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Selection of Best Locations for Decentralized Sulaimania Municipal Wastewater Using GIS and AHP with Potential Treatment and Reusing

A Thesis Submitted to the Council of College of Engineering – University of Sulaimani as a Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy in Environmental Engineering

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بِسْمِ اللهِ الرَّحْمنِ الرَّحيمِ

فَتعالى اللهْ المَلكُ الحَقّ ولا تَعْجَل ْ بِالقُرآن مِنْ
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عِلْمًا۞

صدق الله العظيم

سورة طه الاية 114

Dedication

To the Memory of my Father
To my Mother
To my Family and Friends

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Selection of Best Locations for Decentralized Sulaimania Municipal Wastewater Using GIS and AHP with Potential Treatment and Reusing

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ABSTRACT

Water shortage is one of the crucial problems in Sulaimania city which issued as a result of population growth, climate changes, water overuse and other reasons. To minimize this issue, decentralized wastewater treatment units (DWWTUs) proposed in this research which are efficient, affordable and are easy to install and operate and the treated wastewater will be reused for the irrigation of the green areas of the city. Moreover, there is no wastewater treatment plant in Sulaimania city and the wastewater is discharged directly to Qilyasan stream through several outlet points, and that causes many critical environmental issues. The selected treatment type is activated sludge extended aeration (EA) package treatment plant.

One of the main objectives of this study was to select the best and suitable locations for those DWWTUs. Preliminary 134 nominated areas (NA) were selected at different locations across the city based on general location's suitability. A model was developed to evaluate and optimize the NAs using GIS software integrated with Analytical Hierarchy Process (AHP). Five criteria were used the model; (1) the size of available lands, (2) the distance from the DWWTUs to the GRs, (3) population density around the DWWTUs locations, (4) the slope of the land and (5) the depth of the main existing sewer pipe at the NA. Moreover, the model adopted two restriction factors; (1) the distance from the DWWTUs to the buildings should not be less than 30 m, and (2) the distance between the sewer main boxes and the DWWTUs is < 50 m. From the results of the analysis 6 different classes of suitability levels of the NAs are produced starting from restricted to extremely suitable level.

Each NA has more than one suitability class level. Normalized weighted average (NWAV) of the suitability level % of each NA was found. Areas having NWAV less than 0.5 were eliminated and in conclusion, only 31 suitable locations were selected from the 134 NAs.

The second aim of this study is to develop an optimization model to find the least cost (F_{min}) of treating and conveying the reclaimed water from the DWWTUs to the GRs. The cost includes the construction, operation, and maintenance of the DWWTUs, cost of pumping the reclaimed water, and cost of the conveying pipe networks. The number of GRs are 827 with different sizes and total area of 4.74 Km^2. The reclaimed water conveyed to the GRs through piping networks, which are either gravity flow pipes or pressurized flow pipes based on the magnitude of the pipe head loss and the topography of the locations.

A transportation matrix model of size [31x827] was developed to find the optimum cost of conveying the reclaimed flow from the DWWTUs (origin) to the GRs (destinations). The shortest pipe lengths and best routes were found using Network Analysis - OD Cost Matrix method in ArcGIS 10.2. The elevations of the DWWTUs' locations and the GRs were found from the GIS DTM map.

Genetic Algorithm (GA) in a matrix representation form was used to solve the optimization model using a developed Matlab 2018a software program code. A random number of solutions (*Np* = 100, 200, 300, 400, 500, 600, 700, 800, 900 and 1000) were created based on

different amounts of treated flows, and each solution represents a chromosome. For all *NP* value, three runs and four iterations were tried. The minimum NP size that produces stable optimum results found at *NP* = 500. Different locations of crossover point (*PCO*) examined to achieve the minimum cost value F_{min}. The optimum minimum cost found at *NP* = 500 and *PCO* = 632.

Based on the results of the least value of the objective function (F_{min}) , the optimum capacities of the 31 DWWTUs were obtained, and they were ranged from 150 m^3/day to 2,100 m^3/day with an entire treated flow was found to be 26,150 m^3/day.

The sludge produced from the DWWTUs were digested in aerobic digesters and transported to a one sand drying bed. Suitable location for the sand drying bed was selected at south west of Sulaimania city.

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LIST OF ABREVIATIONS

List of Abbreviations

Symbol	**Description**
AHP	Analytical Hierarchy Process
amsl	Above mean sea level
AS	Activated sludge
DOSS	Directorate of Statistic of Sulaimania
DOSWS	Directorate of Sewerage of Sulaimania
DOWS	Directorate of Water of Sulaimania
DWWTU	Decentralized wastewater treatment unit
DTM	Digital Terrain Model
DWF	Dry weather flow
EA	Extended aeration
EPA	Environmental Protection Agency
ET	Evapotranspiration
ETc	Water requirement for irrigating the crops, , m^3/day
ETo:	Referenced evapotranspiration, mm/day
FMD	Fussy multi criteria decision making process
GA	Genetic Algorithm
GDOSM	General Directorate of Sulaimania Municipality
GDOSM - GIS	General Directorate of Sulaimania Municipality - GIS
GDOSM - Garden	General Directorate of Sulaimania Municipality – Gardens
GIS	Geographical Information System
GR	Green area
HDPE	High density polyethylene
MCDM	Multi - criteria optimization model
MLSS	Mixed liquor suspended solid, mg/L
MLVSS	Mixed liquor volatile suspended solid, mg/L
NA	Nominated area
NP	Number of population of genetic algorithm
NWAV	Normalized weighted average value
O&M	Operation and maintenance
ONA	Optimized nominated area
PCO	Point of Crossover
PE	Population equivalent
Re	Reynold number
WAV	Weighted average value
WHO	World health organization
WLC	Weighted linear combination
WWF	Wet weather flow
WWTP	Wastewater treatment plant

