
350	0.0016	0.0009	0.058
375	0.0014	0.0008[b]	0.066
400	0.0013	0.0008[b]	0.075
450	0.0011	0.0007[b]	0.095
500	0.0010	0.0006[b]	0.118
600	0.0008[b]	0.0005[b]	0.170

[a]Calculated using Manning's equation with nominal pipe diameter and $v = 0.6$ m/s.
[b]The minimum practicable slope for construction is about 0.0008 m/m.

Source: Davis (2010)

TABLE B-2

Typical manhole spacing for straight runs[a]

Pipe diameter	Spacing
375 mm or less	120 m or less
450 to 750 mm	150 m
or	
450 to 750 mm	180 m with adequate cleaning equipment
825 to 1,200 mm	180 m
1,200 mm or greater	460 m

[a] The actual spacing is highly dependent on local conditions and client preference.

Source: Davis (2010)

TABLE B-3

Typical design criteria for gravity sewers

Parameter	Recommendation	Comment
Pipe material	PVC	For main sewers
Pipe diameter	200 mm minimum	Nominal diameter
	Size to carry peak flow rate	See Table 19-2
Slope	To achieve 0.6 m/s	Flowing full, Manning's $n = 0.013$, see Table 19-2
Maximum velocity	2.5 to 3.0 m/s	
Depth to invert	3.0 to 3.5 m minimum	When basements are present or 0.6 m below the lowest basement
	1.8 to 2.4 m	When no basements are present
	8 to 9 m maximum	
Depth to crown	$H = D/6$ minimum	For dead load; granular soil
		For live load see Moser (2001)
Location with respect to water mains	3.0 m horizontally	Edge to edge
	0.45 m vertically	Outside to outside; water above sewer
	When crossing	Full length to joint for water main or encased sewer
Alignment	Straight between manholes	600 mm or less
Manholes	Place at:	
	junction of two sewers	
	change in vertical alignment	
	change in horizontal alignment	
	change in pipe size	
	at end of each line	
	Spacing for straight runs	See Table 19-3
	Effluent elev. = Influent elev. − 0.03 m	To account for entrance and exit losses
Drop manholes	When inflow and outflow inverts differ by more than 0.6 m	
Manhole diameter	1.2 m minimum	Access ≥ 0.6 m

Source: Davis (2010)

TABLE B-4

Typical values of n that are used with the Manning's equation.

	Condition		
Pipe material	Good	Fair	Deteriorated
DIP (lined)	0.011	0.013[a]	0.015
HDPE	0.010[a]	0.011	0.013
PVC	0.010[a]	0.011	0.013
RCP	0.013	0.015[a]	0.018
VCP	0.013[a]	0.015[a]	0.017

[a]Values commonly used in design.

Source: Davis (2010)

APPENDIX C PIPE AND FITTING

TABLE C-1
Hazen-Williams friction coefficients (C) for nonaggressive water [a]

Concrete pipe, centrifugally spun	
New	140
5 y old	130
10 y old	100
20 y old	80
Ductile iron pipe (DIP)	
Lined	
10–40 y old	125–140
Unlined	
5 y old, > 600 mm	130
5 y old, 75 to 600 mm	120
20 y old	100
30 y old	80
Severely tuberculated	40
FRP (higher C for larger diameter)	120–150
Plastic (PVC) (higher C for larger diameter)	120–150
Steel, new, unlined	140
Steel, 10 y old, lined	100

[a] Precipitates from softened water can lower C from 140 to 120 in as short a period as six months. Slime buildup over a 5 y period can lower C from 140 to 100.

Source: Davis (2010)

TABLE C-2
Pipe diameters for mortar-lined ductile iron pipe sizes in SI and U.S. customary units

SI, nominal mm	SI, ID mm	U.S. nominal inches	U.S. ID inches
75	82	3	3.22
100	102	4	4.04
150	155	6	6.10
200	208	8	8.21
250	259	10	10.2
300	312	12	12.3
350	363	14	14.3
400	415	16	16.3
450	468	18	18.4
500	521	20	20.5
600	627	24	24.7
750	781	30	30.7
900	937	36	36.9

TABLE C-3
Sewer pipe sizes in SI and U.S. customary units

SI, nominal mm	SI, ID mm	U.S. nominal inches
100	101.6	4
150	152.4	6
200	203.2	8
250	254.0	10
300	304.8	12
350	355.6	14
400	406.4	16
450	457.2	18
500	508.0	20
600	609.6	24
750	762.0	30
900	914.4	36

Source: Davis (2010)

