Analysis of the Recently Constructed Sewage Network of Gazimağusa

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ABSTRACT

This study covers the partial analysis of the existing Gazimağusa sewer network design and application. In this analysis, the basic hydraulic parameters were reevaluated and Lot 5 which is the part of the Gazimağusa sewer network is recalculated and compared based on this aspect. Parts for this construction, Pipe diameters, minimum, average and maximum velocity values are found to be consistent with European Union related parameters criteria except the designed pipe slopes are observed to be varying.

Due to this design, always the system (since carrying solid particles) will definitely cause clogging and will not function properly so as to meet its main purposes.

Keywords: Sewer, Slope, Velocity, Network Design, Gazimağusa

ÖZ

Bu çalışma, Gazimağusa'daki mevcut kanalizasyon tasarım ve uygulamasının kısmi analizini içermektedir. Bu analizde, tasarım için öngörülüp uygulanan hidrolik parametreler irdelenmiş ve Gazimağusa kanalizasyon sisteminin bir parçası olan Lot 5 ana hatları ile tekrar hesaplanıp karşılaştırılmıştır. Boru çapı, minimum, ortalama ve maksimum hız değerleri Avrupa Birliği konu ile ilgili parametre kriterine uyumlu bulunmuş ancak boruların eğimleri çok değişgen tasatlandığı gözlenmenmiştir. Bu tasarı neticesinde, sistemin her zaman (katı atık taşıması nedeniyle) tıkanmalara maruz kalacağı ve temel işlemini yerine getiremeyaceği gerçeği saptanmıştır.

Anahtar Kelimeler: kanalizasyon, eğim, hız, şebeke tasarımı, Gazimağusa

To my lovely father and mother,

for their endless love and encouragement.

To my dear wife(Hanan), my darling son Ali,

my darling daughter (Naba) who have always loved me and fill my life happiness.

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A : Full flow area (m^2)

a : Partially flow area (m^2)

C : Cheezy's parameter ($m^{0.5}/s$) or Hazen-Williams coefficient ($m^{0.37}/s$)

D : Pipe diameter at partially filled (m)

d : Depth of water (m)

f : Darcy – Weisbach's friction factor

F : Point load (N/m^3)

G: Ground slope (m/m)

g : Acceleration due to gravity ($9.81 m/s^2$)

H: Vertical height from top (m)

K_a: Arithmetic rate of increase for population (person/year)

K_g: Geometric rate of increase for population (person/year)

k : A functional relationship for population

k_g : Wall roughness (mm)

L : Pipe length (m)

N : Manning's roughness coefficient for full flow

n : Manning's roughness coefficient and number of years considered

P_i : Population (persons)

P_n: Future population (person)

P_o: Present population

P,p: Wetted perimeter (m)

R_h : Hydraulic radius at full flow (m)

r : coefficient of probable rate of increase of population

r_h : Hydraulic radius at partially full flow (m)

S : Head loss per unit length (m)

S_e: Slope of energy grade line (m/m)

T_i: Period, time (year)

V: Average flow velocity (m/s)

V_f: Velocity of full flow (m/s)

v : Velocity for partially full flow at depth d (m/s)

v_s : Self – cleansing velocity (m/s)

ν : The kinematic viscosity of the fluid (m^2/sec)

W_c: Vertical external load (N/m)

w : Unit weigh of fill material (kg/m^3)

Q_f : Full flow discharge (m^3/s)

q : Partial full flow discharge (m^3/s)

q_s : Self – cleansing discharge (m^3/s)

DN: Diameter of pipe (mm)

γ_w : Unit weight of water (N/m^3)

Chapter 1

INTRODUCTION

Water is the most important matter for the sustainability of life on earth and as well plays an important role in human life since the beginning of creation. Water has a clear importance of the multiple uses in various fields of agricultural, industrial and municipal requirements. Failure to maintain the natural resources causes increase in pollution of air, water and soil significantly.

Waste water in developing countries is one of the reasons of pollution that directly affects the human health, animals' life and the beauty of nature.

Waste water (sewage) basically contains two types of materials:

i- organic materials (can be nitrogenous (urea, protein) or non-nitrogen (carbohydrates, fats, soaps) and are existing by attaching on objects

ii- Inorganic materials (like sand, clay, salts, etc…)

Municipal waste water contains large proportion of liquid and some solids. These solids are either stuck or dissolved with rate (0.1%) and liquid with (99.9%).

The sewage can be classified into three due to its characteristics:

1. Physical (solids content, smell, color, temperature, etc…),

2. Chemical (pH, chloride content, nitrogen content, content of oils and, fats, BOD, COD, sulfur, H_2S, etc…),

3. Biological (various micro-organisms, bacteria, viruses, etc…).

After the industrial development that had occurred in the world after the nineteenth century, large industrial gatherings and densely populated major cities began to appear. This brings the need for collection and treatment of wastewater to control health. As the time proceeds proper collecting and treatment plants come into the use. The aim of these plants is attempt to clean the waste water that will not cause pollution. Hence, under the controlled conditions, the self-purification methodologies are applied to simulate the natural conditions.

The sewage treatment plant composed of several physical, chemical and biological parts. Their volume and processing capacities are mainly depends on the population density and type of waste water.

Any treatment plant contains:

1. Analysis of organic materials,
2. Suspended solids removal large refineries,
3. Grease and fat by skim removal,
4. Outstanding soft material removal by precipitation and filtration,
5. Sludge treatment.

Sewage in transmitted is a closed conduit called as a sewer, which normally flows partially filled. When the storm water is intended to be excluded, the system is called a separate sewer. On the other hand, the combined sewer is one intended to receive domestic sewage, industrial wastes and storm water. Hydraulic calculations are the most important part in make this system successful during the project; therefore this study (Gazimağusa sewer network) is analyzed.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

There are three main resources of wastewater namely, industrial, residential and commercial from which wastewater is gathered and brought to treatment plants by means of sewer systems. Afterwards, it undergoes the process of treatment in the plant as a result of which it can be reused or by several processes its quality is improved to a desirable level at which it could be disposed to receiving water. The hydraulic sewer system design has not altered significantly in the last decade. The management and construction process of sewer systems has undergone noticeable revisions during the same period (Sharma & Swamee , 2013).

2.2 Channel Water Resistance

There have been several published investigations on different aspects of sewer system design including different methods for design, optimization, management and cost minimization. Manning equation and Hazen-Williams equation have been used widely for the aforementioned investigations. However, in 1963, ASCE task force announced that for open channel resistance, the manning equation is not acceptable. In another study in 1984, Christensen highlighted the limitation of Manning equation which ranges from 0.004 to 0.04 for relative roughness. Therefore, they proposed, Darcy–Weisbach equation to be used afterwards (ASCE, 1963). On the other hand, according to another study by Liou (1998), Hazen–Williams equation has been disapproved for open channel resistance calculation purposes due to several limitations which were mentioned in his study. Besides, in 2002, Fredrich & Rogers performed a comprehensive historical analysis on the usage of Darcy–Weisbach equation for the design stage, and concluded that the aforesaid formula is the most accurate and comprehensive equation to be used for channel water resistance.

In this study, Manning equation is utilized, as it is the most widely used and approved by researchers in the field, and most of the literature were published using aforesaid equation. All these approaches use the Manning equation or Hazen-Williams equation for resistance description (Swamee, 2001).

2.3 Sewer System Design Elements

There are several aspects which should be considered carefully prior to the initiation of sewer system design process. The most important ones are carrying out initial investigations, considering design criteria based on every special case study, population and context, preliminary sewer system preparation, designing every single sewer and contract drawing preparation.“Comprehensive preliminary investigations of the area to be served are required not only to obtain the data needed for design and construction but also to record pertinent information about the local conditions before construction begins”(Alemayehu, 2008).

Sharma, Bhargava&Swamee (1987) demonstrated a method “for the determination of sewer geometry of circular and noncircular shapes for partly full-flowing conditions with the known variables being discharge, bed slope, and Manning’s roughness coefficient”. The appointment of sewer diameter is a long process, tabular approach is not accurate enough and graphical method involves personal mistakes (Sharma, Bhargava, & Swamee, 1987). They also specified that both minimum and maximum velocity requirements must be able to be satisfied by the design criteria. For non-circular sewer pipes which is based on hydraulic balance, a transformation of equivalent circular diameter is indispensable, and the process is rather sophisticated and time consuming.

Swamee, (2001) proposed an algorithm for sewer system design which claimed to be optimal. For simplification purposes and to exclude design variables, the resistance equation was employed in his work “whereas the over-fall depth constraint was dealt with by the Lagrange-multiplier method”.He overcame the problems of sophisticated constraints by splitting the sewer line. Aforesaid constraints included “minimum cover, maximum depth, and minimum average velocity constraints. Reducing the slope and providing a drop satisfied the maximum velocity constraint” (Swamee, 2001).

In a similar report by Alemayehu (2008), the proper option for hydraulic design equation, different available materials for sewer pipes, boundaries of size, velocity and slopes, flow rates of waste, local condition specific of case study and will affect the system operation and alternative alignment were highlighted. He also proposed that sewer size should not be less than twenty centimeter to prevent clogging. The minimum velocity of 0.6 meter per second was also mentioned to be ensured.

2.4 Sewer System Capacity

In order to determine the capacity of municipal sewerage systems for an uncertain projection, the method of adopting excess value in the design flow has generally been used. The design process has normally divided into two main categories. In the first model, it has been decided that the extra capacity may not necessarily be adopted for the sewerage system as additional capacity for even a collection system estimated to be as little as 15%. The second category the efficiency in construction is highlighted and it was established that excess value in design flow can be eliminated (Ichimura & Nakanishi, 1987).

Considering the design capacity, sewer systems was divided into two main categories by Kumar (2012) namely separate and combined. Separate sewer system comprises two types of pipes from one of which the storm-water in transported to treatment system and from the second transported the wastewater. The combined system on the other hand, is the combination of channels for storm water runoffs and sanitary sewage. “This allows the sanitary sewer system to provide backup capacity for the runoff sewer when runoff volumes are unusually high, but it is an antiquated system that is vulnerable to sanitary sewer overflow during peak rainfall events” (Kumar, 2012).

It a report by national program on technology enhanced learning carried out by the government of India, the wastewater quantity estimation reported to be based on storm free scenario in dry seasons which is called dry weather flow. It has been reported that as much as 80% of water supply reaches sewer system. Besides, the maximum daily flow suggested to be two times the average daily flow and the minimum is two third of the average.

They used population equivalent which is used in “conversion of contribution of wastes from industrial establishments for accepting into sanitary sewer systems” (NPTEL, 2013). Several methods for population forecast were also mentioned in their report such as arithmetic, incremental and geometric increase method, logistic curve and ratio method and comparative and simple graphical method.

Chapter 3

SANITARY ENGINEERING

3.1 Introduction

The sewage network system played an active role in transferring waste water from residential buildings, houses, municipal buildings, factories and from all the other utilities in any urban and rural areas. This system consist of large set of different diameters pipes that are designed to collect waste water under non-pressurized regime and disposing it to any waste treatment plant. The cost of this system including design and implementation is very expensive although, the implementation of this system is very important due to enormous preservation causing healthy environment and healthy living organisms.

3.2 Important Terms and Definitions

Refuse: general term used to indicate what is rejected or left out as worthless. It has six components:

i- Garbage: indicating dry refuse. It contains large amounts of organic and putrid type matter like waste paper, decayed vegetables and fruits, grass and sweepings from streets, market and other public placed.

ii- Rubbish: all sun-dry solid wastes that are combustible in nature like broken furniture, paper, rags etc.

iii- Silage: waste water from bathrooms, kitchens, wet places etc. that does not create bad smell. The organic matters content is either absent or is of negligible amount.

iv- Sewage: liquid waste from the community like sludge, discharges from latrines, urinals, industrial waste and also the ground surface with storm water that may be inter into the sewer. It's decomposition produces large quantities of malodorous gases, and it may contain numerous pathogenic or disease causing bacteria.

v- Sub-Soil Water: the ground water that is leaks into sewers through leaks.

vi- Storm water: it is the rain water that flows over.

Sanitary Sewage: denotes sewage mainly derived from the residential buildings and industrial establishments. It may be classified as:

a) Domestic Sewage: It is the sewage collected from latrines, urinals and lavatory basins, and other institutions. It's awfully foul in nature.

b) Industrial Sewage: It is the sewage collected from the industrial and commercial establishments.

Sewer: it is an underground channel or drain through which sewage is carried to a point of discharge or disposal. It can be classified as combined sewers, separate sewers, and partially combined sewers. Sewers are classified and named as below:

1) House Sewer (or Drain): Is a pipe that carries away the sewage from any building to a street sewer (lateral sewer).

2) Main Sewer or Trunk Sewer: Is a sewer that receives sewage from many tributary branches and sewers and serving as an outlet for large territories.

3) Branch Sewer (or Sub main Sewer): Is a sewer which receives sewage from relatively small areas (usually few laterals) and discharges into a main sewer.
4) Lateral Sewer: Is a sewer which collects sewage directly from the houses.
5) Depressed Sewer: Is a section of sewer constructed lower than adjacent sections and passes beneath an obstacle or obstruction.
6) Intercepting Sewer: Is a sewer that laid transversely to general sewer system so as to control the wet-weather conditions.
7) Out Fall Sewer: Is a sewer that receives the sewage from the collection system and transmits it to a point of final discharge or to a disposal plant.
8) Relief Sewer (Overflow Sewer): Is a sewer built to carry the flow in excess of the capacity of an existing sewer.

Sewerage: It is a structures, devices, equipment, and appurtenances intended for the collection, transportation and pumping of sewage and liquid wastes excluding works for the treatment of sewage. It deals with the entire hydraulic details of collecting and carrying sewage.

Wastewater: It is the synonym of sewage. Implies organic and minerals carried through liquid media.

Soil type:

- Top soil: means any surface material suitable for use in soiling areas to be grassed or cultivated.
- Subsoil: any material other than topsoil and rock.
- Rock: is defined as material occurring in solid un-weathered banks or layers which, in the opinion of the Engineer, can only be removed by blasting, percussion drilling, wedging or splitting.

Back fill materials

- Unsuitable Material implying material not suitable for backfilling including materials from swamps, organic and perishable materials, clay and soils with high plasticity indices such as Liquid Limit (LL) > 80 and Plasticity Index (PI) > 55.
- Rock Fill Material implying hard unweathered material of suitable sizes for deposition and compaction that may comprise broken stone, hard brick, concrete or other hard inert materials.
- Selected Fill Materials implying backfilling trenches and foundations that comprise well graded readily compactable material free from roots, vegetable matter, building rubbish and clay lumps.

3.3 Wastewater Transmitting System

Sewage transferring operation which is composed of domestic sewage, industrial sewage, and storm water can be done by three different ways:

3.3.1 Separate Sewer System

This system consists of two separate sewer systems; one for collecting and transferring storm water and the other collecting and transferring domestic sewage with industrial sewage. The significant advantages and disadvantages of this system are:

Advantages:

1. This system uses small diameter size of pipes,
2. Transferred sewage volume is small,

3. The disposal of storm water to the rivers and paracentes is directly done without need to treatment.

Disadvantages:

1. Easily get clogged due to small sizes,
2. Cost of design and implementation is high.

3.3.2 Combined System:

This system has one sewer pipe that collects domestic sewer, industrial sewer, and storm water and transfers it up to the treatment plant before disposing it to the rivers or open channels. Followings are the advantages and disadvantages of this system:

Advantages:

1. Low clogging probability,
2. Does not occupy wide areas,
3. The intensity and concentration of organic and chemical materials in the sewage decreases during wet seasons because of the storm water component.

Disadvantages:

1. Cause difficulty during transportation of pipes due to their large diameters,
2. Mixing of storm water with the existing sewage results an increase in sewage volume that is being transferred to the treatment plant, and this creates a higher sewage volume on the plant.
3. In dry seasons, because of relatively low flow velocity causes a high possibility of organic material sedimentation within in sewer pipes.
4. This system uses large diameters size pipes.

3.3.3 Partially Combined System

This system is the combination of previously mentioned two systems where some parts of the area having combined system, and some other parts having separate system. Purpose of using combined system here is to divert the excess sewage during heavy storms. Followings are some of the advantages and disadvantages of this system:

Advantages:

1. Pipes have suitable diameters sizes,
2. Low probability of sedimentation,
3. Facilitate the disposal of cumulative storm water within the served area.

Disadvantages:

1. Design and implementation of this system has a very high cost,
2. Some parts of this system may suffer from low flow velocity during dry seasons.

3.4 Selecting a Suitable Sewer System

There are many effecting factors that must be studied properly and adequately before taking a suitable decision for selecting the sewer system type. After the display of advantages and disadvantages for all possible systems, it will get clearer to select or reject a certain system type.

3.4.1 Why to Use Separate System?

1. It is economical, where it can be implemented in flat areas of low slope values.
2. It is appropriate for those areas that have high urbanization expansion.
3. It is recommended where there are discharging areas like rivers, lakes and natural channels.

3.4.2 Why to Use Combined System?

1. It is appropriate if the region exposed to a large volume of rain.
2. Where a need of continuous pumping is required.
3. If within the existing area the earthen area is limited and\or crowded by general utilities.
4. If location wise turns out that the liquid waste and rainwater has to be discharged from one point.
5. If the temperature is high and there is a high risk of decomposition during transfer.

3.5 Estimation of Waste Water Quantity

When designing the sewer system pipes and wastewater treatment plant, it is better to identify the current amount of wastewater within this system. This includes waste water from domestic and industrial uses as well as from storm water. So the components of sewage water quantities are:

3.5.1 Domestic Sewage

It is all sewages that result from residential buildings, houses, and utilities buildings. It contains organic and non-organic matters.

3.5.2 Industrial Sewage

It is all sewages that result from small factories or general building. It contains different rate of organic and non-organic matters depending on the type of factory inputs.

3.5.3 Storm Water Sewage

It is all sewages resulting from washed water of streets mainly due rainfall, excess watering of gardens, due extinguishing of fires. It contains dusts, sands, and organic matters that result from streets surfaces and open regions.

3.6 Factors Affecting the Amount of Liquid Phase Sewage

3.6.1 Infiltration

The amount of infiltration directly associated with groundwater level found in soil layers. It can be expressed as:

- Liter/Hectare/Day: the area of the region serving for sewage system pipes accounted by hectare unit. Infiltration amount is obtained from multiplying infiltration rate with the magnitude of the related area.
- Liter/Kilometer/Day: In this case, infiltration amount accounted for every kilometer length of sewer pipes.
- Liter/Centimeter of pipe diameter/Kilometer of pipe length)/Day: It is the best method that accounts the expression of infiltration amount accurately since it involves the pipe diameter and pipe length.