CHAPTER ONE

INTRODUTION

Chapter One
Introduction

1.1 General

Lack of water nowadays is one of the global issues all over the world and there are many reasons behind that such as; population growth, urbanization, climate changes, water overuse, water pollution, and struggles between countries to dominate water sources. On the other hand, there are many demands of water in each community like; domestic, commercial and institutional, irrigation, agricultural, firefighting, street washing, industrial uses, loses and others (J.McGhee, 1999, p. 11). There is no balance between demand and supply and new scales are required, that will be by the use of advanced technologies and managements which must be applied to the education, environment, and establishment. (Pereira, et al., 2002, p. 9).

There are many suggestions to solve the problem of water shortage, for instance, controlling the water losses and non-useful water, promoting groundwater recharging, water gathering like building small dams, and reusing of treated wastewater water. Savoury water, brackish water, agricultural drainage water, toxic water and deposits, as well as treated or untreated sewage effluents are defined as wastewater (Pereira, et al., 2002, p. 13). Reusing of domestic treated wastewater is one of the effective methods that contribute in covering the water requirements of indirect human water uses.

Decentralized wastewater treatment system is considered as a good alternative for water reusing. Decentralization may be defined as the collection, treatment, disposal or reuse of wastewater at or near points of production. It is used to treat wastewaters that are produced from homes, gathers of houses, separated communities, and industrial areas and from other communities' portions (Techobanoglous, 1998, p. 2).While centralized treatment plants require conveying wastewater from large areas to one large plant. DWWTUs are installed inside the city and their costs are less than the costs of centralized treatment units as their sizes are small and they do not need long pipes for conveying the treated wastewater. Moreover, the sizes of the pipes are not large as small amounts of reclaimed water will be conveyed and there is no need to use large sewer collection pipes.

Wide ranges of treated wastewater reusing options for small and decentralized wastewater systems are exist. The reusing purpose of the reclaimed water from the decentralized treatment unit will specify the locations, sizes and the treatment method. Landscape irrigation are the most common forms of water reusing which includes irrigation of : (1) parks, (2) school yards (3) golf course, (4) freeway medians, (5) cemetery, (6) green belts and (7) residential areas (Eddy, 2014, p. 1143).

The optimum design of the DWWTUs will be obtained at maximum benefits of reusing and minimum costs of treatment and conveying the reclaimed water. It is required to select proper locations of the treatment units and select suitable treatment unit's system, type and capacity. There are many factors that should be considered when selecting the locations like environmental, economical, technical, social criteria and health precautions. GIS could be used to select the best location of the treatment units.

1.2 Statement of the Problem

This study is carried out in Sulaimania City which located in North of Iraq and it consists of four suburbs; Main suburbs, Bakrajo, Rapareen and Tasloja (GDOSM-GIS, 2017) and the research is done for Sulaimania main suburb .The study area suffers from lack of water for domestic demand, irrigation and other usages. There are three main sources of water in Sulaimania City which are; (1) Sarchinar natural springs, located 5 Km northwest of the center of the city,(2) Dukan lake and (3) water wells in Sulaimania City which are belong to Directorate of Water of Sulaimania and there are many private wells also (Sharief, 2013, pp. (15 - 20)). The amounts of water that delivered from the sources mentioned are not sufficient to cover all the city's requirements (DOWS, 2017). There are many reasons of the water crisis in the study areas like the rapid expansion of the city as the number of districts at the main suburb was 78 at 2003 (Seureca, 2003, pp. Annex 1 - (1- 4)) and now the number became 156 districts (GDOSM-GIS, 2017). Moreover, many big villages recently became part of the city such as; Kanaswra, Kani Goma, Qaratoghan, Hawana, Kani Bardina, Khewata and Kalakn (GDOSM-GIS, 2017). Those villages are currently supplied with water from the city as well.

In addition to that because of the political reasons after 2003 and 2014 immigrations from the surrounding areas to the city occurred and that also increased the water demand. One of the other important factor is the climate changes in the area, which recently shows increasing in temperature that effects on the amount of precipitation (Al-Ansari, et al., 2018, p. 48).

Reusing the treated wastewater of the city is one of the possible ways to solve the problem of water shortage. There is no wastewater treatment plant in Sulaimania City and all the sewage is discharged into Qilyasan Stream directly without treatment. The sewer network of the city is divided into 10 separate groups and the flow from each group is discharged through outlets to Qilyasan Stream. Fig.(1.1) shows the outlets of two sewer boxes in the study area. DWWTUs are suggested at different places in the city and the treated wastewater will be reused for irrigating the green areas near the units. Also treating the wastewater will reduce the pollution of Qilyasan Stream.

Fig.(1.1): The Locations of Two Sewage Outlets in the Study Area, Outlet 1 and 2 in Awabaraw Asha Spi 418 Sub-Districts, (Researcher)

Figs. (1.2) and (1.3) show the outlets of the sewer boxes in two different districts in Sulaimania city (Awabara and Gwndi Kanaswra).

Fig.(1.2): Double Sewer Box Outlet in Awabara Sub- District, (Researcher)

Fig.(1.3): Sewer Box Outlet in Gwndi Kanaswra, (Researcher)

There are many green areas in Sulaimania City which are in a form of big green parks, located in the road medians, inside the residential areas and residential complexes. The green areas suffer from lack of water as it depends mainly on wells inside some of the areas. Other green areas receive water by trucks (GDOSM-Gardens, 2017). To cover the water shortage in the green areas reclaimed water from treated wastewater could be a good option for irrigation purposes.