TABLE C-4
SI-based velocity and friction headloss

Capacity		Pipe diameters in mm											
		50		75		100		125		150		200	
m^3/h	m^3/s	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f
1	0.0003	0.001	0.12										
2	0.0006	0.004	0.44	0.001	0.06								
3	0.0008	0.009	0.93	0.002	0.13								
4	0.0011	0.016	1.6	0.003	0.22	0.001	0.05						
5	0.0014	0.026	2.4	0.005	0.33	0.002	0.08						
6	0.0017	0.037	3.4	0.007	0.47	0.002	0.11						
7	0.0019	0.050	4.5	0.010	0.62	0.003	0.15						
8	0.0022	0.065	5.7	0.013	0.79	0.004	0.20	0.002	0.07				
9	0.0025	0.083	7.1	0.016	1.0	0.005	0.24	0.002	0.08				
10	0.0028	0.102	8.6	0.020	1.2	0.006	0.30	0.003	0.10	0.001	0.04		
15	0.0042			0.045	2.5	0.014	0.63	0.006	0.21	0.003	0.09		
20	0.0056			0.081	4.3	0.026	1.1	0.010	0.36	0.005	0.15		
25	0.0069			0.126	6.5	0.040	1.6	0.016	0.54	0.008	0.22	0.002	0.06
30	0.0083			0.181	9.1	0.057	2.3	0.024	0.76	0.011	0.31	0.004	0.08
40	0.0111					0.102	3.8	0.042	1.3	0.020	0.53	0.006	0.13
50	0.0139					0.159	5.8	0.065	2.0	0.031	0.80	0.010	0.20
60	0.0167					0.230	8.1	0.094	2.7	0.045	1.1	0.014	0.28
70	0.0194					0.313	10.8	0.128	3.6	0.062	1.5	0.020	0.37
80	0.0222							0.167	4.7	0.081	1.9	0.026	0.47
90	0.0250							0.212	5.8	0.102	2.4	0.032	0.59
100	0.0278							0.261	7.1	0.126	2.9	0.040	0.71
125	0.0347							0.408	10.7	0.197	4.4	0.062	1.1
150	0.0417									0.283	6.1	0.090	1.5
175	0.0486									0.386	8.2	0.122	2.0

Headloss (h_f) in m/100 m.
$C = 100$
Velocity headloss ($v^2/2g$) in m.

(*continued*)

Source: Davis (2010)

TABLE C-4 *(continued)*
SI-based velocity and friction headloss

Capacity		Pipe diameters in mm											
		200		250		300		350		375		400	
m^3/h	m^3/s	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f
30	0.008	0.004	0.08										
40	0.011	0.006	0.13										
50	0.014	0.010	0.20	0.004	0.07								
60	0.017	0.014	0.28	0.006	0.09								
70	0.019	0.020	0.37	0.008	0.12								
80	0.022	0.026	0.47	0.010	0.16	0.005	0.07						
90	0.025	0.032	0.59	0.013	0.20	0.006	0.08						
100	0.028	0.040	0.71	0.016	0.24	0.008	0.10						
125	0.035	0.062	1.1	0.026	0.36	0.012	0.15	0.007	0.07	0.005	0.05		
150	0.042	0.090	1.5	0.037	0.51	0.018	0.21	0.010	0.10	0.007	0.07		
175	0.049	0.122	2.0	0.050	0.68	0.024	0.28	0.013	0.13	0.010	0.09	0.008	0.07
200	0.056	0.159	2.6	0.065	0.87	0.031	0.36	0.017	0.17	0.013	0.12	0.010	0.09
250	0.069	0.249	3.9	0.102	1.3	0.049	0.54	0.027	0.26	0.020	0.18	0.016	0.13
300	0.083	0.359	5.5	0.147	1.8	0.071	0.76	0.038	0.36	0.029	0.26	0.022	0.19
350	0.097	0.488	7.3	0.200	2.4	0.096	1.0	0.052	0.48	0.040	0.34	0.031	0.25
400	0.111	0.638	9.3	0.261	3.1	0.126	1.3	0.068	0.61	0.052	0.44	0.040	0.32
500	0.139			0.408	4.7	0.197	1.9	0.106	0.92	0.081	0.66	0.062	0.48
600	0.167			0.588	6.6	0.283	2.7	0.153	1.3	0.116	0.92	0.090	0.67
700	0.194			0.800	8.8	0.386	3.6	0.208	1.7	0.158	1.2	0.122	0.89
800	0.222			1.045	11.3	0.504	4.6	0.272	2.2	0.206	1.6	0.159	1.1
900	0.250					0.638	5.8	0.344	2.7	0.261	2.0	0.202	1.4
1000	0.278					0.787	7.0	0.425	3.3	0.323	2.4	0.249	1.7
1500	0.417							0.956	7.0	0.726	5.0	0.561	3.7
2000	0.556									1.290	8.5	0.997	6.2

Headloss (h_f) in m/100 m.
C = 100
Velocity headloss ($v^2/2g$) in m.