The amount of infiltration depends on the following parameters:

1. Depth of sewer pipes level comparing with ground water level. The pipes level placed deeper than ground water level causes a larger amount of infiltration.
2. Diameters and lengths of pipes. As diameter and length of pipe gets larger the infiltration amount increases.
3. Quality of materials used in the manufacture of pipes.

4. The nature of the soil.
5. Types of joints used and their qualities.
6. Labors skill.

3.6.2 Quantity of Used Water

To determine the amount of liquid phase sewage, the water supply for the individual during a day should be known as accurate as possible. Since it is difficult to determine the factors effecting in consumption of supply water, therefore there is a difficulty in suggestion a simple bases for the estimation; especially in suggestion estimation based on the factors depending on living standards and the regional variations caused by different daily and seasonal needs. Recently studies found that 60% - 80% of the water consumption of individual converts to sewage.

3.6.3 Population Growth

Minimum population rate of two to three decades is necessary for the estimation of sewer and treatment plant capacities. The widely accepted population density is tabulated below.

Table 3.1: Population Density

Size of town (population)	**Population density per hectare**
Up to 5000	75-150
5000-20000	150-250
20000-50000	250-300
50000-100000	300-350
Above 100000	350-1000

To estimate the population growth rate there are different methods:

a) Arithmetic Method

It is used when the average population growth is stable (not changes with years). The mathematical expression for the arithmetic population growth rate is:

$$K_a = \frac{dp}{dt} \tag{3.1}$$

$$K_a = \frac{\int_{P_i}^{P_f} dp}{\int_{t_i}^{t_f} dt} \tag{3.2}$$

$$K_a = \frac{P_f - P_i}{t_f - t_r} \tag{3.3}$$

where;

K_a : arithmetic population growth rate during the required time period [population/time]

P_i : population at the beginning of the period,

P_f : population at the end of the period,

t_f : beginning year of the period,

t_r : ending year of the period.

b) Graphical Method

In this approach a graph of showing the relationship between the population over the years is drawn and then extrapolated with a consensus to the future that fits the nature of the population growth in the past. Hence the population can be estimated for any year of future.

c) Geometric Method

This method is based on the assumption that the population growth varies with the proportion of the population through a fixed unit of time logarithmically. The geometric constant K_b is expressed by:

$$K_b = \frac{\text{Ln } P_f - \text{Ln } P_i}{t_f - t_i} \tag{3.4}$$

d) Comparative Method

This method is based on the study of the nature of population growth by comparing several cities of similar circumstances like socio-economic, politic etc… and tries to predict the population with the help of drawn curves.

e) Ratio and Correlation Method

In this method, the population growth of any area that is located within the city is studied and the expression is given by:

$$K_r = \frac{\overline{P}_f}{P_f} = \frac{\overline{P}_i}{P_i} \tag{3.5}$$

Where,

K_r : growth constant

P_i : expected population of the area within the city according to last census,

$\overline{P}_i$: existing population of the area within the city according to last census,

P_f : expected population of the city according to last census,

$\overline{P}_f$: existing population of city according to last census.

f) Component Method

This method studies the main reasons of population rate such as births, mortalities, and migrations. It does not give accurate results due to the difficulties of gathering appropriate information [Alhashmi, 1992].

g) The Turkish Bank of Provinces Method

This method is widely used in Turkey. It is expressed as:

$$P_n = P_o(1+r)^n \tag{3.6}$$

Where,

P_n : future population,

P_o : present population,

r : probable rate of increase per year,

n : number of years considered.

The equation below is established for the population of Turkish cities based on 1945 census. The growth rate, K is found by:

$$K = \left[(T_2 - 1945)\sqrt{\frac{P_2}{P_{1945}}} - 1 \right] * 100 \tag{3.7}$$

Limitations:

- If K > 3, then take K=3
- If K < 1, then take K=1
- If 1 < K < 3, then take K as it is.

Population in the future P_n at year T_n is calculate from

$$P_n = P_2\left(1+\frac{K}{100}\right)^{(T_n - T_2)} \tag{3.8}$$

Where,

P_2 : population of the last census in year T_2 [yanmaz, 2006].

3.6.4 Characteristics of the Served Region

The determination of the liquid sewage amount depends on the type of served area. However, the industrial and commercial areas characterized by their rapid development and interoperability, while residential areas characterized by consumption of supply water per individual.

3.7 Calculation of Liquid Phase Sewage Amount

The proportion of the liquid phase sewage amount has two components; the household and industrial that is estimated to be 65% - 75% of the daily consumption of municipal water needs and the infiltration which is estimated to be 0.1 l/ha/sec.

3.7.1 Households and Industrial Sewage Amount

The limiting conditions are:

i- Upper Limit of the discharge

The suggested formula is:

$$M_1 = \frac{4.8}{P^{1/6}} * n \qquad (3.9)$$

Where,

M_1: the upper limit ratio of average discharge,

P : the number of population in thousands, n : seasonally varying factor.

ii- Lower Limit of Discharge

The following equation is suggested:

$$M_2 = 0.2P^{1/6} \tag{3.10}$$

Where,

M_2 : the lower limit ratio of average discharge [Alhashmi,1992].

3.7.2 Rain Water Amount

Parameters influencing this quantity

- Rainfall intensity,
- Frequency of the rain,
- The concentration period,
- Area under consideration,
- The runoff factor `C`.

There are two common methods available in literature are used for the estimation about quantities:

a- Rational Method

$$Q = 0.278\ CiA \tag{3.11}$$

b- Burkli – Ziegler Formula

$$Q = 296\,Ci\sqrt{A^3 S} \tag{3.12}$$

c- McMath’s Formula

$$Q=292\,Ci\,\sqrt[5]{A^4\,S} \tag{3.13}$$

Where,

Q: discharge [m^3/sec],

A: area [km^2],

i: intensity of rain [mm/hr],

C: runoff coefficient,

S: slope of the area.

Chapter 4

SEWER-PIPE MATERIALS AND APPURTENANCES

4.1 Introduction

Sewer pipes are broadly classified as either rigged or flexible. The type of pipe material to be used in any particular case is controlled by several factors:

1- The type of wastewater to be transported,

2- The scour and abrasion condition,

3- The installation requirements,

4- The type of soil,

5- The trench-load conditions,

6- The bedding and existing backfill materials,

7- The infiltration and ex-filtration,

8- The cost effectiveness.

4.2 System Layout

The system layout for sewer network depends on:

1- The selection of the outlet,

2- The existing tributary area,

3- The location of the trunk and main sewers,

4- The location of pumping stations and power mains, 5- The location of the underground rock formations,

6- The location of underground existing utilities like water and gas lines etc.

4.3 Materials of Sewers

Up until recent years the sewer pipe materials were made up different materials. These are:

4.3.1 Asbestos Pipes

The sewer pipes are made from a mixture of asbestos, fiber, cement and silica. These pipes are available in various sizes ranging from 75 to 914 mm in diameter and 3 – 4m in length.

4.3.2 Plain or Reinforced Concrete Pipes

Sizes 80 to 610 mm plain concrete and for sizes larger than 610 mm reinforced concrete pipes should be used. The manufactured reinforced pipe sizes are 305 – 4570 mm.

4.3.3 Brick Pipes

Bricks are being used for sewer pipes since early days. To be appropriate the sewer pipes should be plastered from outer site so as to make it impervious against ground water whereas inner part should be stoneware or ceramic so as to reduce friction against flows.

4.3.4 Cast Iron Pipes

Cast iron pipes have high strength. They are structurally strong so as to withstand tensile, compressive as well as bending stresses. They are available easily in the markets from 150 mm – 750 mm in diameter and up to 3 – 3.5m in length.

4.3.5 Steel Pipes

It is used at those locations where high external and/or internal pressures are encountered. They are used for mains, outfall and trunk sewers having large diameters. It has a high resistance to corrosion.

4.3.6 Plastic Pipes

The use of plastics for sewer pipes nowadays are very popular especially recommended for domestic sewers at wet region. Such pipes have high hydraulic efficiency, thus permitting flatter slopes because of very low coefficient of friction. They never experiences corrosion. They are in longer lengths and can be joined easily. These pipes are more flexible and permitting cold bends.

4.4 Sewer Appurtenances

Sewer appurtenances are those structures of the sewerage system that are constructed at suitable intervals and wherever requires along the sewer line. Important sewer appurtenances are:

4.4.1 Inlets

It is a device meant to admit the storm water or surface wash and convey it into a storm sewer or a combined sewer. Inlets are placed at the gutters as shown in Figure 4.1.

Figure 4.1: Typical Inlet Structure

4.4.2 Catch Basins or Pits

It is a special type of inlet, in which a basin is provided with grit where sand and debris etc. can be deposited. The outlet is usually trapped to prevent escape of odors from the sewers and to retain floating matter. There are two types of catch basins the combined gutter and the curb inlet. Figure 4.2 shows these details.

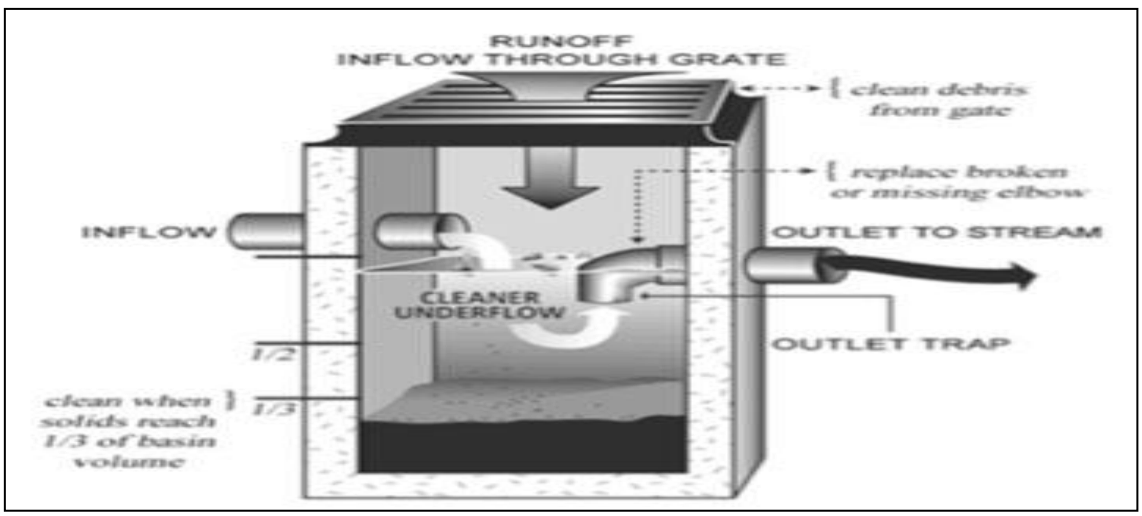

Figure 4.2: Catch Basin

4.4.3 Clean Outs

It is an inclined pipe with its one end connected to the underground sewer line and the other end brought up to ground level with a proper cover at the top so as to used for cleaning purposes whenever is needed as shown in Figure 4.3.

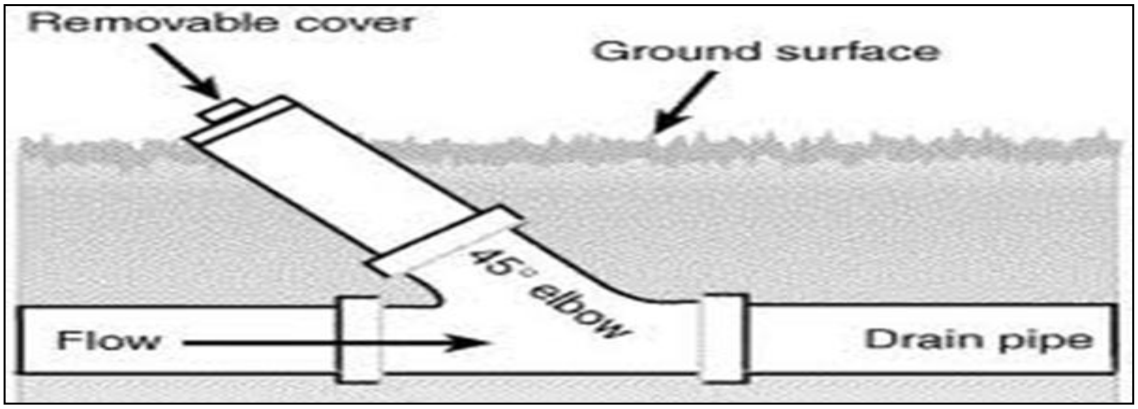

Figure 4.3: Clean Out Pipe

4.4.4 Manholes

A manhole is a masonry or reinforced concrete chamber constructed on the alignment of the sewer for providing access to the sewer for the purpose of inspection, testing, cleaning and removing obstructions along the sewer line.

4.4.4.1 Location of Manholes

They are placed at interval of 90 – 120 m and wherever there is a change in the direction, in the gradient, and in diameters of pipes. They usually constructed cylindrical of diameter 700 mm – 1200 mm. Spacing of the manholes varies. Table 4.1 details the maximum manhole spacing.

Table 4.1: Maximum Manhole Spacing in Europe and Turkey

Pipe diameter (cm)	**Manhole Spacing**	
	in Europe (m)	**in Turkey (m)**
20 – 25	60 – 75	50
30 – 35	65 – 80	50
40 – 45	70 – 80	50
50 – 60	75 – 90	70
60 – 80	80 – 100	70
90 – 140	80 – 100	90
140 <	80 – 100	125 - 150

4.4.4.2 Classification of Manholes

Manholes are classified according to their depths:

1- Shallow Manholes

These manholes have depth about 0.75 – 0.90 m, and they are constructed at the start of the sewer branch.

2- Normal Manholes

These manholes are about 1.5 m in depth. They are constructed either in square 1.0 m * 1.0 m or rectangular 0.8 m * 1.2 m in cross – section.

3- Deep Manholes

The manholes are deeper than 1.5 m where the size of such a manhole is larger at the bottom and get decreased at the toper part so as to reduce size of manhole cover as shown is Figure 4.4.

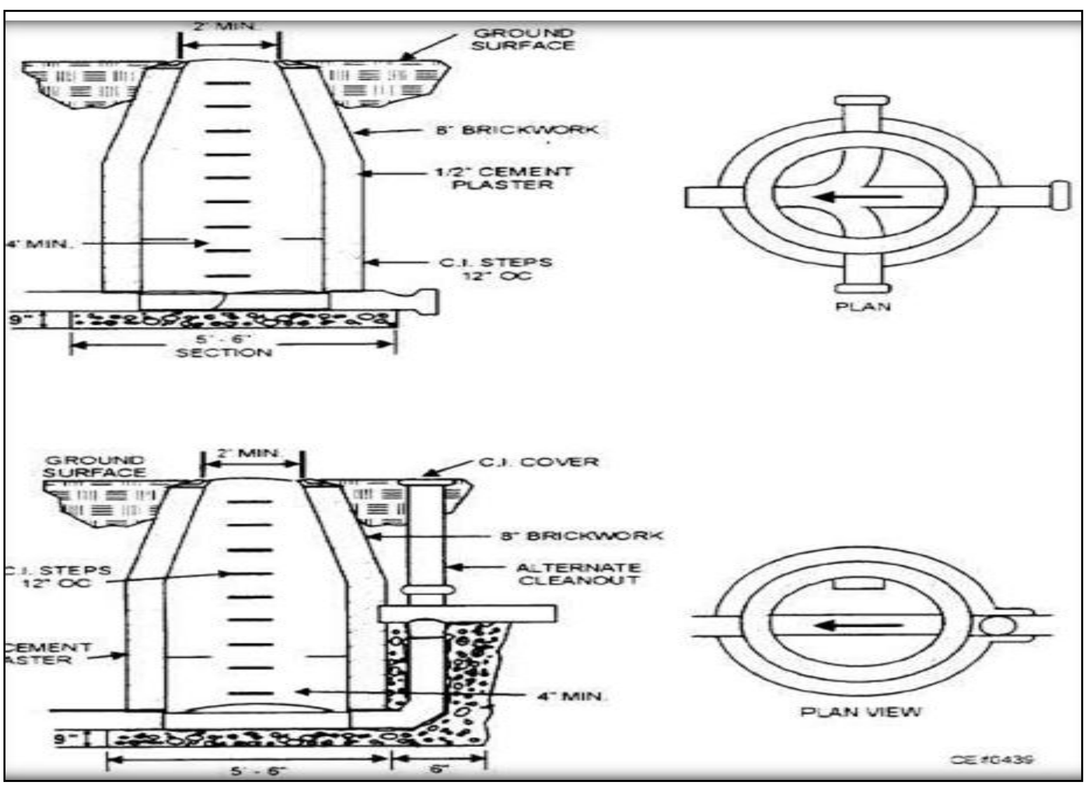

Figure 4.4: Manhole Cross-Section Details

4- Drop Manhole

The drop manhole is a special type of manhole which is constructed to provide connection between high level branch sewers to low level main sewer.

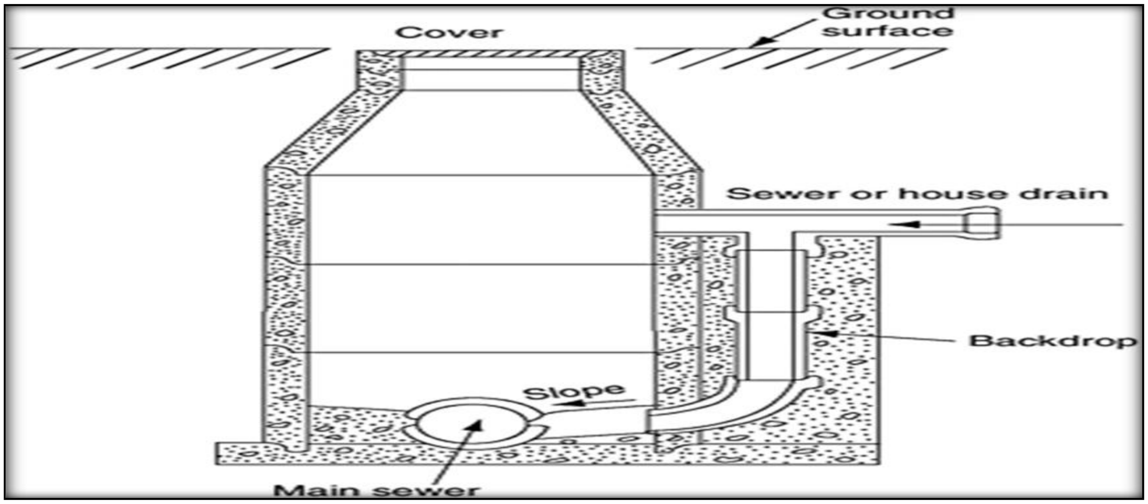

Figure 4.5: Drop Manhole

4.4.5 Flushing Tanks

It is an arrangement which holds water and then throws into the sewer for the purpose of flushing. Flushing may achieved, either hand operated or automatic.