To get the maximum benefit from the DWWTUs it is required to specify the optimum sizes of the units and their proper locations inside the city. The suitable locations will have an effect on the method and cost of conveying the treated water to the green areas.

1.3 The Objectives of the Study:

The main aims of this study are:

1. Find optimum locations and numbers of the DWWTUs for Sulaimania City as a case study and to be used for irrigation purposes using GIS and AHP.
2. Determine optimum cost of the treatment units and the conveying cost of the reclaimed water to the irrigation areas. Moreover, specifying the

optimum size of each DWWTU for the maximum benefit using optimization transportation model and GA.
3. Design treatment processes of the produced sludge from the DWWTUs.

1.4 Scope of the Work

Several steps were done to achieve the work aims as in below:

a. Site investigations and authority representative visits and interviews were done to collect data and information related to the sewerage and water systems, population, GIS maps and study of the green areas.
b. Correct some GIS maps of the green areas, main sewer boxes and add missed drawings from as built drawings with the corporation of the Project Executive Department of Sulaimania Municipality. Moreover, adding all the information related to the sewer box into the GIS attribute file such as; the boxes dimensions, depths, slopes and the hydraulic elements.
c. Estimating the water demand of the GRs by collecting information from the Directorate of the Gardens of Sulaimania City related to the GR size and locations.
d. A number of DWWTUs were suggested inside Sulaimania City to solve the problem of water scarcity by reusing the treated water for irrigating the green areas. GIS models were accomplished to find the optimum number and best locations of the DWWTUs
e. Network analysis – OD matrix GIS model is used to find the best paths of the supplying pipes from the DWWTUs to the GRs.
f. Using a matrix form of GA optimization model to find the best sizes of the DWWTUs that have a minim cost. Sensitivity analysis was done by changing the population number NP for different PCO locations to find the stable solution.
g. A preliminary design of the DWWTUs were done and a location was suggested for the sludge drying beds in the city using GIS map.

1.5 The Novelty of the Work:

The novelty of this research could be categorized as in below:

1. In this research, a new optimization model with an original objective function was developed that coupled a matrix GA and GIS to find the best locations and sizes of the DWWTUs.
2. The size of the transportation array that used in the GA model is large and it is a first time that such matrix scale to be used in GA.
3. Moreover, the crossover method in the mating process was done by column to satisfy the constraint and the process was coded in Matlab program.
4. In addition, GIS – Network Analysis OD Matrix tool is used for the first time in finding the best path rout of pipe networks.

1.6 The Thesis Layout:

The layout of the thesis divided into six chapters and two appendices (A and B). Chapter one is the introduction chapter and chapter two is the literature review chapter which shows the previous works of other researchers related to optimization methods of wastewater treatment plants, reusing of wastewater and the sludge treatments. Chapter three is about the Theoretical Concepts of the work and it explains the applications of GIS, Transportation model and GA in solving the model.

Chapter four consists of four major parts which are; (1) finding the suitable locations of the DWWTUs in the study area using GIS and AHP, (2) finding the optimum numbers and sizes of the DWWTUs to be reused for irrigating the green areas of the study area using transportation model and GA, (3) designing the dimensions of the treatment units and (4) treating the sludge that produced from each unit. Chapter five is related to Results and discussion. Chapter six is about the conclusion of the results and the recommendations for future work as well as the publications related to the study.

CHAPTER TWO

LITERATURE REVIEW

Chapter Two
Literature Review

2.1 Introduction

Many studies have been done related to DWWTUs all over the world from many perspectives and some of the studies adopted the theoretical side and other researches implemented practical works. This chapter will focus on previous literatures related to cost optimization models of wastewater treatment units and reclaimed water conveying, using GIS in selecting suitable locations for the treatment unit locations, reusing of the treated wastewater researches, wastewater treatment technologies and sludge treatment methods.

2.2 Wastewater in Sulaimania City

In Sulaimania City there is no wastewater treatment plant and the sewage is discharge directly to Qilyasan Stream through a number of sewer box outlets without treatment which causes many environmental pollutions. (Rasheed, 2017) tested the wastewater flow from nine outlets and they found that the wastewater contains many contaminates such as heavy metals with concentrations exceeding the allowable limits of environmental regulations. The amount of BOD and COD that they found were (66.75 – 79.5) mg/L and (65 – 116) mg/L respectively. Another study in Sulaimania City was performed by (Amin, 2018) to find the quality of the wastewater in three different places. The results showed that the BOD values ranged from (15 – 58) mg/L and COD ranged from (10 -110) mg/L and regarding the TSS it was (84-284) mg/L.

2.3 Wastewater Treatment

Sewerage systems were developed to collect and remove wastewaters from the sources to a safe disposal point. The treatment system could be centralized or decentralized and in centralized treatment, it is required to transmit wastewater from a large area to one large treatment plant. While decentralization is defined as the gathering, treatment, and reuse of wastewater at or near its source of generation (Techobanoglous, 1998, p. 39).