Source: Davis (2010)

TABLE C-4 *(continued)*
SI-based velocity and friction headloss

Capacity		Pipe diameters in mm											
		450		500		525		600		675		750	
m^3/h	m^3/s	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f	$v^2/2g$	h_f
350	0.097	0.019	0.14	0.013	0.08	0.010	0.07						
400	0.111	0.025	0.18	0.016	0.11	0.013	0.08						
500	0.139	0.039	0.27	0.026	0.16	0.021	0.13	0.012	0.07				
600	0.167	0.056	0.38	0.037	0.23	0.030	0.18	0.018	0.09				
500	0.139	0.039	0.27	0.026	0.16	0.021	0.13	0.012	0.07				
700	0.194	0.076	0.50	0.050	0.30	0.041	0.24	0.024	0.12	0.015	0.07		
800	0.222	0.10	0.65	0.065	0.39	0.054	0.30	0.031	0.16	0.020	0.09		
900	0.250	0.13	0.80	0.083	0.48	0.068	0.38	0.040	0.20	0.025	0.11	0.016	0.07
1000	0.278	0.16	0.98	0.10	0.58	0.084	0.46	0.049	0.24	0.031	0.14	0.020	0.08
1500	0.417	0.35	2.1	0.23	1.2	0.19	0.97	0.11	0.51	0.069	0.29	0.045	0.17
2000	0.556	0.62	3.5	0.41	2.1	0.34	1.7	0.20	0.87	0.12	0.49	0.081	0.29
2500	0.694	0.97	5.3	0.64	3.2	0.52	2.5	0.31	1.3	0.19	0.74	0.13	0.44
3000	0.833	1.40	7.4	0.92	4.5	0.76	3.5	0.44	1.8	0.28	1.0	0.18	0.62
3500	0.972	1.91	9.9	1.25	5.9	1.03	4.7	0.60	2.4	0.38	1.4	0.25	0.82
4000	1.111			1.63	7.6	1.34	6.0	0.79	3.1	0.49	1.8	0.32	1.1
4500	1.250			2.07	9.4	1.70	7.4	1.00	3.9	0.62	2.2	0.41	1.3
5000	1.389					2.10	9.0	1.23	4.7	0.77	2.7	0.50	1.6
5500	1.528							1.49	5.6	0.93	3.2	0.61	1.9
6000	1.667							1.77	6.6	1.11	3.7	0.73	2.2
6500	1.806							2.08	7.7	1.30	4.3	0.85	2.6
7000	1.944							2.41	8.8	1.51	5.0	0.99	3.0
7500	2.083							2.77	10.0	1.73	5.6	1.13	3.4
8000	2.222									1.97	6.3	1.29	3.8
8500	2.361									2.22	7.1	1.46	4.2

Headloss (h_f) in m/100 m.
$C = 100$
Velocity headloss ($v^2/2g$) in m.

Source: Davis (2010)

TABLE C-5
Hydraulic headlosses for appurtenances

Appurtenance—alphabetically	Headloss as multiple of ($v^2/2g$)	Appurtenance—alphabetically	Headloss as multiple of ($v^2/2g$)
1. *Butterfly valves*		8. *Entrance losses*	
Fully open	0.3	Bell mouthed	0.04
Angle closed, θ = 10°	0.46	Pipe flush with tank	0.5
θ = 20°	1.38	Pipe projecting into tank (Borda entrance)	0.83–1.0
θ = 30°	3.6	Slightly rounded	0.23
θ = 40°	10	Strainer and foot valve	2.50
θ = 50°	31	9. *Gate valves*	
θ = 60°	94	Open	0.19
2. *Check (reflux) valves*		¼ closed	1.15
Ball type (fully open)	2.5–3.5	½ closed	5.6
Horizontal lift type	8–12	¾ closed	24.0
Swing check	0.6–2.3	also see Sluice gates	
Swing check (fully open)	2.5	10. *Increasers*	
3. *Contraction—sudden*		$0.25\ (v_1^2/2g - v_2^2/2g)$	
4:1 (in terms of velocities of small end)	0.42	where v_1 = velocity at small end	
2:1	0.33	11. *Miter bends* Deflection angle, θ	
4:3	0.19	5°	0.016–0.024
also see Reducers		10°	0.034–0.044
4. *Diaphragm valve*		15°	0.042–0.062
Fully open	2.3	22.5°	0.066–0.154
¾ open	2.6	30°	0.130–0.165
½ open	4.3	45°	0.236–0.320
¼ open	21.0	60°	0.471–0.684
5. *Elbow—90°*		90	1.129–1.265
Flanged—regular	0.21–0.30	12. *Obstructions in pipes* (in terms of pipe velocities)	
Flanged—long radius	0.18–0.20	Pipe to obstruction area ratio	
Intersection of two cylinders (welded pipe—not rounded)	1.25–1.8	1.1	0.21
Screwed—short radius	0.9	1.4	1.15
Screwed—medium radius	0.75	1.6	2.40
Screwed—long radius	0.60	2.0	5.55
6. *Elbow—45°*		3.0	15.0
Flanged—regular	0.20–0.30	4.0	27.3
Flanged—long radius	0.18–0.20	5.0	42.0
Screwed—regular	0.30–0.42	6.0	57.0
7. *Enlargement—sudden*		7.0	72.5
1:4 (in terms of velocities of small end)	0.92	10.0	121.0
1:2	0.56		
3:4	0.19		
also see Increasers			