4.4.6 Grease and Oil Traps

Grease and oil traps are specially built chambers on the sewers to exclude grease and oil from sewage system before they enter to the sewer line. Figure 4.6 shows typical grease and oil traps.

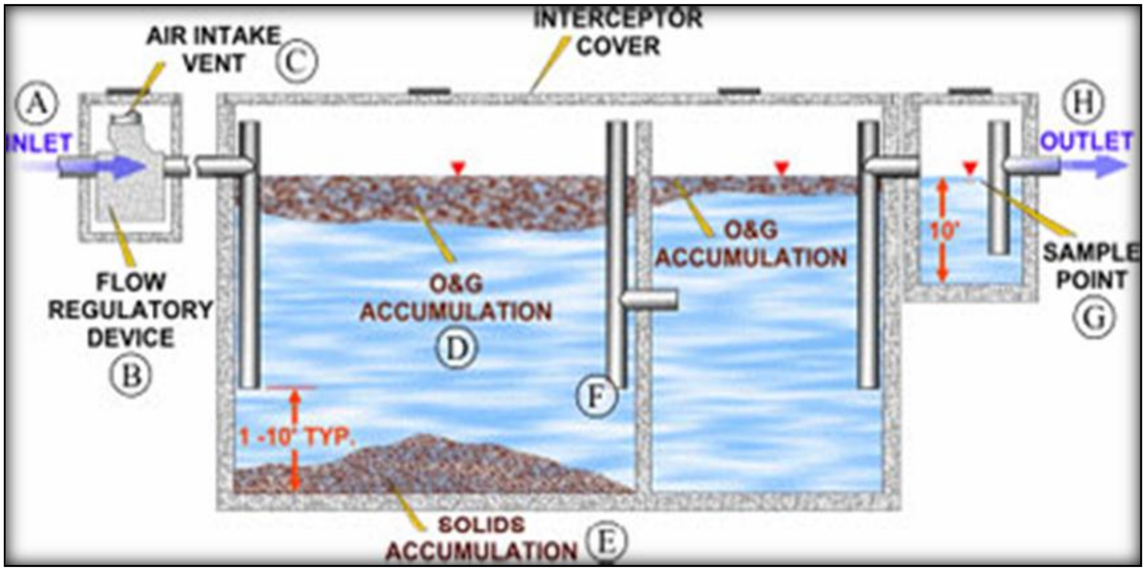

Figure 4.6: Grease and Oil Traps

4.4.7 Inverted Siphons

When a sewer line reaches below the hydraulic grade line or if there is an obstruction such as roadway, railway, river etc., these pipes constructed so as to carry the sewer under them by forces the system to regain as much elevation as possible.

4.5. Sewage Pumping

The necessity of lifting sewage arises under the following circumstances:

a- Where some area of a urban or rural area having low elevation and cannot drained by gravity to discharge within the sewer system.

b- When sewers has to go across a high ridge.

4.5.1 Pumping Stations

4.5.1.1 Location of Pumping Stations

Proper location of pumping station requires a comprehensive study of the area under consideration so as to ensure that, the entire area can be adequately drained. The pumping station should be located and constructed in such a manner that it will not be flooded at any time. The pumping station should be so located that it can easily accessible under all weather conditions [Punmia,1998].

4.5.1.2 Elements of Pumping Stations

A sewage pumping station consists:

i- Grit channel,

ii- Coarse and fine screens,

iii- Sump and wet well,

iv- Pump room or dry well,

v- Pumps with driving engine or motor,

vi- Miscellaneous accessories such as pipes, valves, floating switches etc.

4.5.2 Types of Pumps

There are different types of pumps commonly used for sewage and storm water systems [Punmia,1998]:

1- Centrifugal Pumps

It is the most widely used for sewage and storm waters. These can be easily installed in pits and sumps, and can easily transport the suspended matter existing within the sewage without getting clogged too often. There are three classes of centrifugal pumps:

a- Disintegrator pumps,

b- Full way pumps,

c- Free way pumps.

2- Reciprocating Pumps

There are more or less absolute in modern sewage pumping station since they are liable to be clogged by solids or fibrous material, even though sewage may have passed through coarse screens.

3- Propeller or Axial Flow Pumps

Its impeller resembles the propeller of a ship. The efficiency of axial flow pump is very low up to the extent of about 25 %. These pumps are suitable to pump large volume of sewage against low head.

4- Air Pressure Pumps or Pneumatic Ejectors

Pneumatic ejector works on the action of compressed air, and its use where the small quantities of sewage is to be lifted from basements of building, and where the quantity of waste water from a low – lying area does not justify the construction of pumping station and where the centrifugal pumps of smaller capacity are likely to be clogged.

Chapter 5

HYDRAULIC DESIGN OF SEWERS

5.1 Theory

Sewage is mostly liquid containing hardly 0.1 to 0.2 percent of solid matter in the form of mainly organic matter with sediments and minerals. These solid particles settle at the bottom and have to be dragged during the sewage transport

Sewer design mainly based on gravity flow where considered as open channel system that run under gravity. The sewer infrequently run full and the hydraulic gradient line falls within the sewer. One of the key elements in the hydraulic design is to calculate the peak flow carried by runoff pipes at full capacity with enough velocity to prevent sedimentation and erosion. The design hence, includes selecting a suitable gradient value that causes self-cleansing. Manhole has to be added to control gradient changes and at the entrance to the pipe. Usually, the size of the pipe is not found with the same properties of the design diameter for optimal depth. So selecting proper pipe size for that amount of flow is the basis of engineering design. In practice using one bigger size or modifying the slope or redesign for both is unavoidable. This directly affects the cost of pipe and cost of excavation.

The depth of a sanitary sewer should be sufficient so that all house sewer connected to it will drain by gravity. A depth of 2.0 - 2.5 m below ground surface is usually sufficient.

Factors affecting the flow of sewer:

1- Sewer slope,

2- Geometry of sewer pipe.

3- Roughness of the inner surface of sewer pipe,

4- Bends, transition and obstructions,

5- Flow conditions,

6- Pressure characteristic of sewage.

5.2 Hydraulic Design Formulas

The hydraulics designs of sewer depend mainly on open channel theory. To satisfy that; partially full pipes under gravity flow will be assumed for design. Widely used hydraulic design formulas are:

5.2.1 Chezy's Formula

This empirical formula is determined in 1775.

$$v = C\sqrt{R*S} \tag{5.1}$$

v = velocity of the flow (m/sec)
S = hydraulic gradient or slope of the sewer (m/m)
R = hydraulic mean radius R=A/P (m)
A = wetted cross sectional area (m^2)
P = wetted perimeter (m)
C = Chezy's constant. It is difficult to determine "C" in nature. The value of Chezy's constant found by Kutter's or by Basin's formula.

- Kutter's formula

$$C = \frac{23 + \frac{0.00155}{S} + \frac{1}{N}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{R}} \tag{5.2}$$

R = hydraulic mean radius (m)

S = hydraulic gradient or slope of the sewer (m/m)
N = Rugosity coefficient, depend upon the nature of inside surface of the sewer

- Bazin's formula

$$C = \frac{157.6}{1.81 + \frac{K}{\sqrt{R}}} \qquad (5.3)$$

K= Bazin's constant, the value can be selected from Table 3.1.

Table 5.1: Bazin Constant 'K'

#	Condition of interior service	K
1-	Very smooth surface	0.109
2-	Smooth brick or concrete surface	0.29
3-	Rough brick or concrete surface	0.833
4-	Smooth rubble masonry surface	1.54
5-	Good earthen channels	0.50
6-	Rough earthen channels	3.17

5.2.2 Manning's Formula

Robert Manning suggested in 1890 and in USA as most of the developing countries.

$$v = \frac{1}{n} * R^{2\backslash 3} * S^{1/2} \qquad \text{(SI unit)} \qquad (5.4)$$

v = velocity of sewer flow (m/sec)

S = hydraulic gradient or slope of the sewer (m/m)

R = hydraulic mean radius R = A/P (m)

The flow rate in circular pipe running full

$$R = A/P = \frac{D}{4} \qquad (5.5)$$

$$Q = \frac{0.463}{n} * D^{8/3} * S^{1/2} \tag{5.6}$$

Q = discharge (m^3/sec),

n = Manning's coefficient varies with pipe types can be selected from Table 5.2.

Table 5.2: Manning's Coefficient for Different Materials

Conduit material	Condition of interior service	
	Good	Fair
1- salt glassed stone ware	0.012	0.014
2- cement concrete	0.013	0.015
3- cast iron	0.012	0.013
4- brick, unglazed	0.013	0.015
5- asbestos cement	0.011	0.012
6- plastic smooth	0.011	0.011

5.2.3 Crimp and Bruges Formula

This empirical formula commonly used in England.

$$v = 83.47 * R^{2/3} * S^{1/2} \tag{5.7}$$

Comparing this formula with Manning formula

$$v = 83.47 * R^{2/3} * S^{1/2} = \frac{R^{2/3} * S^{1/2}}{n} \tag{5.8}$$

gives n = 1/83.47 = 0.012

Hence, Crimp and Bruges equation is a very specific case of Manning equation where n = 0.012

For circular pipe full flow

$$R = A/P = \frac{D}{4} \tag{5.9}$$

$$v = 83.47*(\frac{D}{4})^{2/3} * S^{1/2} \qquad (5.10)$$

$$Q = 26.02 * D^{8/3} * S^{1/2} \qquad (5.11)$$

5.2.4 Hazen and Williams Formula

After 1902, this empirical formula most commonly used for pressurized pipes.

$$v = 0.85 * C * R^{0.63} * S^{0.54} \quad \text{(SI unit)} \qquad (5.12)$$

If (D/4) is substituted for the hydraulic radius R, the flow rate Q becomes

$$Q = 0.278 * C * D^{2.63} * S^{0.54} \quad \text{(SI unit)} \qquad (5.13)$$

Where, Q is the flow rate (m^3/sec).

The coefficient (C) is given in Table 5.3.

Table 5.3: Coefficient 'C' for Hazen and Williams Formula

#	Types of material	C
1-	Steel pipe under future conditions	95
2-	Old Cast Iron pipe	100
3-	Brick sewer good condition	110
4-	Stone ware in good condition	110
5-	Cement lined pipes	110
6-	New riveted steel pipe	120
7-	Wood stave pipe	130
8-	New Cast Iron pipe	140
9-	Pipe with very smooth surface	140

5.3 Velocity of Flow

5.3.1 Minimum Velocity

Such a minimum velocity is known as self – cleansing velocity. It may be defined as that velocity at which, the solid particles will remain in suspension without settling at the bottom at the sewer. Self – cleansing velocity should be maintained at least once even for short period of time for each day during the working life of the pipe.

5.3.2 Maximum Velocity

It is such a velocity that causes no scouring action or abrasion and known as non–scouring velocity. It depends upon the material used for the construction of sewer. ASCE 1982 recommends that flow velocity in sewers shouldn't be less than 0.6 m/s or greater than 3.5 m/sec.

Graphs and charts are available for non-scouring velocities.

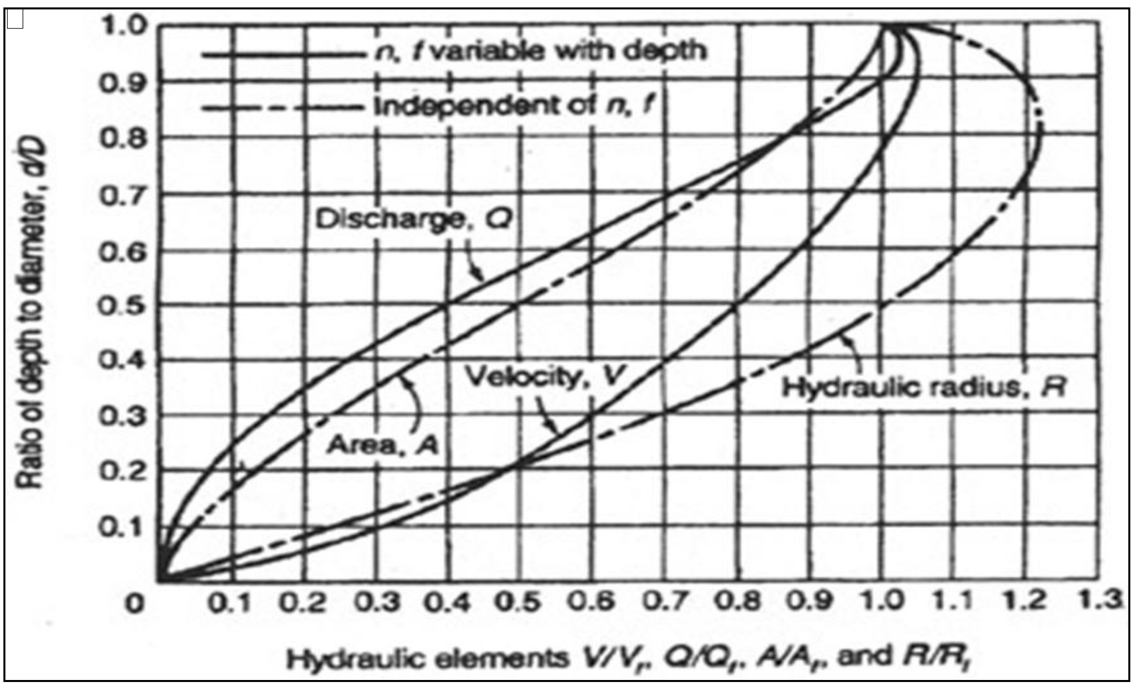

Figure 5.1: Variation of Ratios of Hydraulic Elements with the Depth Ratio.

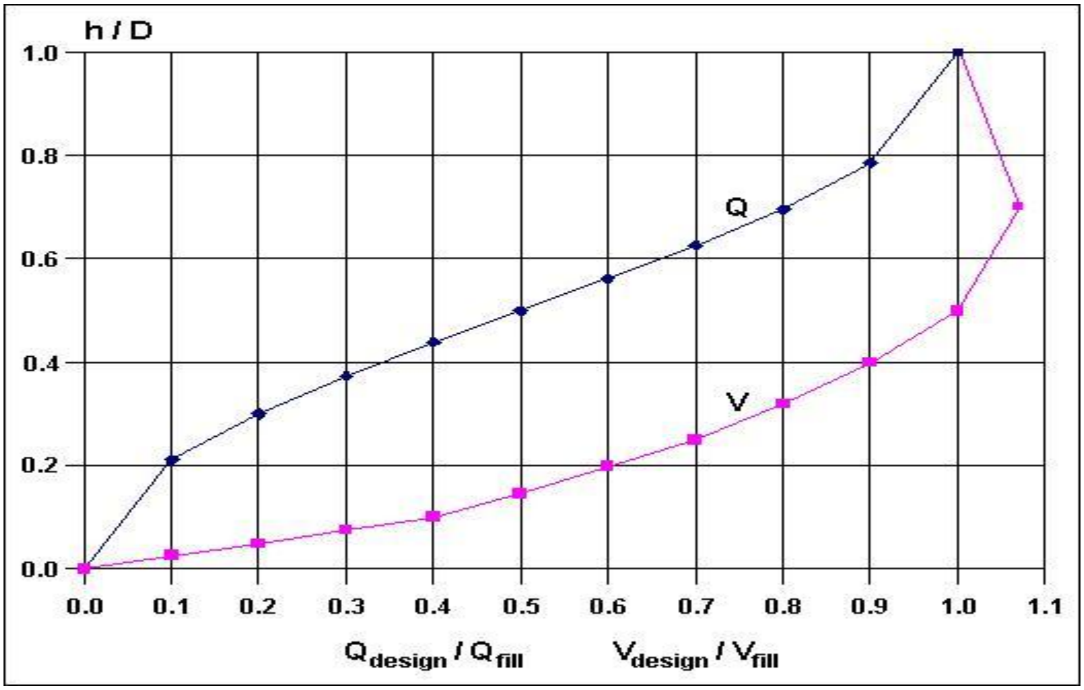

Figure 5.2: Hydraulic Elements of Circular Sewer with Equal Self-Cleansing Velocities

5.4 Design Components

In order to compute the size and slope of the sewer pipe, the important required information are:

1- Topographic map,

2- Tributary areas of each pipe system,

3- The ground surface elevation along each pipeline,

4- Elevation of the basement of low-lying houses and other building,

5- The elevation of existing sanitary sewer system (if exists).

5.5 Widely used Sewer Pipe Cross-Sections

5.5.1 Egg Shape Sewer

It is used for combined system and also called ovoid sewer. The principle advantage is that, it gives slightly higher velocity for low flow compared with circular sewers of equal capacity. An egg shape sewer disadvantage is its instability due to the small end of the egg part since it has to support the weight of the upper section. It is difficult to construct, expensive and has the absence of equate gradient.

There are two very common forms of egg shaped sewers:

a- Standard or metropolitan section which is also called the old form. It has upper portion a semi- circle of radius and equal to one third the depth or the long diameter and in invert curved on radius of one sixth of the long diameter.

b- New shape section which resembles horse-shoe.