2.3.1 Decentralized Wastewater Treatment

Many technologies used in DWWTUs, which depend on influent quality and effluent requirements. There are many benefits in using DWWTUs as they are economic, have flexibility of construction, operation and maintenance and the treated wastewater could be reused easily for many purposes. Decentralization has proceed from the needs of reclamation and cities expansion and most of the previous work suggests that it is more of a recent demand. Various researches were done to evaluate methods of treatments and reclaimed water reusing. (Singh, 2015) reviewed a list of implemented full-scale DWWTUs all over the world in terms of their technology and performance, area required and cost of construction, operation and maintenance. Four main types of DWW technologies were categorized; (1) natural treatment system, (2) aerobic treatment system, (3) anaerobic treatment system, and (4) combined (aerobic, anaerobic and natural) system. Examples for existing plants were given for each type and comparisons were made between them. It was found that natural method had low cost, low energy consuming, satisfied effluent quality but it requires large area and high hydraulic retention time (HRT). Aerobic system was more efficient and needs less HRT, the starting time was (2 - 4) weeks, small footprint was required, no odor released, and small amount of produced sludge, but it needed high energy and high operation skills. The effluents from anaerobic system were not efficient, produce odor and it took (3 - 4) months to start up which was a long period. Meanwhile, (Shehabi, 2012) made an evaluation of existing centralized and decentralized wastewater treatment systems in California in terms of treatment and distribution processes, water reuse, energy recovery and gas emission from the treatment process. The decentralized system consisted of septic tanks that used for 47–lots suburb subdivision of Stonehurst in Martinez City (California) with capacity of 5.7 m^3 for each lot. The effluent from each septic tank was transported through sewer pipes of 5 cm diameter (gravity pipelines or pressurized pipelines) to a community treatment plant. The treatment plant consisted of; a recirculating sand filter, disinfection by ultra - violet (UV), pumping units and dosing tank. The reclaimed water was conveyed to community soil absorption area of 10,000 m^2 . Moreover, the treated wastewater used for irrigation using subsurface drip system for a small park. Desludging process were done each 5 years and the produced sludge were anaerobically digested and dewatered then disposed in landfills. The centralized treatment unit was utilized for 500,000 capita and conventional wastewater treatment was used. The results showed that the energy required for decentralized system operation is seven times more the energy required for centralized system. The DWWTU scheme was considered as low technology design as operation system required significant electricity. Moreover, concentrations of greenhouse gas emission expressed in CO2 emission was much higher in DWWTU because of the anaerobic reaction, which produces methane gas.

2.3.2 Centralized Wastewater Treatment

Centralized wastewater treatment, widely practiced in developed areas, involves transporting wastewater from large urban or industrial areas to a large capacity plant using a single network of sewers. (Arslan, , 2007) listed the existing urban UWWTPs in Turkey. Only 43 Governorate had WWTP out of 81 and the number of plants that were operating was 129 WWTPs. Three of those plants were in big industrial areas. The effluents from the WWTP were discharged both into coastal water and into inland water or disposed over land. The study focused on four selected WWTPs and samples were taken from the influent and effluent of the plants to be analyzed and the performance of the plants were evaluated based on their capacity, treatment technique and discharge method and reusing potential. The treatment method of the first WWTP was activated sludge with treatment capacity of 1,350 m^3/day and the second WWTP's design was activated sludge followed by oxic and anoxic zones with a capacity of 100,000 m^3/day. Both treatment plants discharged their flow into a river. The other two WWTPs had tertiary treatment systems with nitrogen and phosphorus removal and their capacities were 110,000 m^3/day and 227,000 m^3/day. The effluents of the two WWTPs were discharged into the Black Sea and the Mediterranean Sea respectively. The results of the experiment analysis of the four WWTPs showed that the plants were operating efficiently in terms of percentage of removal of organic matters and sulfate. According to the National Irrigation Water Quality in Turkey for water reusing standards, the effluents from each plant in the study were evaluated and it was clear that none of the plants were suitable for irrigation because of fecal coliform's values as there was no disinfection in the plants. The authors recommended to apply disinfection units in the treatment process to get suitable water for reusing for irrigation.

(Al-Shammari, 2019) evaluated the performance of Jahar EA treatment plant in Kuwait with a capacity of 65,000 m^3/day. The plant consists of equalization basin, grit removal chamber, 6 aeration tanks, 6 secondary clarifiers, chlorination and filtration. The evaluation was done by taking weekly samples from the influent and effluent of the EA plant lines for a duration of 12 months to assess the quality of the treated wastewater. The samples were tested and the collected data were statically analyzed. The results showed that the plant had a high efficiency performance for the removal of BOD, COD and TSS with a percentages equal to 85%, 81% and 86.3 % respectively.

2.4 Land Suitability Selection Using GIS

Site selections can be successfully achieved by decision analysis tool used in GIS. (Meinzinger, 2003) selected suitable locations to use Land Application Method for a treated sewage produced from a wastewater treatment plant in Christchurch City in New Zealand which had a flow of 630,000 m^3/day by using GIS. The nominated areas to be evaluated and analyzed using GIS were located in Christchurch City and three other neighboring regions. This method is very effective for water reusing for agricultural purposes. The selection was based on a number of factors which were; (1) social acceptability, (2) land use by using land cover database to specify areas where a land application for wastewater is possible , keeping residential areas far from the selected sites by applying a buffer layer of 150 m from the buildings , historic places were excluded and, transport distance from the site to the treatment plant was considered as a critical factor in the ranking process,(3) soil ; soil types, depth (> 0.6 m was selected) and pH (5.5 – 8.3), (4) economic criteria, (5) climate, (6) the land slope (in DEM map slope > 35% was excluded), (7) environmental factors related to surface water pollution and groundwater table > 1m. The criteria above were weighted and introduced into the GIS. The results of the GIS were illustrated in a raster map showed the suitable lands for the application of the method. Additional selections were made from the selected suitable land results for areas > (16,000) ha, as a minimum requirement and in conclusion, four suitable lands were founded in the study area.