(continued)

Source: Davis (2010)

TABLE C-5 *(continued)*
Hydraulic headlosses for appurtenances

Appurtenance—alphabetically	Headloss as multiple of ($v^2/2g$)
13. *Orifice meters* (in terms of velocities of pipe) Orifice to pipe diameter ratio	
0.25 (1:4)	4.8
0.33 (1:3)	2.5
0.50 (1:2)	1.0
0.67 (2:3)	0.4
0.75 (3:4)	0.24
14. *Outlet losses*	
Bell mouthed outlet	$0.1\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}\right)$
Sharp cornered outlet	$\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}\right)$
Pipe into still water or air (free discharge)	1.0
15. *Plug globe or stop valve*	
Fully open	4.0
¾ open	4.6
½ open	6.4
¼ open	780.0
16. *Reducers*	
Ordinary (in terms of velocities of small end)	0.25
Bell mouthed	0.10
Standard	0.04
Bushing or coupling	0.05–2.0
17. *Return bend (2 nos. 90°)*	
Flanged—regular	0.38
Flanged—long radius	0.25
Screwed	2.2
18. *Sluice gates*	
Contraction in conduit	0.5
Same as conduit width without top submergence	0.2
Submerged port in 12 inch wall	0.8
19. *Tees*	
Standard—bifurcating	1.5–1.8
Standard—90° turn	1.80
Standard—run of tee	0.60
Reducing—run of tee	
2:1 (based on velocities	0.90
4:1 of smaller end)	0.75
20. *Venturl meters*	
The headloss occurs mostly in and downstream of throat, but losses shown are given *in terms of velocities at inlet ends to assist in design.*	
Long tube type—throat-to-inlet diameter ratio	
0.33 (1:3)	1.0–1.2
0.50 (1:2)	0.44–0.52
0.67 (2:3)	0.25–0.30
0.75 (3:4)	0.20–0.23
Short tube type—throat-to-inlet diameter ratio	
0.33 (1:3)	2.43
0.50 (1:2)	0.72
0.67 (2:3)	0.32
0.75 (3:4)	0.24

Source: A. Amirtharajah (1978) "Design of Granular Media Filter Units," in R. L. Sanks (Ed.), *Water Treatment Plant Design*, Ann Arbor Science, Ann Arbor, Michigan, PP. 702–704.

Source: Davis (2010)

APPENDIX D INFILTRATION ALLOWANCES

TABLE 18-3
Typical infiltration allowances

	Infiltration Permitted	
Pipe diameter, mm	$m^3/d \cdot km$	$L/d \cdot mm \cdot km$
Regulations prior to 1980		
200	8 to 12	40 to 60
300	10.5 to 13.5	35 to 40
600	24 to 27	40 to 45
New pipe		
		10 to 40

Adapted from ASCE, 1982.

Source: Davis (2010)

APPENDIX E PRICES FOR EXTIMATION

Pipe unit price by diameter:

4” Gray PVC Schedule 40 Pipe	P-318	U$7.37/foot	=	U$24/meter
8” Gray PVC Schedule 40 Pipe	P-318	U$17.53/foot	=	U$56/meter

Source: United States Plastic Corp (USP). Access: April 2018.

Manhole unit price: U$4000/ manhole.

Sources: CODOT.gov, “Type T Manholes”, (1993); Riverdale City, “Sanitary Sewer Project Cost Estimates & Location Maps Appendix D,” *Riverdale City Capital Facilities Plan (2009)*

Cleanout box unit price: U$300/ cleanout box.

Sources: Riverdale City, “Sanitary Sewer Project Cost Estimates & Location Maps Appendix F,” *Riverdale City Capital Facilities Plan (2009)*