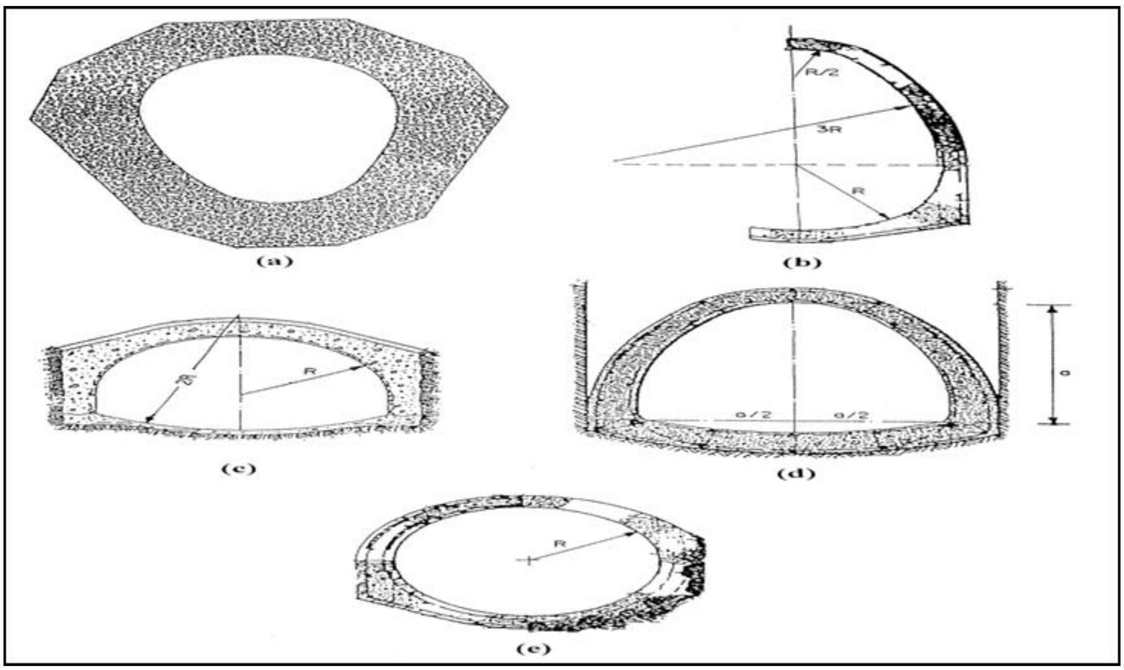

Figure 5.3: Typical Egg Shaped Sewers

5.5.2 Circular Sewers

Sewers of circular cross section are more common having the advantages:

1- Easily manufactured,

2- Has the maximum area for a given perimeter,

3- Has the most economical section,

4- Offers less opportunity for deposits.

Circular sewer may run either full or partially full:

i. Circular section running full (not recommended)

Let diameter D be the internal diameter of circular sewer

Area of wetted cross section $A = (\pi/4)*D^2$

Wetted perimeter $P=\pi*D$

Hydraulic Radius $R=A/P = D/4$

ii. Circular section running partially full (very widely used):

Let d be the depth at partial flow and let θ be the central angle.

a = area of the wetted cross section

p = wetted perimeter

r = hydraulic radius

v = velocity of flow

Central angle is given by $\cos(1/2\theta) = 1 - (2d/D)$

1- Depth of the flow $d = \frac{D}{2} - \frac{D}{2}\cos\frac{\theta}{2} = \frac{D}{2}(1 - \cos\frac{\theta}{2})$

the proportional depth = d/D = $\frac{1}{2}(1-\cos\frac{\theta}{2})$

2- Area of the flow $a = \frac{\pi}{4}D^2 * \frac{\theta}{360} - \left(\frac{D}{2}\cos\frac{\theta}{2} * \frac{D}{2}\sin\frac{\theta}{2}\right)$

the proportional area = $\frac{a}{A}\left(\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right)$

3- Wetted perimeter $p = \pi * D * \frac{\theta}{360}$

the proportional wetted perimeter $= \frac{p}{P} = \frac{\theta}{360}$

4- Wetted hydraulic radius $r = \frac{D}{4}[1 - \frac{360\sin\theta}{2\pi\theta}]$

the proportional hydraulic radius $\frac{r}{R} = [1 - \frac{360\sin\theta}{2\pi\theta}]$

5- Velocity of flow based on Manning formula due partially full

$v_p = \frac{1}{n} * r^{2\backslash 3} * S^{1/2}$

the proportional velocity $\frac{v_p}{v} = (\frac{r}{R})^{\frac{2}{3}} = [1 - \frac{360\sin\theta}{2\pi\theta}]^{\frac{2}{3}}$

6- Discharge of flow based on Manning formula due partially full $q = a * v_p$

the proportional discharge = $\frac{q}{Q} = \frac{a * v_p}{Av} = \frac{a}{A} * (\frac{r}{R})^{\frac{2}{3}}$

Note that for variable values of n_p/n the equation becomes $\frac{q}{Q} = \frac{n_p}{n}(\frac{a}{A})(\frac{r}{R})^{\frac{2}{3}}$

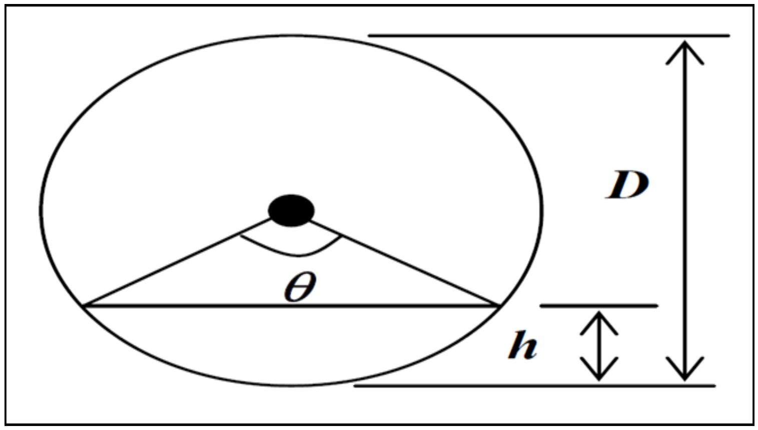

Figure 5.4: A Circular Pipe Running Partially Full

5.6 Slope Selection

In wastewater network design, one of the important factors limiting the design is the slope. According to table below one can decided the minimum slopes for different pipe sizes for sewer systems. Details are given in Table 5.4.

Table 5.4: Minimum Slope (S_{min}) for Different Pipe Sizes (D)

D (mm)	150	200	250	300	375	450	525	600
S_{min}	0.0043	0.0033	0.0025	0.0019	0.0014	0.0011	0.00092	0.00077

Chapter 6

GAZİMAĞUSA (FAMAGUSTA) SEWAGE NETWORK SYSTEM

6.1 Introduction

Gazimağusa (Famagusta) sewage network is the first project in this region. This project will serve for 56000 people during its service life of 25 years. For that, the city is really divided into seven sections. For some parts, for the collection network system pump station were used so as to collect and convey the sewage water to the wastewater treatment plant which was constructed just southern hills of Gazimağusa.

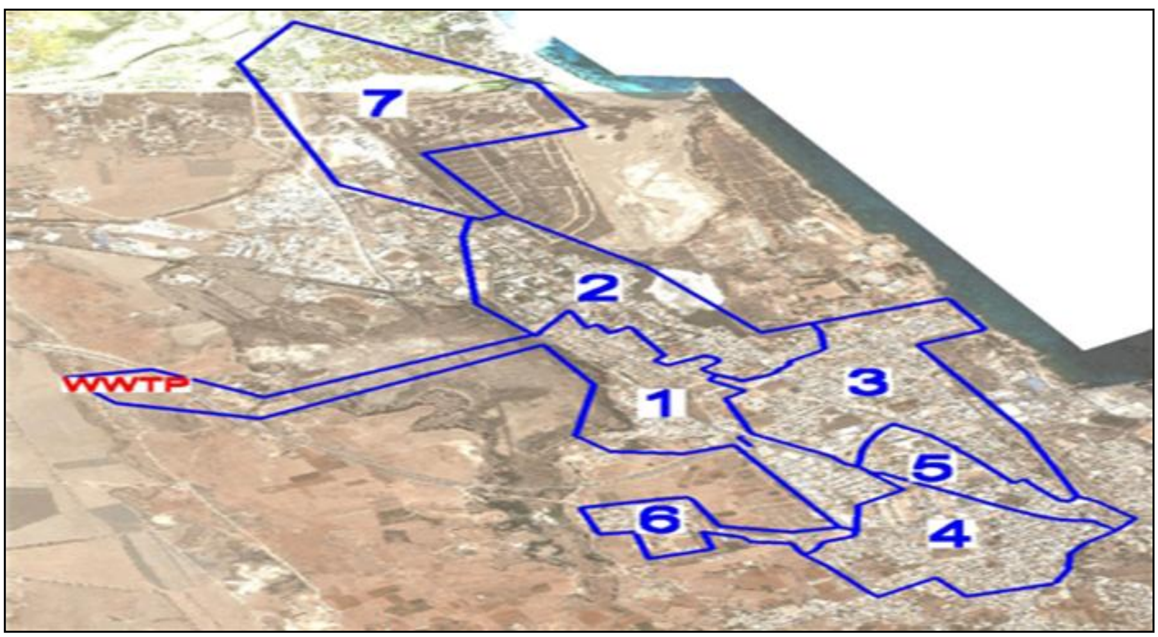

Figure 6.1: General Layout of Famagusta Networks

6.2 Description of Famagusta Network Sewage System

The project network had a total pipeline length 71.2 km where corrugated high density polyethylene (HDPE) pipe with a largest inner diameter 600 mm was used. The project has 7 pump stations of capacities ranging 3.5 - 260 l/s having a pressurized pipe portion of nearly 4.6 km. The total constructed manholes are 1404. There are 1465 connections with unplasticized polyvinyl chloride (UPVC) pipes of total length 9.4 km.

6.2.1 Hydraulic details of Section 1

a) total pipeline length is 15.4 km,

b) maximum and minimum elevations are 17.00 m - 4.53 m respectively,

c) main trunks total length is 1.77 km (DN600-DN500)

d) sewer networks total length is 10.2 km (DN200)

e) pressurized pipe length from Pump Station 1 is 2.31 km (DN630)

f) pressurized pipe length from Pump Station 2 is 0.76 km (DN400)

g) pressurized pipe length from Pump Station 5 is 0.31 km (DN110)

h) number of manholes are 216 within the network except house connections,

i) number of house connection manholes are 305.

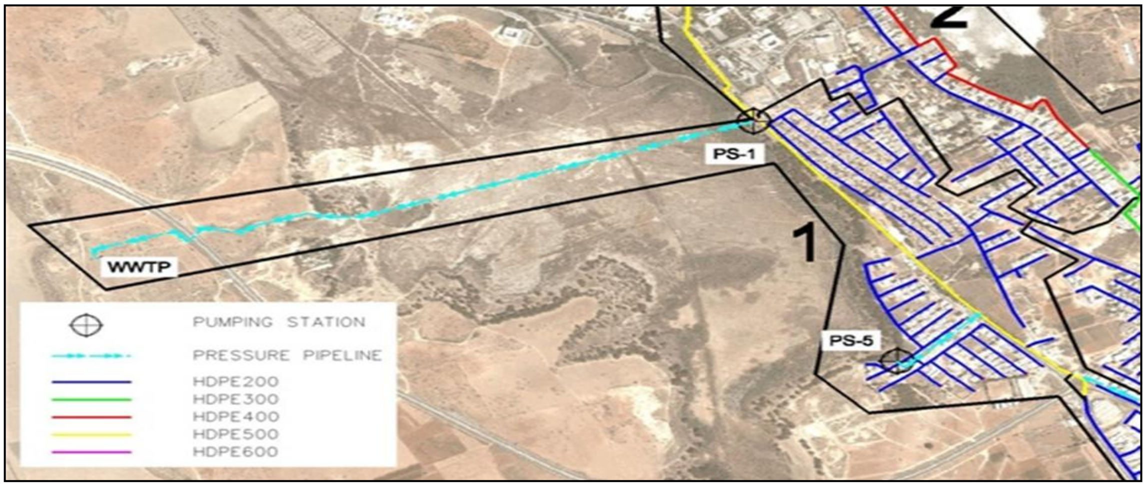

Figure 6.2: Sewer Network of Section 1

6.2.2 Hydraulic details of Section 2

a) total pipeline length is 5.3 km,

b) maximum and minimum elevations are 17.47 m - 2.15 m respectively,

c) main trunks total length is 20 m (DN500),

d) secondary trunks total length is 1.8 km (DN400-DN300),

e) sewer networks total length is 3.3 km (DN200),

f) pressurized pipe length from Pump Station 3 is 180 m (DN400),

g) pressurized pipe length from Pump Station 7 is 387 m (DN250),

h) number of manholes 97 within the network except house connections,

i) number of house connection manholes are 170

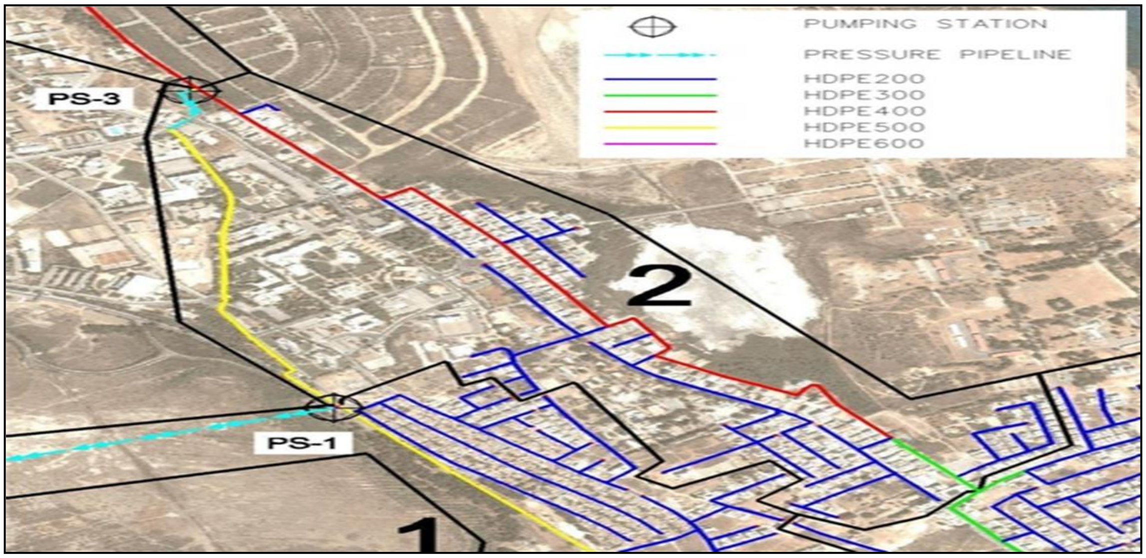

Figure 6.3: Sewer Network of Section 2

6.2.3 Hydraulic details of Section 3

a) total pipeline length is 12.7 km,

b) maximum and minimum elevations are 14.06 m - 4.60 m respectively,

c) secondary trunks total length is 0.6 km (DN300),

d) sewer networks total length is 12 km (DN200),

e) pressurized pipe length from Pump Station 4 is 195 m (DN90)

f) number of manholes 229 within the network except house connections,

g) number of house connection manholes are 465.

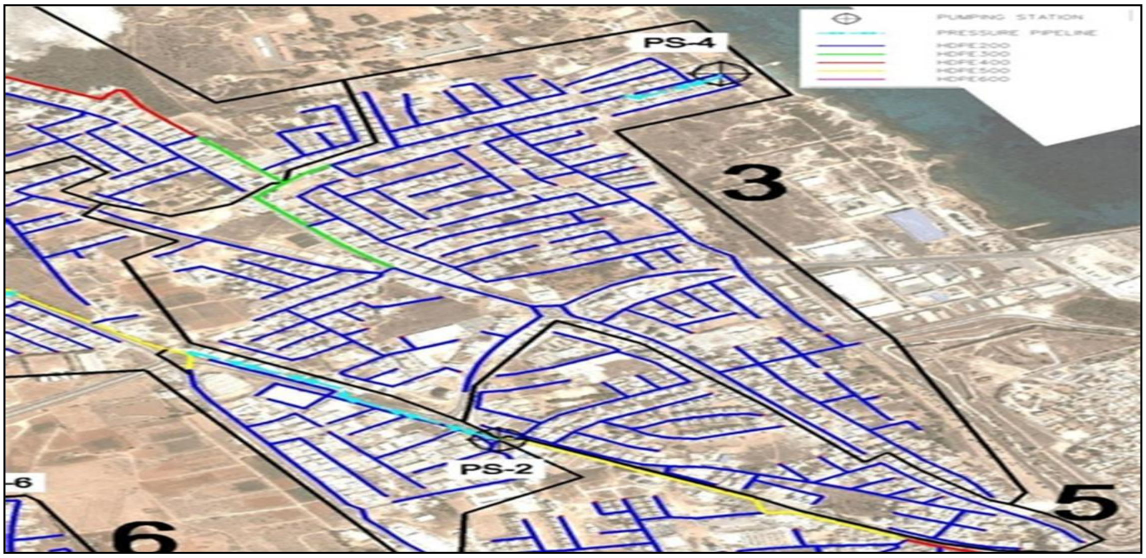

Figure 6.4: Sewer Network of Section 3

6.2.4 Hydraulic details of Section 4

a) total pipeline length is 9.8 km,

b) maximum and minimum elevations are 15.54 m - 9.32 m respectively,

c) main trunks total length is 0.85 km (DN500),

d) secondary trunks total length is 1.6 km (DN400 - DN300),

e) sewer networks total length is 7.4 km (DN200),

f) number of manholes 192 within the network except house connections,

g) number of house connection manholes are 465.

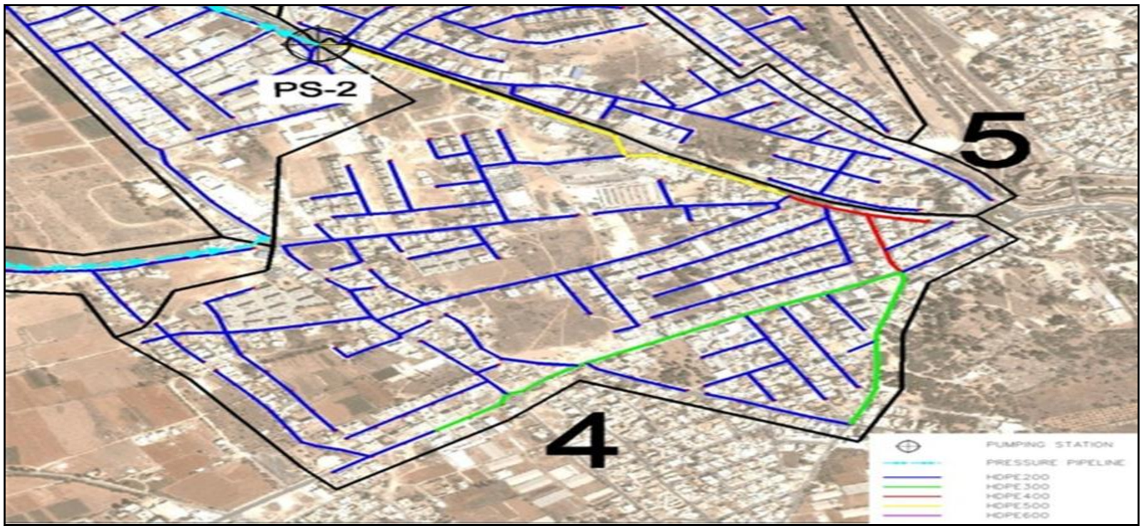

Figure 6.5: Sewer Network of Section 4

6.2.5 Hydraulic details of Section 5

a) total pipeline length is 3.8 km,

b) maximum and minimum elevations are 13.07 m - 7.82 m respectively,

c) sewer networks total length is 7.4 km (DN200),

d) number of manholes 74 within the network except house connections,

e) number of house connection manholes are 145.