(Deepa, 2012) used GIS with AHP to build a multi -criterion model to find Cumulative Suitability Index (CSI) to select suitable locations for DWWTUs in Chennai City in India, the study area was 118 Km^2. The authors selected six parameters (layers in GIS) for determination of suitable sites, which were: (1) land use (land availability), (2) population density, (3) soil type, (4) slope, (5) cost and (6) technology. The parameters were weighted by using AHP method and the results are shown in table (2.1).

Table (2.1): The Weights of the Six Layers, (Deepa, 2012)

Layer	Land Use	Slope	Population	Soil	Cost	Technology
Weight %	26	26	26	8	9	5

In the GIS, each layer was ranked by using re-classification process. The weights of the parameters were used in the GIS model by using Weighted Overlying Analytical tool and the CSI was calculated by applying the formula below:

$$CSI = [Weights\ (AHP) \times Rank\ (GIS)] \qquad (2.12)$$

As a result the city was classified into three suitability levels: high potential 21.707%, moderate potential 30.89% and law potential 47.40%.

(Gemitzi, 2007) used GIS for siting areas for stabilization pond system (SP) to be used for treatment of wastewater of rural areas in 36 municipalities in Thrace (Northeast Greece), in which septic tanks were used to collect the sewage. The factors that considered in the selection methodology were; (1) environmental criteria, (2) land topography, slope of more than 5% was not taken , (3) land use which was classified into two types; non-forest areas, and grass areas while the remaining parts were dense forest areas which was rejected, (4) geological formation, the region was classified into aquifer and aquitard areas and the first class was excluded from the selection to prevent groundwater pollution, (5) distance from the SP units to the major rivers and lakes was equal to 500 m , (6) distance to the existing cities and villages was ≥ 500 m to keep pollutant away from residents, (7) temperature, (8) existence of environmentally protected areas, (9) population ,(10) the distance to existing roads and railways from the SP system was equal to 300 m, and (11) effluent characteristics. The factors mentioned above were applied into GIS software to analyze the variables. The results showed the suitable areas as Km^2 and as a percentage of total municipality area and it was illustrated in a raster map. In conclusion, this method was fast, simple and effective to find the specific locations for the SP units.

2.5 Optimization Models of Wastewater Treatment Systems

Many optimization models are applied widely in decentralized wastewater systems to find minimum costs and get highest benefits from the reclaimed treated wastewater reusing. (Naik, 2014) developed an optimization model using GA to find optimum design arrangements of DWWTUs in terms of optimum locations and number of treatment units. The parameters that been considered in the objective function were; construction and operation costs of the treatment units, construction cost of the collection and reclamation pipes and cost of water lifting .The method consisted of dividing a particular area into grids of 16 cells in which the DWWTUs were located and connected to sewer collecting pipes in addition to the reclaimed water network. The model consisted of a number of algorithms which were road network, DWWTUs cost, junction mapping, sewer link design, flow ratio, hydraulic iteration, minimum slope check and reclamation link design. The optimum solution was obtained when 8 DWWTUs were used .The details are shown in Fig. (2.1). In addition, the same work was done by using one centralized treatment unit and the results showed that the cost of the decentralized system was 1.5 million $ less than the centralized system, because of the long distances of the reclamation pipes of the centralized units as shown in Fig.(2.2).

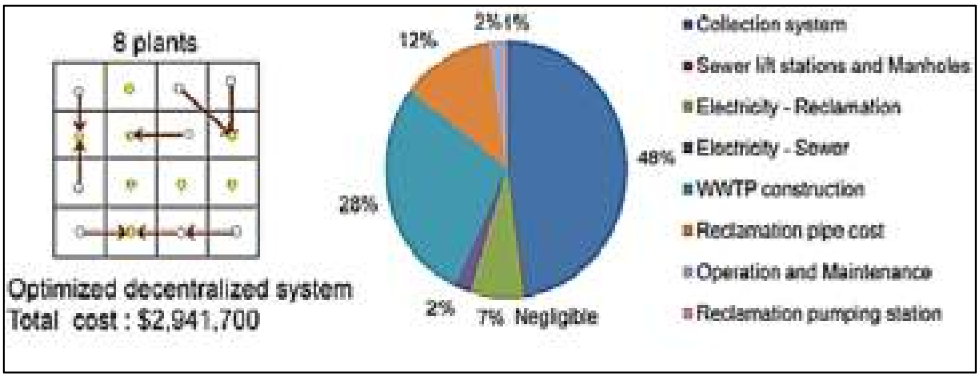

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Fig. (2.1): The Results of the 8 DWWTUs (Naik, , 2014)

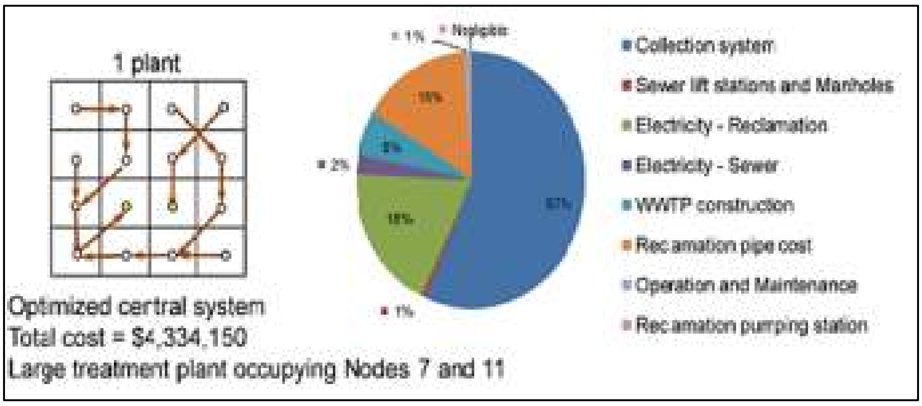

Fig.(2.2): The Results of One Centralized Wastewater Treatment Unit (Naik, , 2014)