Figure 6.6: Sewer Network of Section 5

6.2.6 Section 6

a) total pipeline length is 4.3 km,

b) maximum and minimum elevations are 14.25 m – 8.9 m respectively,

c) main trunks total length is 3.20 km (DN200),

d) pressurized pipe length from Pump Station 6 is 1.10 km (DN140)

e) number of manholes 59 within the network except house connections,

f) number of house connection manholes are 112.

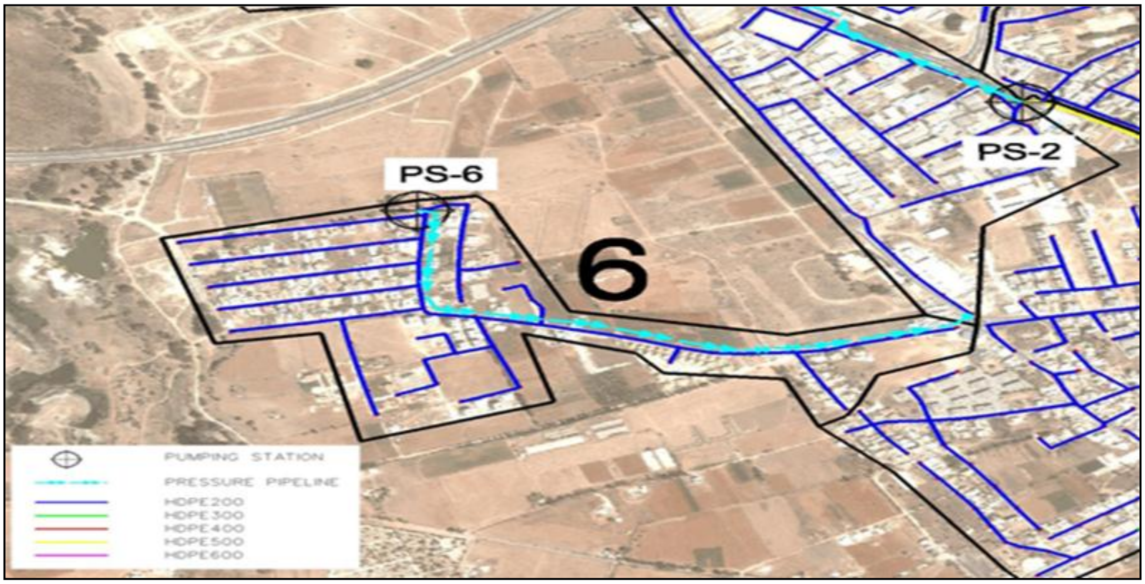

Figure 6.7: Sewer Network of Section 6

6.2.7 Section 7

a) total pipeline length is 6.2 km,

b) maximum and minimum elevations are 8.06 m – 2.54 m respectively,

c) secondary trunks total length is 2.35 km (DN400-DN300)

d) sewer networks total length is 3.8 km (DN200)

e) number of manholes 115 within the network except house connections,

f) number of house connection manholes are 125.

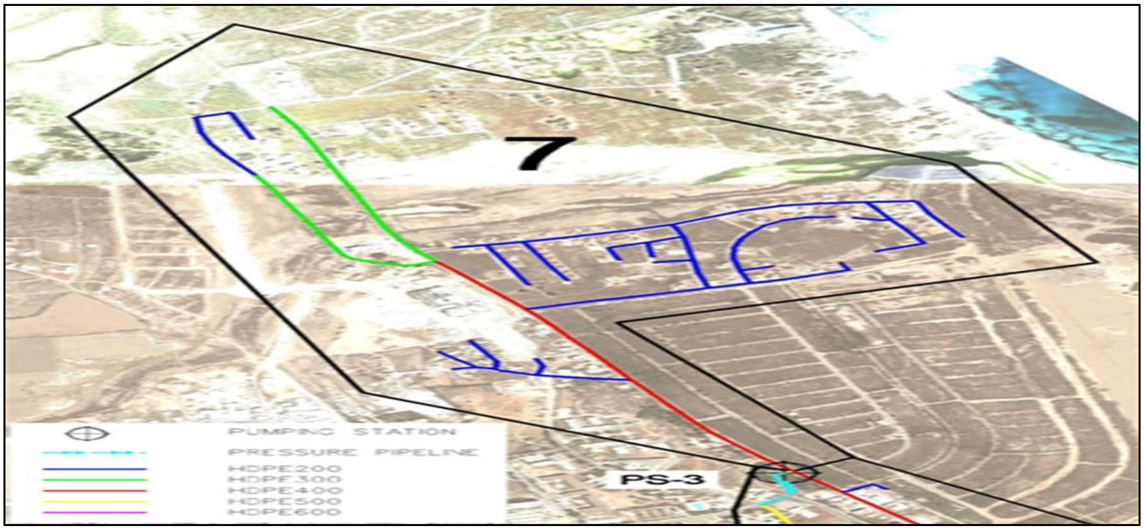

Figure6.8: Sewer Network of Section 7

6.3 Details of Famagusta Pump Station Sewage System

The unique reason of building pump station is because of the elevation differences due topography within each area and the far distance of treatment plant. In this project there are two types of pump station designs.

a) Pump Station with two pumps having circular cross-section wet well with a chamber made of precast polymer concrete of two types:

    i- located along a side way where no vertical load exerts,

    ii- located within the road that can carry up to a total load 40 force.

b) Pump Station with three pumps having rectangular cross-section wet well with a chamber made of in situ concrete.

6.3.1 Pump Station One (PS1)

The location of this pump station is along the Lefkosa-Gazimagusa that passes along EMU main road just in front of the sideway junction of EMU. Its design discharge is 130 l/s (468 m^3/hr).

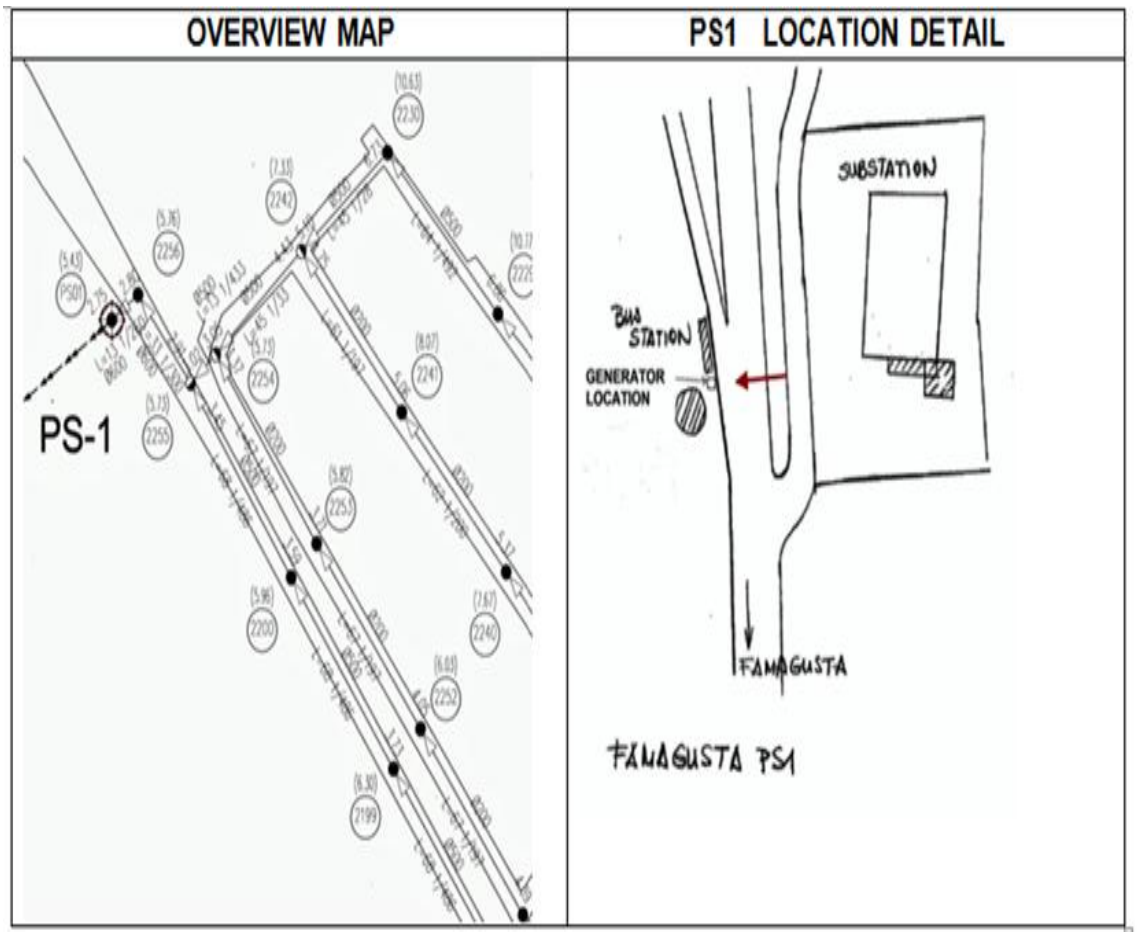

Figure 6.9: Location of Pump Station One

6.3.2 Pump Station Two (PS2)

The location of this pump station is beside The Dee European Hotel along the side way. Its design discharge is 125 l/s (450 m^3 /hr).

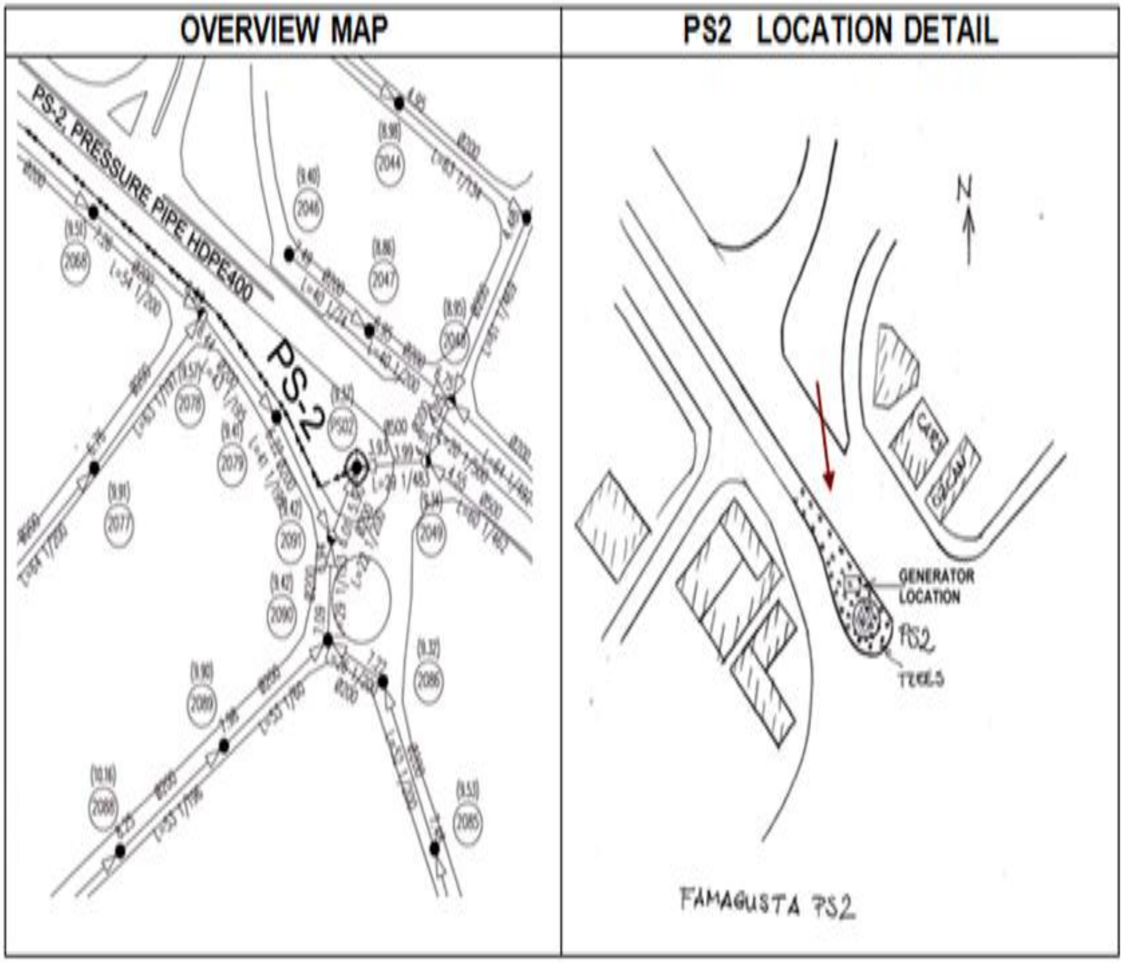

Figure 6.10: Location of Pump Station Two

6.3.3 Pump Station Three (PS3)

The location of this pump station is along the Karpas-Gazimagusa main road at the side way of eastern exit of EMU campus. Its design discharge is 60 l/s (216 m^3/hr).

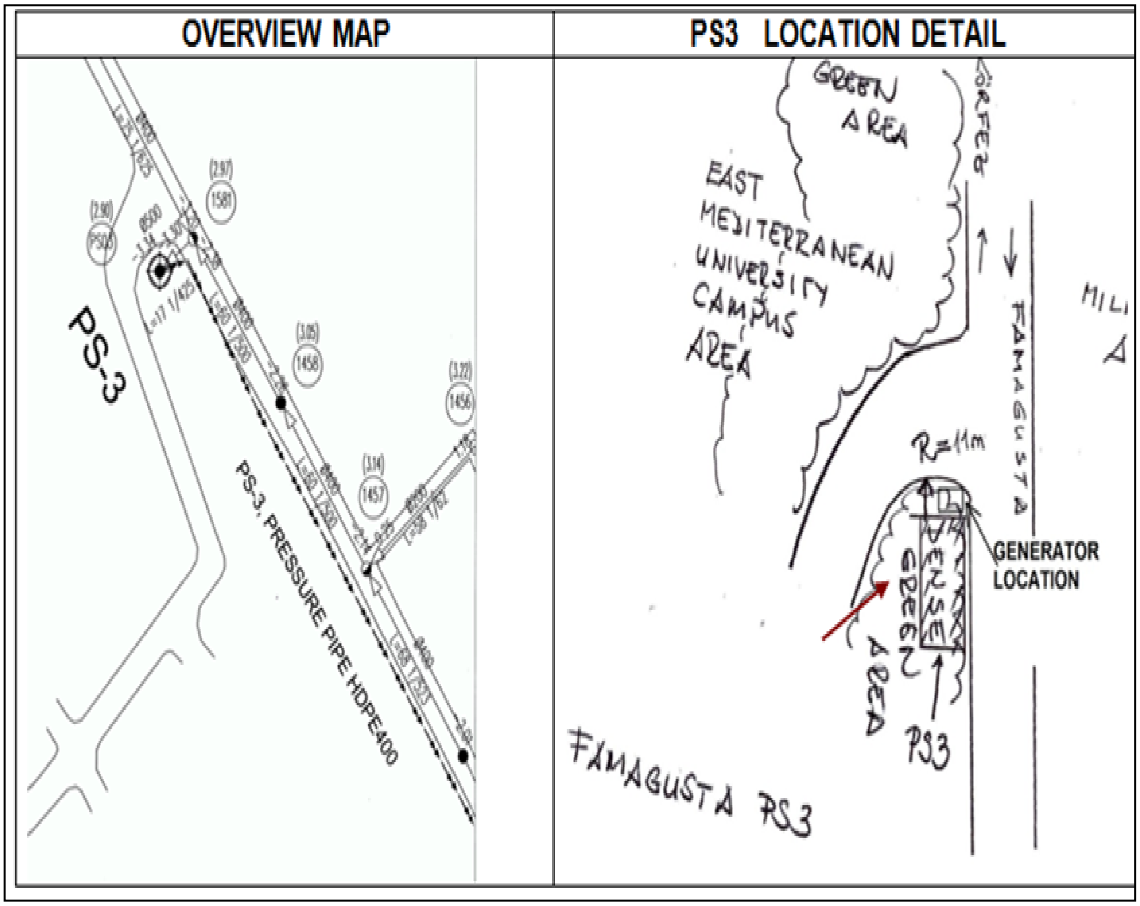

Figure6.11: Location of Pump Station Three

6.3.4 Pump Station Four (PS4)

The location of this pump station is at the side way of Karakol beach. Its design discharge is 3.5 l/s (13 m^3/hr).

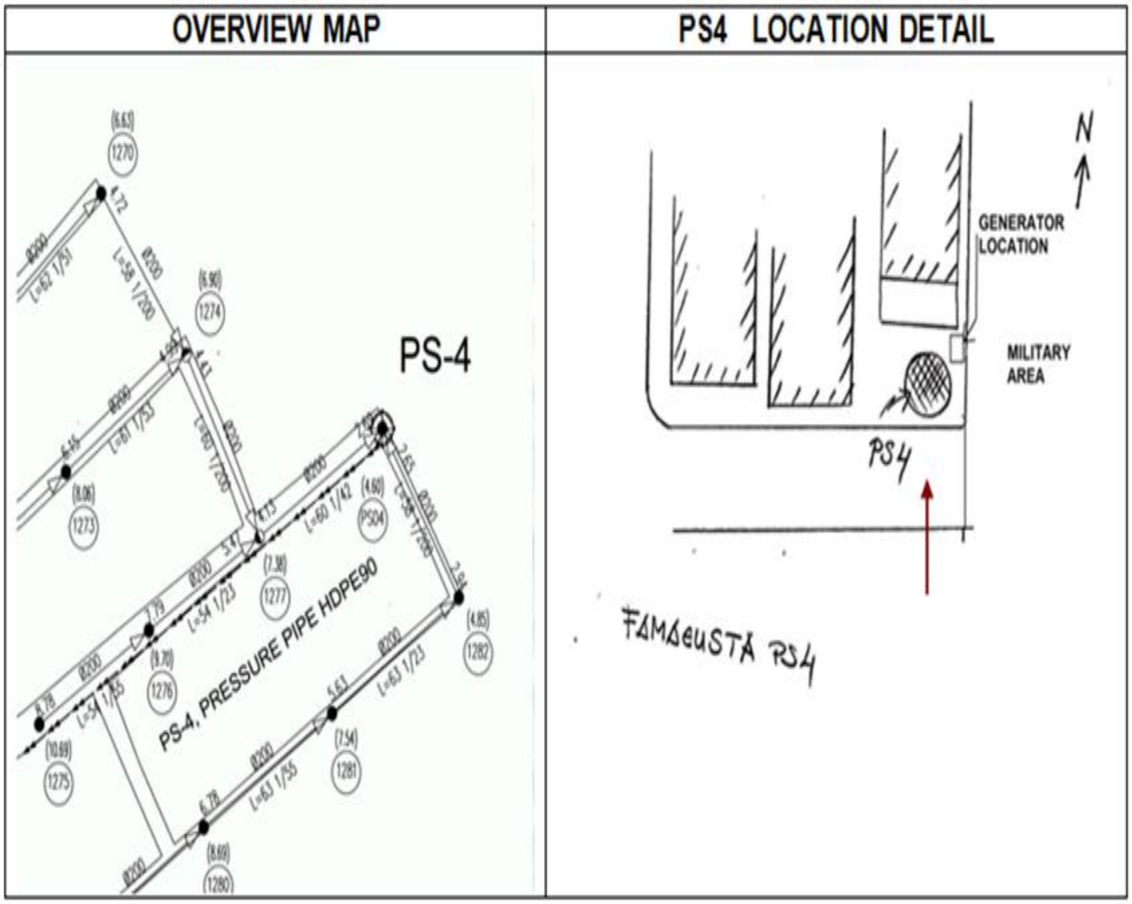

Figure6.12: Location of Pump Station Four

6.3.5 Pump Station Five (PS5)

The location of this pump station is along the side way at Kaliland region. Its design discharge is 5.4 l/s (19 m^3 /hr).