(Hortua, 2009) presented a mathematical model to optimize direct recycle-reuse networks together with wastewater treatment processes. The model was used to minimize the annual cost of the system, which includes the cost of the treatment and piping using disjunctive programming formulation. The methodology of the work used a number of fresh water sources mixed with a number of wastewater sources both had a specified flow rate, composition and properties. The mixed flow will be discharged into a set of treatment units (centralized or decentralized) called sink points in which the wastewater will be treated and reused. The model was subjected to constraints related to environmental restrictions and amount of reused flow. A portion of wastewater will be treated while the remaining will be discharged into a stream waste and also a percentage of fresh water will be utilized in the method .The method was applied on a case study using two scenarios (A and B). In scenario A environmental constraints were not included while in scenario B the environmental constraints were considered.

(Brand, 2011) generated an optimization model using GA to minimize the capital and operation costs of regional wastewater treatment system. The model structure links the wastewater source, the pipeline network to convey the wastewater, the treatment units and the final disposal site. The algorithm search for the optimum pipe diameters, flow, number of treatment units and locations, the pump power and the required excavation works. Empirical equations were adopted to find each individual cost as shown below:

(1) Construction cost of pressurized pipelines

$$C_{PP} = 382.5\, D_p^{1.455}\, L \tag{2.1}$$

Where:

C_{PP} Pipeline construction cost, $

D_p Pipeline diameter (pumping line), cm

L Pipe length, Km

(2) Construction cost of gravity pipelines

Shallow excavation, for $H1 \leq 4$ m

$$C_2 = 21.6\, D_g^{2.26}\, L + 7\, \frac{H1^2 - C_{min}^2}{2(J - J_s)}\, L_w \tag{2.2}$$

Deep excavation, for $H1 > 4$ m

$$C_2 = 21.6\, D_g^{2.26}\, L + 7\, \frac{H1^2 - C_{min}^2}{2(J - J_s)}\, L_w + 10\, [L C_{min} + \frac{L^2}{2}\, (J - J_s) - \frac{H1^2 - C_{min}^2}{2(J - J_s)}\,]\, L_w \tag{2.3}$$

Where:

C_2 annual gravitational pipeline construction cost, \$/year
D_g pipeline diameter (gravitational pipe), cm
$H1$ least excavation cost depth, m
C_{min} minimum pipeline depth, m
L_w pipe excavation width, m
J, J_s gravitational required pipeline slope and soil slope respectively.

(3)Pump construction cost

$$C_3 = 64920\, P^{0.33} = 64920\, [3.454\, \Delta h\, Q + 6409\, (Q^{2.852}\, D_p^{-4.87}\, L)]^{0.33} \tag{2.4}$$

(4)Pump energy cost

$$C_4 = \frac{EC\, HR}{1000}\, [3.454\, \Delta h\, Q + 6409\, (Q^{2.852}\, D_p^{-4.87}\, L)] \tag{2.5}$$

Where:

C_3 Annual pumping construction cost, \$/year
C_4 Annual pump energy cost , \$/year
P Pump power, W
Δh Total head loss of the pipe, m
Q Flow of the pipe, m^3/hr
D_p Pipe diameter (pressurized pipe), cm
EC Energy cost, \$/Kw.hr
HR Number of annual operation hours, hr/year

(5) Treatment plant construction cost

$$C_5 = 85825\, Q_T^{0.71} + 1000\, Q_T \tag{2.6}$$

Where:

C_5 Annual treatment plant construction cost, \$/year
Q_T Treated flow, m^3/hr

The model applied on an example which consists of two cities connected through four optional gravitational and pumping pipelines to three possible treatment plants. The three treatment units are further connected to a central collection point through three gravitational pipelines. The optimum solution was found when using one treatment plant which receive the flow from city 1 through one pressurized pipe and city 2 through one gravitational pipes. The cost for the construction of the treatment unit was 83.3 % of the total cost.

(Rathnayake, 2012) studied the effects of the pollutant loads from combined sewer overflow (CSOs) on water bodies. They developed a multi – criteria optimization model to control the wastewater system from urban areas. The model consisted of two objective functions, the first one aimed to minimize the pollution load and the second function was to minimize the cost of the treatment plant. Calculations of pollution load to receiving water from the CSOs and a full hydraulic simulation was carried out. Effluent quality index (EQI) in Kg/day was formulated to evaluate the pollutant load on the received water body. Wastewater treatment cost was calculated using generic cost function and they took into account different amount of flow scenarios as shown below:

$$C = 916.862 \times (86400 \times V)^{0.659}, \quad (V \leq 3DWF) \tag{2.7a}$$

$$C = 916.862 \times (3DWF)^{0.659} + \frac{2}{3}(1.69(V - 3DWF) + 11376), \quad (6DWF \leq V \leq 3DWF) \tag{2.7b}$$

$$C = 916.862 \times (3DWF)^{0.659} + \frac{2}{3}((1.69 \times 3DWF) + 11376), \quad (V > 6DWF) \tag{2.7c}$$

Where:

C Total wastewater treatment plant cost (construction and M&O) £/year

V Treatment flow rate in m^3/sec

DWF Dry weather flow in m^3/sec

NSGA II was used to minimize EQI and C and the feasible solution was found with a mutation probability of 0.6.