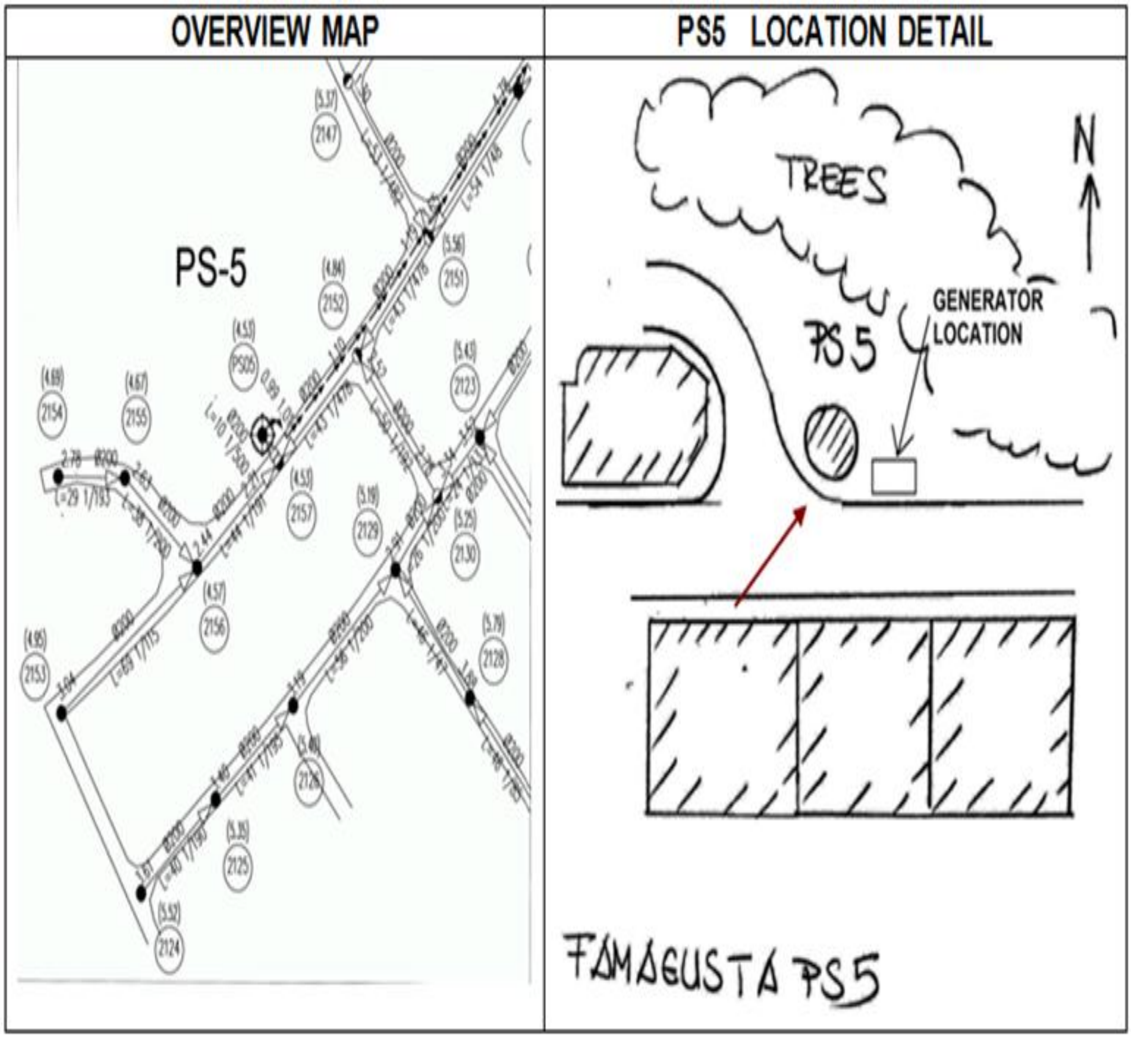

Figure 6.13: Location of Pump Station Five

6.3.6 Pump Station Six (PS6)

The location of this pump station is along the side way at the Kandulular area back of Küçük Sanayi. Its design discharge is 8.8 l/s (32 m^3/hr).

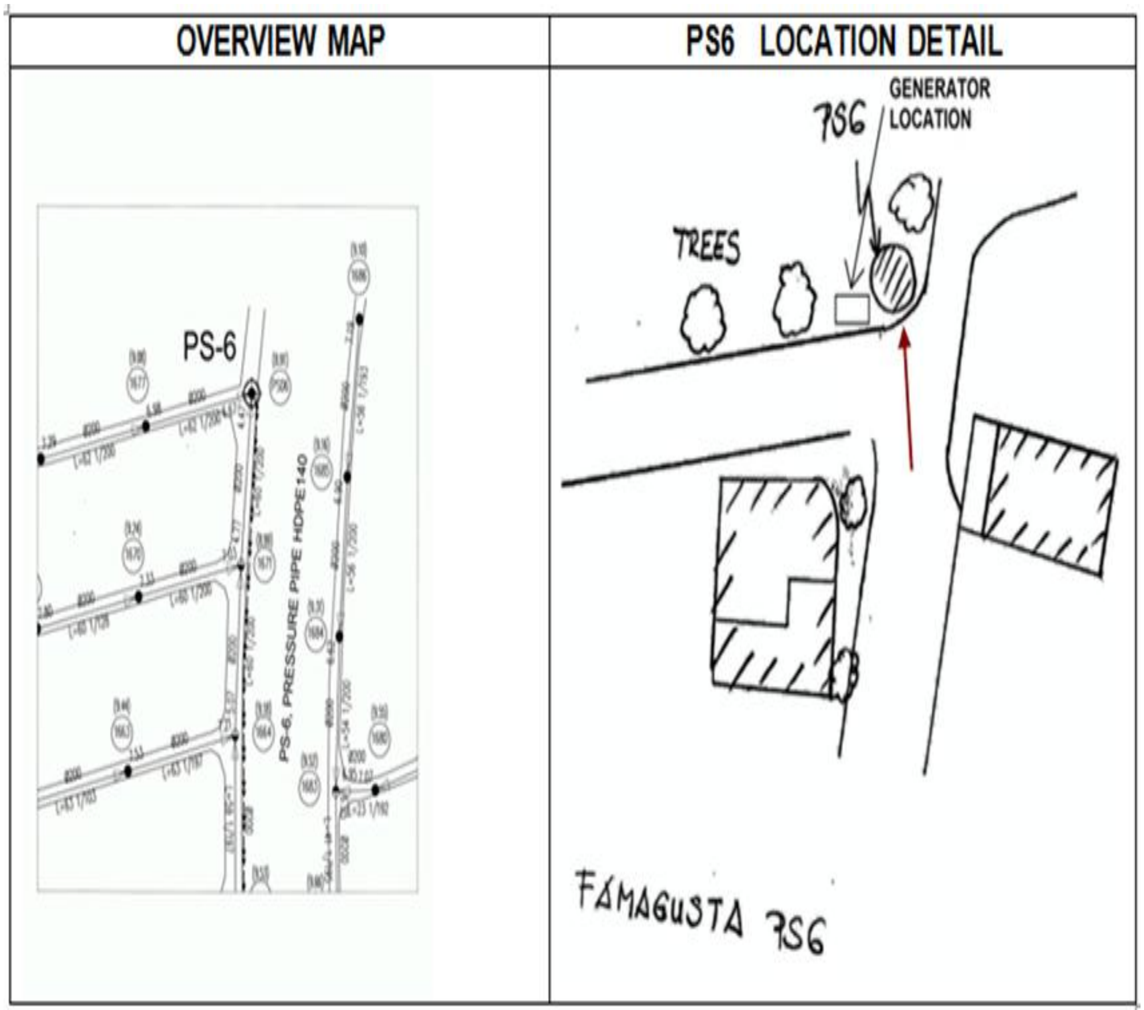

Figure 6.14: Location of Pump Station Six

6.3.7 Pump Station Seven (PS7)

The location of this pump station is within the road junction of Karpaz-Gazimağusa main road and Karakol route just near the new Lemar mall. Its design discharge is 37 l/s (133 m^3/hr).

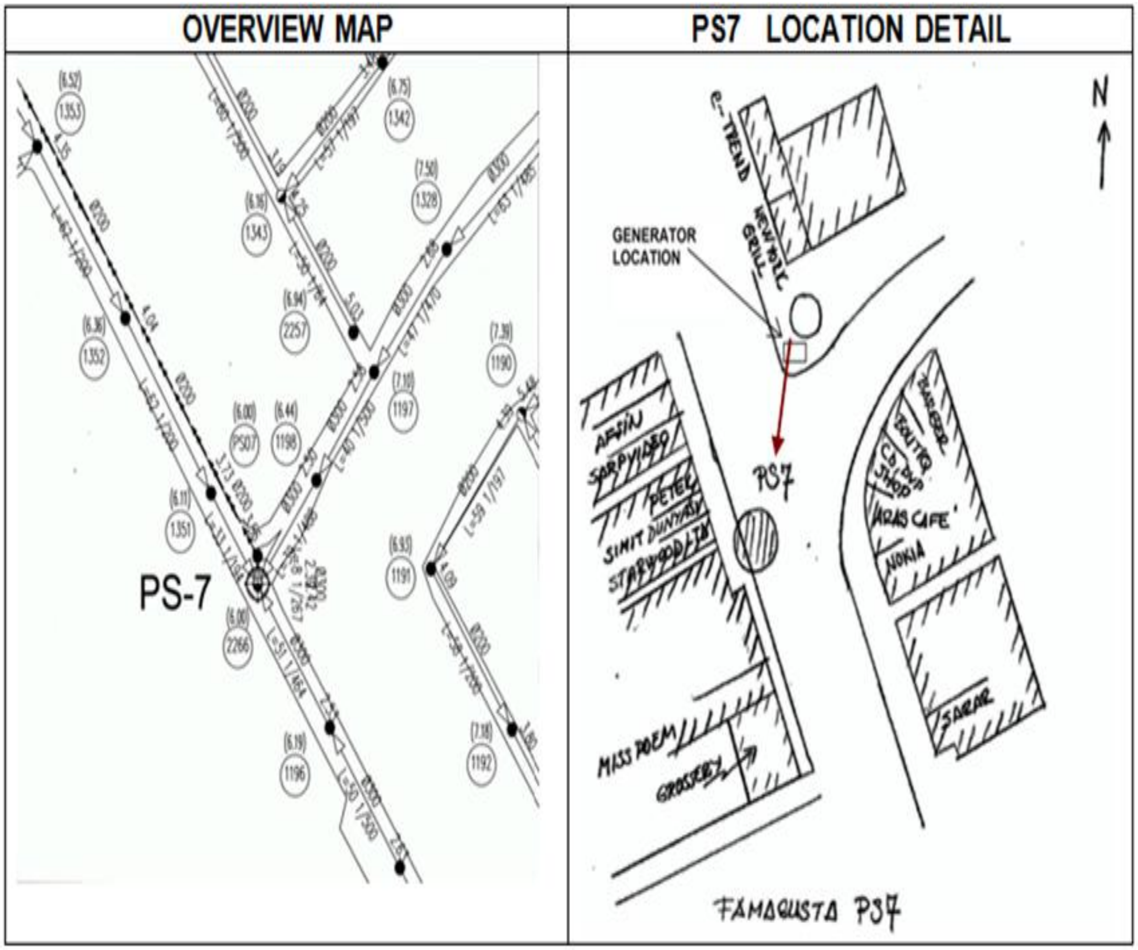

Figure 6.15: Location of Pump Station Seven

6.4 Construction Materials of Famagusta Sewer System

6.4.1 Pipe Types

Different pipes types were used in this network. The selection of different types is due to discharge volume and transmitting ways. Hence, in this project used pipe types are:

- UPVC pipe having gasket-type joint of homogenous material.
- HDPE pipe having diameter less than 100 mm with corrugated shape.
- Reinforced concrete pipes and fittings with flexible joints where all pipes and fittings have gasket-type joints of spigot and socket.

6.4.2 Manhole Types

6.4.2.1 HDPE Manholes

- Circular shape which is pre-fabricated.
- The thickness of HDPE manholes are 30 mm.
- Covers and frames are designed for 40 tons (D 400).
- These manholes have pre-cast HDPE steps.
- Inlet and outlet pipes connected to the manhole the using electro-fusion welding.

6.4.2.2 Pre-cast Manholes

Pre-cast concrete chamber and shaft sections are constructed without steps. The pre-cast concrete manholes consists of several elements like base slab element, one or more current ring – type elements and a joint cone pipe ring. Their inner diameters are 1 m.

Chapter 7

HYDRAULIC CALCULATION DETAILS OF SEWER FOR GAZİMAĞUSA (FAMAGUSTA)

7.1 Populations

Check population forecast of Famagusta city for year 2025 by several methods:

7.1.1 Arithmetic Method

$$K_a = \frac{P_f - P_i}{t_f - t_r} \qquad K_a = \frac{Pf - 53247}{2025 - 2020}$$

gives K_a= 498 person/year, the population in 2025 will be 55737 people.

7.1.2 Geometric Method

$$K_b = \frac{LnP_f - LnP_i}{t_f - t_i} \qquad K_b = \frac{Ln5324 - Ln48140}{2020 - 2010}$$

gives K_b=0.01 ln person\year, the population in 2025 will be 55972 people.

7.1.3 Method of Turkish Bank

$$K = \left[(T_2 - 2010)\sqrt{\frac{P_2}{P_{2010}}} - 1 \right] * 100 \qquad K = \left[(2020 - 2010)\sqrt{\frac{53247}{48140}} - 1 \right] * 100$$

gives K=0.3 because of K<1 then take K=1

$$P_n = P_2\left(1+\frac{K}{100}\right)^{(T_n - T_2)} \qquad P_n = 53247\left(1+\frac{1}{100}\right)^{(2025-2020)}$$

the population in 2025 will be 55963 people.

The results of these three popular equations were compared by the design criteria of Famagusta sewer system as detailed in Appendix 1 which is 56000 and is concluded that it is acceptable.

7.2 Determination of Average, Maximum and Minimum Waste Water Discharges

Potable water consumption rate for Famagusta residential area is (120 lt/cap/day) and the existing population 48140. The amount of domestic sewage is 80 % of the potable water [Gazimagusa Baldiyesi,2006].

7.2.1 Average Discharge

96*48140 = 4621440 lt/cap/day = 53.5 lt/sec/capita

7.2.2 Maximum Discharge

$$M_1 = \frac{4.8}{P^{1/6}} \qquad M_1 = \frac{4.8}{48^{1/6}}$$

So M_1=1.9064 therefore the maximum discharge is:

53.5 lt/sec *1.906 = 102 lt/sec/capita

7.2.3 Minimum Discharge

$$M_2 = 0.2 * 48^{1/6}$$

So M_2=0.381 therefore the minimum discharge is 20.4 lt/sec/capita.

7.3 Fluctuations in Water Use

a) Winter is usually about 20 % lower than the annual daily average:

120 - (120*0.2) = 96 lt/day/capita

b) Summer is usually about 20-30 % higher than the annual daily average:

120 + (120*0.3) = 156 lt/day/capita

c) Early mornings the hourly variations are high up to 25 – 40 % of the average hourly use of a day.

120 + (0.4*120) = 168 lt/day/capita

d) Near noon the demand usually hits the peak about 150-200 % of the average hourly use of a day.

120 * 2 =240 lt/day/capita

e) From most communities, the maximum daily use is about 180 % of the average noon of a day

120 * 1.8 * 2 = 432 lt/day/capita

7.4 Hourly/daily peak factors

For Famagusta city the peak factor (p.f.) hourly is 2.5 and daily is 4.

1- Hourly peak factor

P.F hourly = D.m.h. (max. hourly demand during a day) / D.a.h. (average hourly demand during a day).

So 2.5 = D.m.h./120 D.m.h.= 300 lt/day/capital

2- Daily peak factor

P.F daily = D.m.h. (max. hourly demand during a year) / D.a.d. (average hourly demand in a year) [Gazimagusa,2006].

So 4 = D.m.d./120 D.m.d. = 480 lt/day/capita

7.5 Design of Pipe Trunk that Transmits to Treatment Plant

7.5.1 Pressure Pipe Diameter

From appendix 2 getting data for calculation

Q_{max} = 0.663 m^3/sec. Hp =17.5 m Z_2 = 2.75 m and L =110 m

Assume f=0.01

- From Bernoulli equation

$$Hp = \frac{V_2{}^2}{2g} + Z_2 + f\,\frac{L}{D}\,\frac{V_2{}^2}{2g}$$

$$17.5m = \frac{0.663^{\;2}\,m3/sec}{12.1D^4} + 2.75m + 0.01\frac{110m}{D^5}\,\frac{0.663^2\,m3/sec}{12.1}$$

Where,

Q_{max}: discharge (m^3/sec)

Hp : total head (m)

Z_2 : inlet elevation (m)

L : distance (m)

V_2: velocity (m/sec)

D : diameter (m)

By using trial and error method the proper diameter (D) ranges 0.3 m – 0.33 m.

Comparing with D in use for Famagusta checked in Appendix 4 the selected design diameter is correct.

7.5.2 Gravity Pipe Diameter

Total area = 11430.2 hectares so converted to 114302000 m^2

Infiltration = 20 % enters to the system

Since effect of ppt. max. is in Decembers = 82.3 mm/month

by assuming half of ppt. infiltration to the sewer system 82.3/2 = 41.15mm/month.

$$\frac{41.15 mm / month}{31 day * 1000 * 86400 sec} = 1.536 * 10^{-8} m / sec$$

Q_{ppt} = 1.536*10^{-8} m/sec * 0.2 * 114302000 m^2 = = 0.352 m^3/sec

Maximum discharge = 480 lt/day/capital

$$\frac{480 lt / cap / day}{86400 sec * 1000 m3} * 56000 = 0.311 m^3 / sec$$

Total discharge Q = 0.311m^3/sec + 0.352 m^3/sec = 0.663 m^3/sec

Slope average is 4.22 * 10^{-2}

$$D^{\frac{8}{3}} = 3.208 * \frac{0.011 * 0.663}{\sqrt{4.22 * 10^{-2}}}$$

So the average diameter D = 0.913 m

In Famagusta sewer system, the designer used pipes ranging from 420 mm -1290 mm as details in Appendix 4.

Q = AV

$$V = \frac{0.663 m^3 / sec}{\frac{\pi}{4}(0.913 m)^2} \qquad V=1.02 \text{ m/sec}$$

7.6 Checking the Design Calculations of Section Lot 5

Total length for all the sewer networks for Gazimağusa (Famagusta) are 57.7 km, population for 2025 is estimated to be 56000.

Population per km is 56000/57.5 = 974 person/km

For Lot 5:

Length of the pipe is 3.8 km so the population of lot 5 estimate is 3.8*974=3700 person.

Leaving area per person is 40 m^2 from EU standards so 3700 * 40 = 148000 person/m^2

Hence 148000 person /m^2 / 3800 m = 39 m^2/m per person.

All the calculation are detailed in Table 7.1 and Appendix 3.

Table 7.1: Hydraulic Design of Famagusta Sewer System of Lot 5.

7.6.1 Checking Velocity

Depending on calculation in Table 7.1 D is found 0.2. Pipe is plastic, So N/n=1.

$$Qf = \frac{D^{\frac{8}{3}} * S^{\frac{1}{2}}}{3.208 * n} \quad = \quad Qf = \frac{0.2^{\frac{8}{3}} * 0.0051^{\frac{1}{2}}}{3.208 * 0.011} = 0.0274 \text{ m}^3/\text{sec}$$

$$Vf = \frac{Qf}{A} = \frac{0.0247}{0.1^2 * \pi} = 0.873 \text{ m/sec}$$

7.6.1.2 Checking for Maximum velocity

$$\frac{q}{Q} = \frac{9.031 \text{lt}/\text{sec} * \frac{\text{m3}}{1000}}{0.0274 \text{m3}/\text{sec}} = 0.328$$

From Figure 5.1 d/D=0.4

v/Vf=0.9 are obtain v =0.873 m/sec*0.9 m/sec= 0.785

0.6<0.785<4 then O.k

7.6.1.3 Checking for cleansing

$$\frac{q}{Q}=\frac{5.1061\,lt/sec*\frac{m3}{1000}}{0.0274\,m3/sec}=0.186$$

From Figure 5.1 d/D=0.26

v/V_f=0.92

v=0.92*0.873=0.803 m/sec

since v_s=0.803 m/sec from Figure 5.2, one gets Vmin =0.6

Then v_s=0.48 m/sec

We have v=0.803 m/sec > v_s=0.48 m/sec this mean O.k.

Chapter 8

CONCLUSION AND RECOMMENDATION

8.1 Results

Table 8.1: Comparison of the Results

Item	**Used Data in the Project**	**Determined Data in this Study**	**Comment**
Population	56000	55972	Acceptable
V max	0.6-4	0.785	Acceptable
V s	0.48	0.803	Acceptable
Slope	0.0019	0.0033	Not Acceptable
D (pipe diameter)	200 mm	200 mm	Acceptable

8.2 Conclusions

From all above analyses and results for Gazimağusa sewer network project, it is concluded that:

i- The designed diameter used was correct based on the checks for Lot 5.

ii- The designer in (Lot 5) used variable slopes in this project that may cause serious cloggings.

iii- Comparing the slope that was used with the specification, they need to use minimum 0.0033 slopes for diameter 200 mm which they didn't so will create serious clogging problems within the system during wet periods.

iv- Depending on the slope values, the velocity will change suddenly within the pipe system that will lead sedimentation and clogging.

8.3 Recommendations

From this study, it is recommended that:

1- They need to solve the problem of designed slopes by adopting the specification of minimum slope-diameter criteria.

2- For the pressurized pipe system a unique diameter size was used. This definitely creates an enormous extra pressure to the pumps. Instead, if several pipe diameter sizes in descending order is being used, the high pressure acting on the pump will be released.

3- For the maintenance, only the skilled workers should be used so as to increase the life span of the sewer network.

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APPENDICES

Appendix 1a: Available Population Projections for Famagusta.

Scenario	Parameter	Year			
		2010	2015	2020	2025
Base Scenario	Inhabitants	48.140	50.629	53.247	56.000
	Growth rate	1,01%	1,01%	1,01%	1,01%

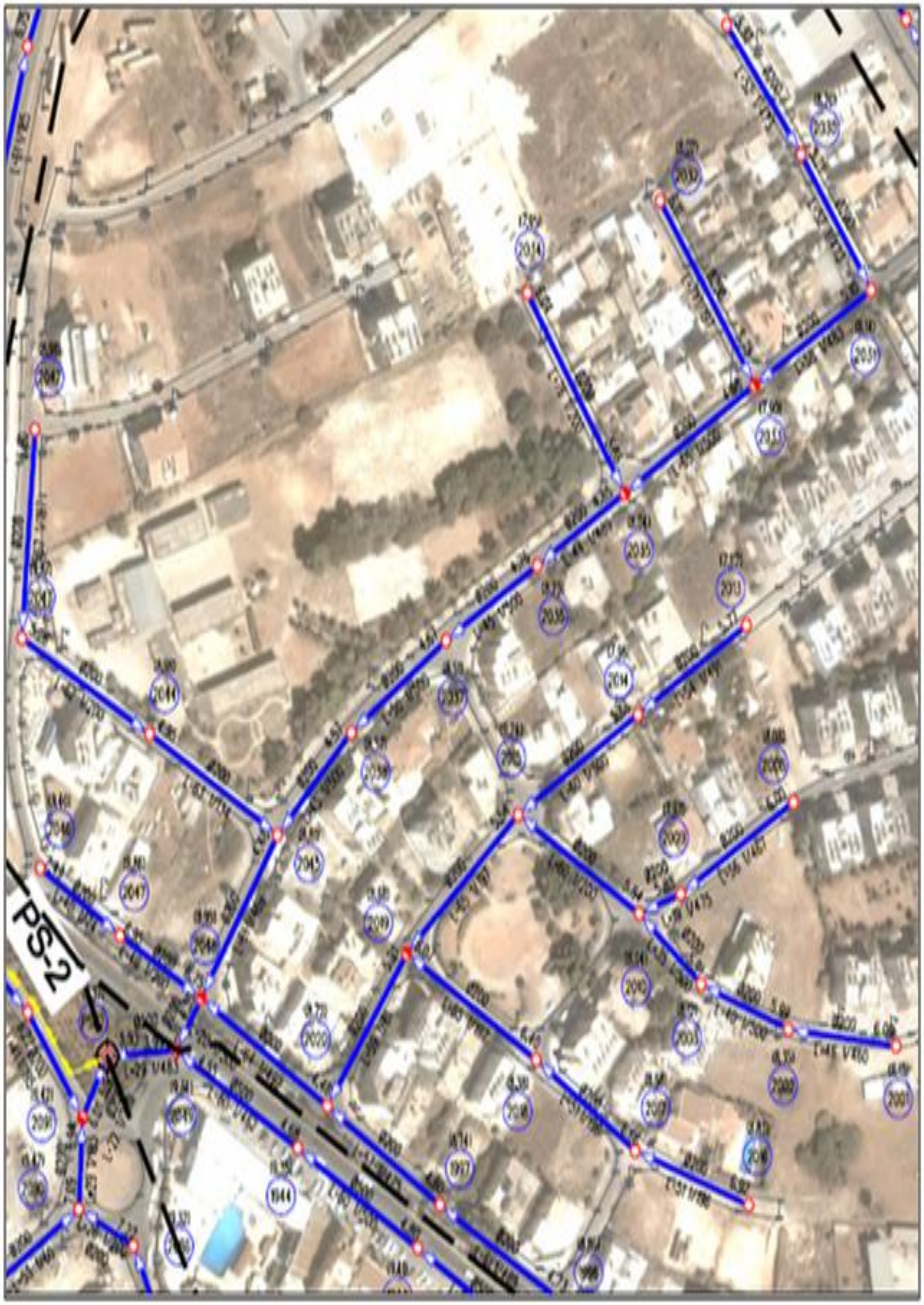
Appendix 1b: Average Rainfall (year 2008) of Famagusta Region (mm)

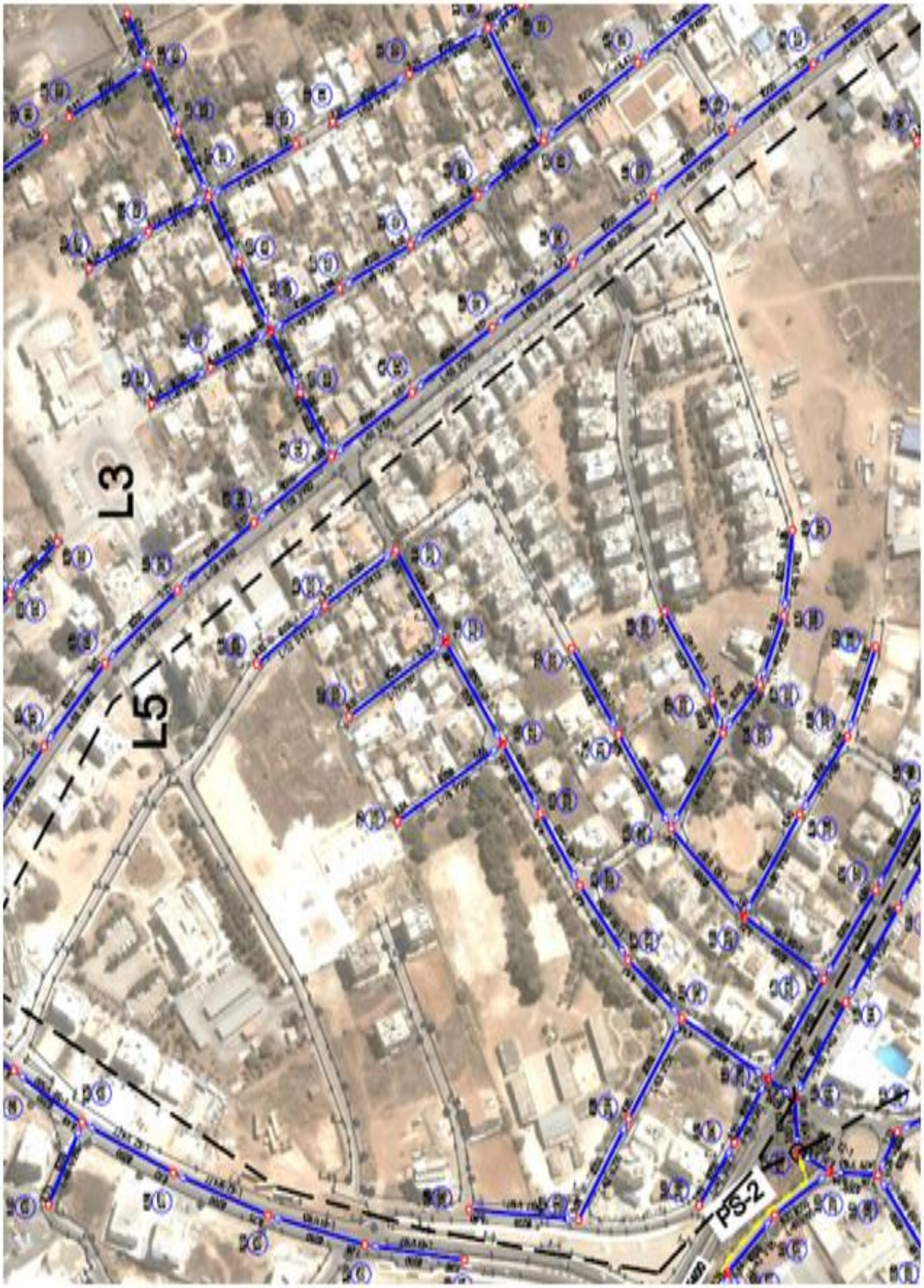
Annual	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
335.0	63.9	50.0	37.6	20.4	9.6	3.3	0.6	0.4	4.3	16.2	46.3	82.3

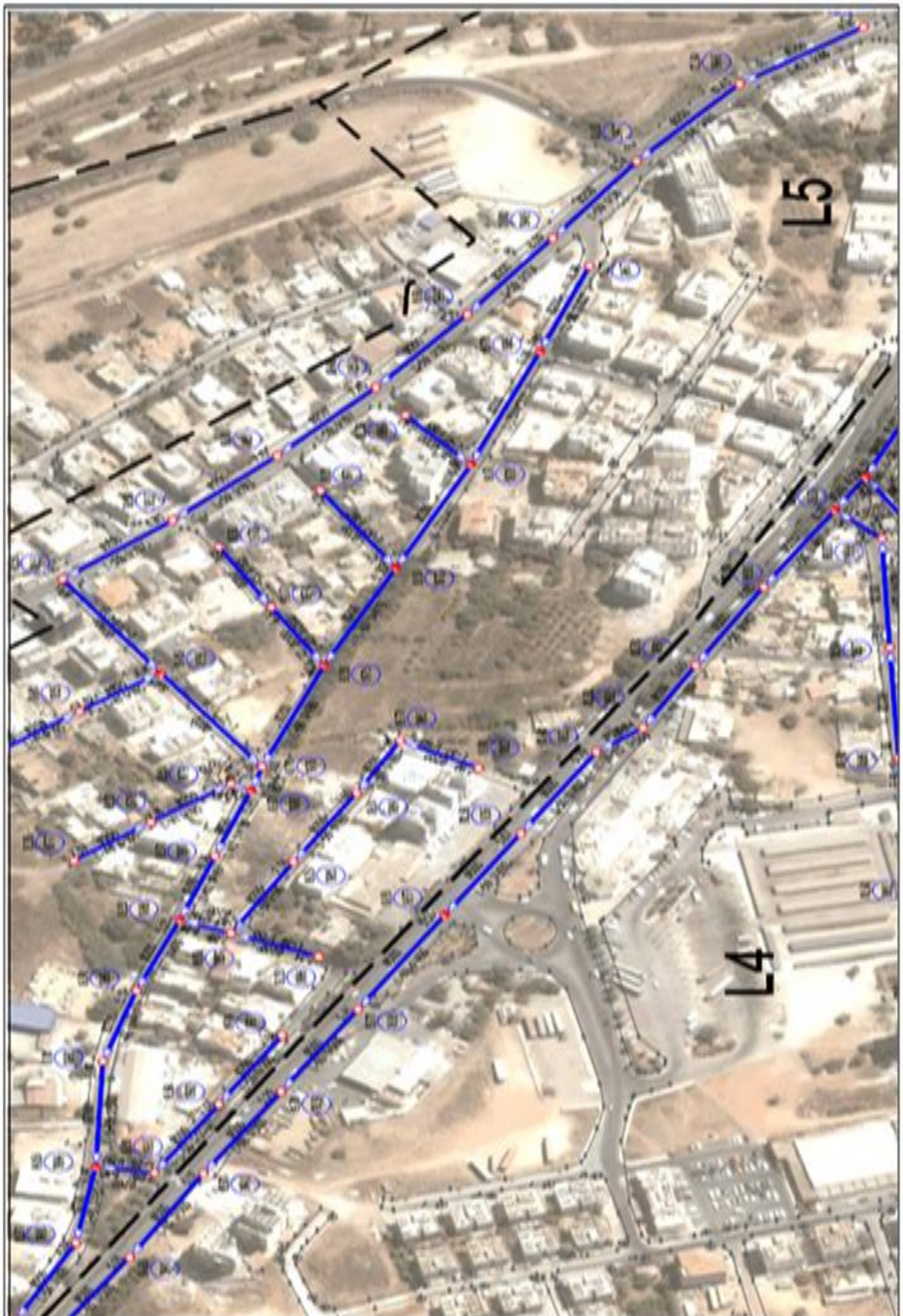
Appendix 2: Details for the Various Pump Stations are given in the Data

PS Famagusta	PS No.	1 (Main)	2	3	4	5	6	7
Pumped Flow Q	l/s	130,0	125,0	60,0	3,5	5,4	8,8	37,0
	m³/h	468,0	450,0	216,0	13,0	19,0	32,0	133,0
Total Head m	m	17,50	12,60	22,20	11,10	13,00	17,10	8,40
Diameter Wet Well	mm	2m / 3m	2m / 3m	2m / 3m	1.200	1.500	2.000	2.200
A Inlet Elevation	m	2,75	3,93	-3,3	2,65	0,99	4,47	2,39
B Well Ground Elevation	m	5,43	9,37	2,90	4,60	4,53	8,91	6,00
C Depth of Wet Well	m	4,33	7,09	7,85	3,60	5,19	6,09	5,01
D Invert Level Pressure Pipe	m	3,80	7,97	1,50	3,51	3,42	7,77	4,75
DN Inlet	mm	600	500 and 200	500	200	200	200	300
DN Pressure Pipe Pump at Discharge Port	mm	150	150	100	50	50	65	150
HDPE Pressure Pipe Outlet	mm	630	400	400	90	110	140	250
DN Flushing and Emptying Device	mm	150	150	100	50	50	65	150
DN Inlet Sluice Gate Valve	mm	600	500 and 200	500	200	200	200	300
Planned area around Inlet Sluice Gate Valve	cm x cm	80x80	80x80	80x80	50x50	50x50	50x50	70 x70
DN Check Valve	mm	150	150	100	50	50	65	150
Type of Check Valve		Swing	Swing	Swing	Ball	Ball	Ball	Swing
DN Gate Valve	mm	150	150	100	50	50	65	150
Cover size	cm	110x 300	110 x 300	110 x 300	100 x 120	100 x 120	100x 120	120 x 220
Number of Pumps (duty + stand by)		3 (2+1)	3(2+1)	3 (2+1)	2 (1+1)	2(1+1)	2 (1+1)	2 (1+1)
Free passage	mm	135	135	135	40	40	65	120
Power Input of each Pump	KW	38	18	16,5	1,3	2,3	4,2	8,7
Variable Frequency Drive		Yes	Yes	Yes	No	No	No	No
Type of PS: Roadway / Sideway		S	S	S	S	S	S	R
Generator Capacity	KW	120	60	60	5	5	10	15
Generator type: Fix / Mobile		F	F	F	F	F	F	F
Switchboard type: Freestanding / Integrated in generator		I	I	I	I	I	I	I

Appendix 3: Famagusta Sewer Lot 5.