(Velez, 2012) developed a multi objective optimization model used in the southern part of Cali City in Colombia to find the optimum design of an urban sewer network, activated sludge WWTP impact of the effluent on Lili River and minimum flood volume during WWF. The optimization of the sewer network can be considered as a multi –objective optimization in which the aim was to find the combination of pipe diameter, storage volume and pumping flow that minimize the flooding, the pollution impacts and the cost of the system. The formulation of the objective function of the optimum design of the sewer network was as in below:

$$MinF=\{f_{TFlood}, f_{Tpollution}, f_{TCost}\} \tag{2.8}$$

The cost objective function was equal to the cost of the sewer network and the cost of the storage estimated as shown in Eq. (2.9)

$$T_{Cost} = \sum_{i=1}^{Pipes} Cu_i L_i + \sum_{j=1}^{Assets} Cu_j A_j \tag{2.9}$$

Where:

Cu_iL_i : Total cost of the pipe network (excavation cost in €/m, cost of pipe supply in €/m and cost of manhole in €/unit).

Cu_jA_j : Total cost of storage, €/m

The selected Algorithm was NSGA II and the results showed that it was possible to optimize the sewer network design and reduce the cost on average up to 15% when compare with the pre-designed system, maintaining the same level of protection against flooding.

(Gillot, 2004) presented an optimization model to find objective economic index of the capital and operation costs of a WWTP. In this paper the cost equation was standardized to compare different treatment scenarios. The total cost of the WWTP was found by using present worth method. The cost model was applied on a design phase of an industrial WWTP, which consisted of activated sludge treatment system and biological nitrogen removal. WAST++ (Wastewater Treatment plant) simulator was used and two sets of maximum expecting loads rates were applied (Maximum 1 and 2). Two reactor sizes were determined and the investment cost of the larger one was 5% higher. The costs of the two alternatives were compared and the results showed that both reactor sizes reached the required effluent standards. The cost was increased when the flow increased for both sizes and the results showed that the cost will be less and more economical for larger plants especially for maximum flows.

(Chen R., 2009) developed a net benefit value (NBV) model to evaluate the cost – benefit of DWWTUs and reusing. Three main cost parameters were taken; construction cost of the treatment unit and the cost of piping C_1, cost of operation C_2, and cost of maintenance C_3. On the other hand, three main benefits were introduced in the model; water reuse B1, decreasing number of labors in case of using decentralized system instead of centralized units B2, and benefit to environment B3 . The net benefit value equation is shown below:

$$NBV = \sum B_i - \sum C_i \tag{2.10}$$

Where:

NBV: Net benefit value
B_i: Benefit value of item *i*
C_i: Cost value of item *i*

The model was applied on a case study of a DWWTP in a residential area in Xi'an, China. The wastewater were collected in two separate pipes, one for black water which was treated in a septic tank and the greywater was collected in the other pipe to be treated and reused for gardening, artificial pond refilling and other uses. Two scenarios were applied; Scenario 1 the reusing was for irrigation only and, Scenario 2 the reusing was for irrigation and for the replacement of the artificial pond. Moreover, two sets of cost – benefit evaluation were considered; one by ignoring the environmental benefits B3 and the second evaluation was by including B3. The results showed that when considering the environmental benefit B3 the NBV of scenario 2 was greater than scenario 1, also when B3 was not considered the NBV for scenario 1 was < 0 which means that the total cost was greater than the total benefit.

(Iqbal, 2009) carried out a mulita-objective optimization model for an operating WWTP using GA. The treatment plant that used was a typical completely mixed activated sludge model with EA system having influent flow rate equal to 1,500 m^3/day. The study adopted two optimization approaches; the first consisted of one objective function to calculate the kinetic parameters of the activated sludge. The second consisted of 5 optimization scenarios to enhance the operation of the treatment plant and it consisted of three objective functions which were; maximizing the influent amount, minimizing the effluent pollutants BOD and minimizing the operation cost of the plant. Flow from Jharkhand, India WWTP's data were used and a number of decision variables were taken. For the second optimization approach (operation optimization) the cost equation below was used by applying 5 scenarios of objective functions as follows:

$$OC = [C_{RSP}Q\Delta P] + [C_{SRP\cdot 1}Q_r^{2} + C_{SRP\cdot 2}Q_r + C_{SRP\cdot 3}] + [C_{AER.1}Q + C_{AER\cdot 2}] \quad (2.11)$$

Where:

OC:	Operation cost, \$/day
ΔP:	Discharge raw sewage pump pressure, m
Q_r:	Sludge recirculation rate, m^3/day
$C_{AER.1}$:	First cost coefficient associated with the sludge recirculation pump, \$
$C_{AER.2}$:	Second cost coefficient associated with the sludge recirculation pump, \$
C_{RSP}:	Cost coefficient associated with the raw sewage pump
$C_{RSP.1}$:	First cost coefficient associated with the mechanical aerators.
$C_{RSP.2}$:	Second cost coefficient associated with the mechanical aerators
$C_{RSP.3}$:	Third cost coefficient associated with the mechanical aerators

NSGA-II was applied to optimize the objective functions. Optimum values of kinetic parameters were obtained which had been used in the equations of the operation optimization costs.