L3
L5
PS-2


L5
L4

Appendix 4: The Main Discharge Pipeline Details

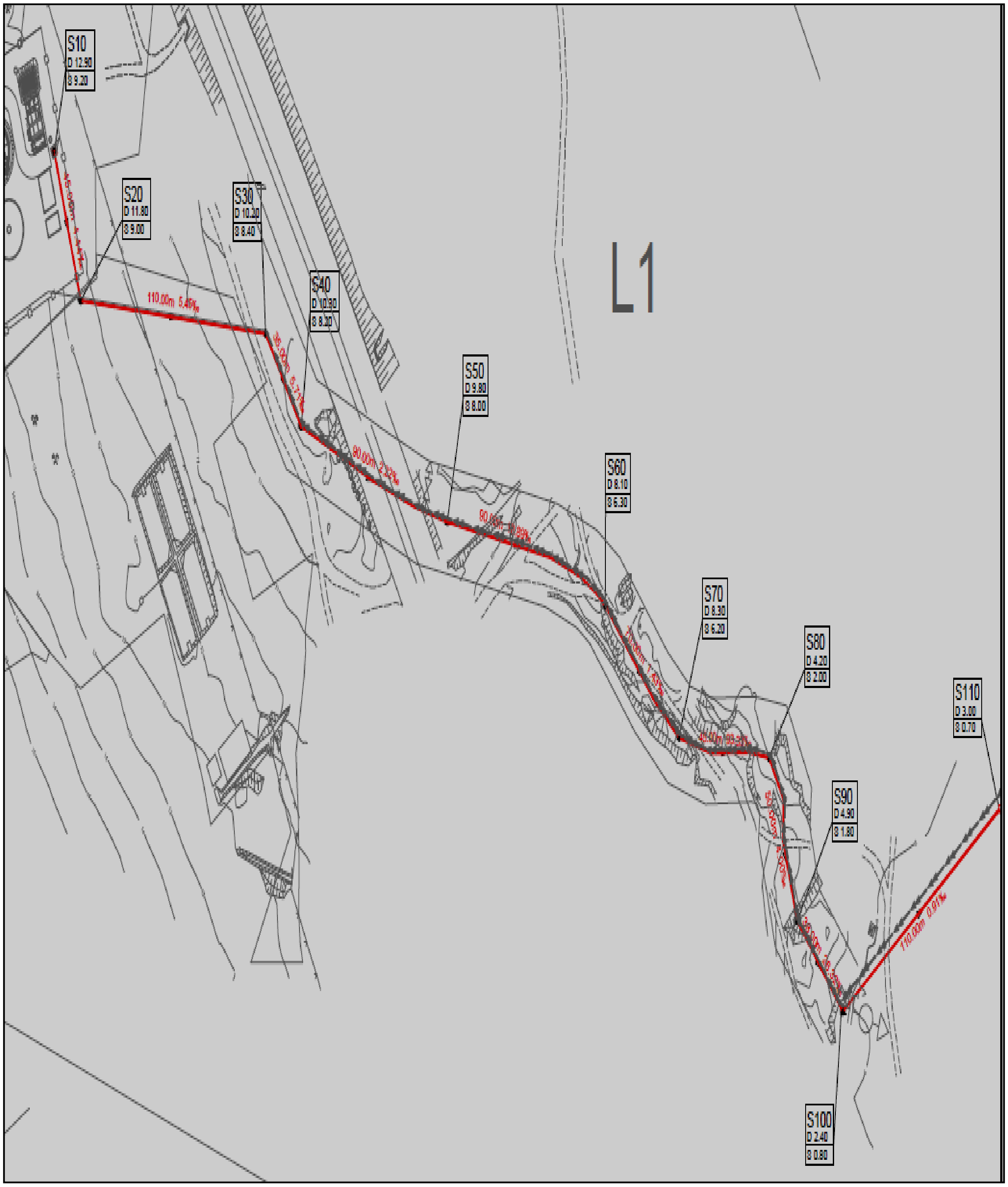


Table 7.1 Hydrulic design of Famagusta sewer system of Lot 5

From MH	To MH	Distance (m)	Slope	Diamete mm	Ground level(m)	Invert level(m)	Depth(m)	WW.discharge lt/s	Sum.Contributy areas m2	Over all peak factor	Max. WW dischargee lt/sec	Infltration lt/sec	Rain fall lt/sec	Q max lt/sec	Cum. Qmax lt/sec	24 hour P.F	24 hour Qmax lt/sec	Qdry lt/sec	Cum. Qdry lt/sec	Diameter calculation m	Diameter checking m
1945	1946	63	0,0116	200	13,07	11,16	1,91	0,085	2457	4	0,341	0,049	0,074	0,464	0,464	2,5	0,213	0,262	0,262	0,037	0,2
1946	1947	64	0,0107	200	12,34	10,43	1,91	0,087	2496	4	0,347	0,050	0,075	0,471	0,936	2,5	0,21667	0,267	0,262	0,049	0,2
1947	1948	59	0,0277	200	11,65	9,74	1,91	0,080	2301	4	0,320	0,046	0,069	0,435	1,370	2,5	0,19974	0,246	0,262	0,047	0,2
1948	1949	59	0,0135	200	10	8,09	1,91	0,080	2301	4	0,320	0,046	0,069	0,435	1,805	2,5	0,19974	0,246	0,262	0,060	0,2
1949	1950	59	0,0051	200	9,66	7,75	1,91	0,080	2301	4	0,320	0,046	0,069	0,435	2,239	2,5	0,19974	0,246	0,262	0,078	0,2
1950	1951	59	0,0051	200	9,4	7,45	1,95	0,080	2301	4	0,320	0,046	0,069	0,435	2,674	2,5	0,19974	0,246	0,262	0,083	0,2
1951	1952	59	0,0051	200	9,31	7,15	2,16	0,080	2301	4	0,320	0,046	0,069	0,435	3,109	2,5	0,19974	0,246	0,262	0,088	0,2
1952	1953	59	0,0051	200	9,59	6,85	2,74	0,080	2301	4	0,320	0,046	0,069	0,435	3,543	2,5	0,19974	0,246	0,262	0,093	0,2
1953	1956	71	0,0021	200	9,59	6,55	3,04	0,096	2769	4	0,385	0,055	0,083	0,523	4,066	2,5	0,24036	0,296	0,262	0,115	0,2
1956	1976	72	0,013	200	9,45	6,4	3,05	0,098	2808	4	0,390	0,056	0,084	0,530	4,597	2,5	0,24375	0,300	0,262	0,086	0,2
1967	1968	56	0,0098	200	10,27	8,36	1,91	0,076	2184	4	0,303	0,044	0,066	0,413	5,009	2,5	0,18958	0,233	0,262	0,093	0,2
1968	1970	75	0,002	200	9,72	6,06	3,66	0,102	2925	4	0,406	0,059	0,088	0,553	5,562	2,5	0,25391	0,312	0,262	0,131	0,2
1970	1972	71	0,0021	200	9,17	5,91	3,26	0,096	2769	4	0,385	0,055	0,083	0,523	6,085	2,5	0,24036	0,296	0,262	0,134	0,2
1972	1975	67	0,0021	200	9,13	5,76	3,37	0,091	2613	4	0,363	0,052	0,078	0,494	6,578	2,5	0,22682	0,279	0,262	0,138	0,2
1975	1976	68	0,0021	200	9,33	5,62	3,71	0,092	2652	4	0,368	0,053	0,080	0,501	7,079	2,5	0,23021	0,283	0,262	0,142	0,2
1969	1970	41	0,0051	200	9,33	7,42	1,91	0,056	1599	4	0,222	0,032	0,048	0,302	7,381	2,5	0,1388	0,171	0,262	0,122	0,2
1971	1972	57	0,0051	200	9,12	7,21	1,91	0,077	2223	4	0,309	0,044	0,067	0,420	7,801	2,5	0,19297	0,237	0,262	0,124	0,2
1973	1974	43	0,0051	200	9,06	7,15	1,91	0,058	1677	4	0,233	0,034	0,050	0,317	8,118	2,5	0,14557	0,179	0,262	0,126	0,2
1974	1975	42	0,005	200	9,07	6,93	2,14	0,057	1638	4	0,228	0,033	0,049	0,309	8,427	2,5	0,14219	0,175	0,262	0,129	0,2
1954	1955	41	0,0051	200	9,2	7	2,2	0,056	1599	4	0,222	0,032	0,048	0,302	8,730	2,5	0,1388	0,171	0,262	0,130	0,2
1955	1956	41	0,0051	200	9,41	7,32	2,09	0,056	1599	4	0,222	0,032	0,048	0,302	9,032	2,5	0,1388	0,171	0,262	0,131	0,2
1976	1980	15	0,002	200	9,78	5,48	4,3	0,020	585	4	0,081	0,012	0,018	0,111	9,142	2,5	0,05078	0,062	0,262	0,157	0,2
1977	1978	40	0,0053	200	9,28	7,37	1,91	0,054	1560	4	0,217	0,031	0,047	0,295	9,437	2,5	0,13542	0,167	0,262	0,133	0,2
1978	1979	41	0,0051	200	9,52	7,16	2,36	0,056	1599	4	0,222	0,032	0,048	0,302	9,739	2,5	0,1388	0,171	0,262	0,135	0,2
1979	1980	9	0,0055	200	9,55	6,95	2,6	0,012	351	4	0,049	0,007	0,011	0,066	9,805	2,5	0,03047	0,037	0,262	0,134	0,2
1980	1981	43	0,0021	200	9,67	5,45	4,22	0,058	1677	4	0,233	0,034	0,050	0,317	10,122	2,5	0,14557	0,179	0,262	0,162	0,2
1981	1988	42	0,0021	200	9,75	5,36	4,39	0,057	1638	4	0,228	0,033	0,049	0,309	10,431	2,5	0,14219	0,175	0,262	0,164	0,2
1982	1983	37	0,0051	200	9,9	7,99	1,91	0,050	1443	4	0,200	0,029	0,043	0,273	10,704	2,5	0,12526	0,154	0,262	0,140	0,2
1983	1984	37	0,0054	200	9,77	7,8	1,97	0,050	1443	4	0,200	0,029	0,043	0,273	10,976	2,5	0,12526	0,154	0,262	0,140	0,2
1984	1985	50	0,0052	200	9,51	7,6	1,91	0,068	1950	4	0,271	0,039	0,059	0,368	11,345	2,5	0,16927	0,208	0,262	0,143	0,2
1985	1987	51	0,0051	200	9,25	7,34	1,91	0,069	1989	4	0,276	0,040	0,060	0,376	11,720	2,5	0,17266	0,212	0,262	0,145	0,2
1987	1988	23	0,0052	200	9,05	7,08	1,97	0,031	897	4	0,125	0,018	0,027	0,169	11,890	2,5	0,07786	0,096	0,262	0,145	0,2
1986	1987	39	0,0053	200	9,25	7,34	1,91	0,053	1521	4	0,211	0,030	0,046	0,287	12,177	2,5	0,13203	0,162	0,262	0,146	0,2
1988	1989	47	0,0021	200	9,25	5,27	3,98	0,064	1833	4	0,255	0,037	0,055	0,346	12,523	2,5	0,15911	0,196	0,262	0,175	0,2
1989	1990	46	0,0025	200	9,34	5,7	3,64	0,062	1794	4	0,249	0,036	0,054	0,339	12,862	2,5	0,15573	0,192	0,262	0,171	0,2
1990	1994	65	0,002	200	9,8	5,07	4,73	0,088	2535	4	0,352	0,051	0,076	0,479	13,341	2,5	0,22005	0,271	0,262	0,181	0,2
2031	2033	58	0,0021	200	8,14	5,1	3,04	0,079	2262	4	0,314	0,045	0,068	0,427	13,768	2,5	0,19635	0,242	0,262	0,182	0,2
2033	2035	65	0,002	200	7,9	4,98	2,92	0,088	2535	4	0,352	0,051	0,076	0,479	14,247	2,5	0,22005	0,271	0,262	0,186	0,2
2035	2036	44	0,0021	200	8,04	4,85	3,19	0,060	1716	4	0,238	0,034	0,051	0,324	14,571	2,5	0,14896	0,183	0,262	0,186	0,2
2036	2037	45	0,002	200	8,23	4,76	3,47	0,061	1755	4	0,244	0,035	0,053	0,332	14,903	2,5	0,15234	0,187	0,262	0,189	0,2
2037	2038	50	0,002	200	8,51	4,67	3,84	0,068	1950	4	0,271	0,039	0,059	0,368	15,271	2,5	0,16927	0,208	0,262	0,191	0,2
2038	2045	45	0,002	200	8,57	4,57	4	0,061	1755	4	0,244	0,035	0,053	0,332	15,603	2,5	0,15234	0,187	0,262	0,192	0,2
2045	2048	61	0,002	200	8,81	4,48	4,33	0,083	2379	4	0,330	0,048	0,071	0,449	16,052	2,5	0,20651	0,254	0,262	0,194	0,2
2042	2043	67	0,0051	200	8,98	5,6	3,38	0,091	2613	4	0,363	0,052	0,078	0,494	16,546	2,5	0,22682	0,279	0,262	0,165	0,2
2043	2044	62	0,005	200	9,27	5,26	4,01	0,084	2418	4	0,336	0,048	0,073	0,457	17,002	2,5	0,2099	0,258	0,262	0,167	0,2
2044	2045	63	0,0074	200	8,98	4,95	4,03	0,085	2457	4	0,341	0,049	0,074	0,464	17,466	2,5	0,21328	0,262	0,262	0,157	0,2
2046	2047	40	0,0135	200	9,4	7,49	1,91	0,054	1560	4	0,217	0,031	0,047	0,295	17,761	2,5	0,13542	0,167	0,262	0,141	0,2
2047	2048	40	0,005	200	8,86	6,95	1,91	0,054	1560	4	0,217	0,031	0,047	0,295	18,056	2,5	0,13542	0,167	0,262	0,171	0,2
2032	2033	71	0,005	200	8,02	6,11	1,91	0,096	2769	4	0,385	0,055	0,083	0,523	18,579	2,5	0,24036	0,296	0,262	0,173	0,2
2034	2035	76	0,005	200	7,95	6,04	1,91	0,103	2964	4	0,412	0,059	0,089	0,560	19,139	2,5	0,25729	0,317	0,262	0,175	0,2
2029	2030	52	0,0021	200	8,3	5,32	2,98	0,070	2028	4	0,282	0,041	0,061	0,383	19,522	2,5	0,17604	0,217	0,262	0,207	0,2
2030	2031	52	0,002	200	8,2	5,21	2,99	0,070	2028	4	0,282	0,041	0,061	0,383	19,905	2,5	0,17604	0,217	0,262	0,211	0,2
2013	2014	54	0,002	200	7,82	5,77	2,05	0,073	2106	4	0,293	0,042	0,063	0,398	20,303	2,5	0,18281	0,225	0,262	0,212	0,2
2014	2015	60	0,002	200	7,98	5,66	2,32	0,081	2340	4	0,325	0,047	0,070	0,442	20,745	2,5	0,20313	0,250	0,262	0,214	0,2
2015	2019	65	0,005	200	8,24	5,54	2,7	0,088	2535	4	0,352	0,051	0,076	0,479	21,223	2,5	0,22005	0,271	0,262	0,182	0,2
2019	2020	59	0,0123	200	8,68	5,21	3,47	0,080	2301	4	0,320	0,046	0,069	0,435	21,658	2,5	0,19974	0,246	0,262	0,155	0,2
2020	2048	64	0,005	200	8,72	4,48	4,24	0,087	2496	4	0,347	0,050	0,075	0,471	22,129	2,5	0,21667	0,267	0,262	0,185	0,2
1996	1997	61	0,0021	200	8,86	4,73	4,13	0,083	2379	4	0,330	0,048	0,071	0,449	22,579	2,5	0,20651	0,254	0,262	0,219	0,2
1997	2020	57	0,002	200	8,74	4,6	4,14	0,077	2223	4	0,309	0,044	0,067	0,420	22,999	2,5	0,19297	0,237	0,262	0,222	0,2
2016	2017	51	0,005	200	8,83	6,29	2,54	0,069	1989	4	0,276	0,040	0,060	0,376	23,374	2,5	0,17266	0,212	0,262	0,188	0,2
2017	2018	51	0,005	200	8,68	6,66	2,02	0,069	1989	4	0,276	0,040	0,060	0,376	23,750	2,5	0,17266	0,212	0,262	0,190	0,2
2018	2019	65	0,005	200	8,38	6,4	1,98	0,088	2535	4	0,352	0,051	0,076	0,479	24,229	2,5	0,22005	0,271	0,262	0,191	0,2
2001	2002	46	0,002	200	8,13	6,02	2,11	0,062	1794	4	0,249	0,036	0,054	0,339	24,568	2,5	0,15573	0,192	0,262	0,228	0,2
2002	2003	40	0,002	200	8,35	5,99	2,36	0,054	1560	4	0,217	0,031	0,047	0,295	24,863	2,5	0,13542	0,167	0,262	0,229	0,2
2003	2010	35	0,002	200	8,12	5,91	2,21	0,047	1365	4	0,190	0,027	0,041	0,258	25,120	2,5	0,11849	0,146	0,262	0,230	0,2
2010	2015	60	0,005	200	8,04	5,84	2,2	0,081	2340	4	0,325	0,047	0,070	0,442	25,562	2,5	0,20313	0,250	0,262	0,195	0,2
2008	2009	56	0,002	200	8,08	6	2,08	0,076	2184	4	0,303	0,044	0,066	0,413	25,975	2,5	0,18958	0,233	0,262	0,233	0,2
2009	2010	19	0,002	200	7,97	5,88	2,09	0,026	741	4	0,103	0,015	0,022	0,140	26,115	2,5	0,06432	0,079	0,262	0,233	0,2