(Ansari, 2017) used AHP using Expert Choice 11 software to select suitable locations of DWWTUs in Qom city in Iran. The criteria that been selected were :(1) population density, (2) slope, (3) land use, and (4) reuse, with regard to the environmental, economic, and social conditions of Qom. In addition, they sub- classed each criterion into further classes. Four suitable locations were found in Qom city to be used for DWWTUs.

(Engin, 2006) **c**reated a methodology to calculate the cost of wastewater treatment of small communities. They applied three scenarios for their work and they selected Gebza town and 22 surrounding villages in Turkey as a case study. The three scenarios were; Scenario 1, they used classical sewer and WWTP system in which the wastewater collected and conveyed to a big WWTP within a distance equal to 25 Km. Scenario 2, they adopted cluster system and used septic tanks for each house hold in the community and transfer the sewage to a large WWTP located within 25 Km. Scenario 3, they used individual package treatment system for each small community. The cost calculation for the three scenarios was based on the distances from those villages to the treatment plant and on the time life of the projects staring from 1 year to 25 years. The elements that considered for each scenario were the costs of; the sewer network construction, the treatment unit, the package treatment system and the operation and maintenance. The results of the first and second cases showed that total cost increasing with increasing time and distance. For case three, there was no effect of distance on the cost. The results also showed that the clustered system could be efficient if the distance will be equal or less than 7 Km and the operating time do not exceed 20 years. A second analysis was done by keeping the time constant and they only changed the distance from the main sewer to the treatment units. They found the distance that gave the lowest cost for each case.

(Dodane, 2012) made a study in Dakar, Senegal about their wastewater treatment and they calculated the cost of operation and construction of the two existing systems; parallel sewer base system SB (centralized system) and fecal sludge management system FSM (decentralized system). In the SB system the capital cost of all components were calculated such as; house hold connections, cost of the network system, pump station and treatment plant. The annual operation cost was taken from records. Moreover, some products released from the treatment process and had been considered in the cost calculations such as; the reclaimed water used for irrigation, the bio-solid used for soil conditioning and the methane gas captured to be used for energy. In the FSM system the capital cost of all components were calculated such as; costs of septic tanks and vacuum truck for transporting the produced sludge. The operation cost was for emptying the septic tank. Moreover, the benefits from reusing the materials that produced from the treatment process were considered in addition to the fees that paid by the householders. For both systems cost of the sludge processing was considered, which consisted of settling thickening tanks followed unplanted beds, with effluent going to a WWTP. The results showed that annual cost of the SB is much higher than the annual cost of the FSM.

(Hernandez-Sancho, 2011) developed cost models of different WWTPs using statistical data of 341 treatment units in Spain. The cost equations were as a function of the capacity of the plant expressed as the number of PE and per capita daily discharge of sewage. The formulation of the extended cost is as shown in Eq. (2.13).

$$C = A\ V^{b}\ e^{\Sigma(\alpha ixi)} \tag{2.13}$$

Where:

C : Total cost per year, €/m^3

b, α: Parameters,

A : Age of the plant , yr

V : Volume of wastewater treated per year , m^3

x_i : Different kinds of variables representative of the treatment process such as; age of the facility, the % of removal of the followings; SS, COD, BOD, N and P.

The model parameters were obtained by ordinary least squares regression analysis. Non – linear optimization model was used in GAMS software (General Algebraic Modeling System). Effluents from the treatment plants were very similar and were originally domestic wastewater. The WWTPs were classified into two main types; attached growth biological treatment and suspended growth biological treatment. Three technologies of attached growth types were used; (1) bacterial beds (BB), (2) pets beds (PB) and (3) biodisk beds (BD). For suspended growth type, also, three technologies were used; (1) EA (2) activated sludge without nutrient removal (AS) and (3) activated sludge with nutrient removal (NR). The results of the cost equations of the plants are shown in table (2.2).

Table (2.2): The Cost Function for Each Treatment Technologies,
(Hernandez-Sancho, 2011)

Technology	Cost Functions	R^2
EA	$C = 169.4844\ V^{0.4540}\ e^{(0.0009A+0.6086SS)}$	0.6133
AS	$C = 2.1165\ V^{0.7128}\ e^{(0.0174A+1.5122SS+0.0372BOD)}$	0.6849
NR	$C = 2.518\ V^{0.7153}\ e^{(0.0007A+1.455COD+0258N+0.243P)}$	0.7301
BB	$C = 17.3617\ V^{0.5771}\ e^{(0.1006A+0.6932COD)}$	0.9862
PB	$C = 1.51084\ V^{0.2596}\ e^{(0.0171SS)}$	0.5240
BD	$C = 28.9522\ V^{0.4493}\ e^{(2.3771SS)}$	0.8058

(Haghighi, 2012) created an optimization model to design sewer networks using Adaptive GA. Each chromosome consisted of the pipe hydraulic characteristics such as; pipe diameter (D), slope (S) and pump indicator (P). Hydraulic constraints are satisfied and the optimal design was to obtain the minimum cost. An existing network was taking as a case study to compare the results of the model. The construction cost of the sewer system was the objective function of the model which was minimized as shown below:

$$C(D,S,P) = \sum_{i=1}^{NP}(CP_i + P_i \,.\, CL_i) + \sum_{i=1}^{NP+1} CM_i \quad (2.14)$$

Where:

C: Cost function, $

CP: Construction cost of sewers (function of the D and pipe depth), $

CM: Construction cost of manholes (function of the D and pipe depth), $

CL: Construction cost of pump stations (function of sewer flow),$

Pi Pump location indicator

NP: Number of pipes

The case study was a network consisting of 79 pipes (with 24 different pipe diameter sizes) and 80 manholes in a residential area of a 260 ha. The GA population sizes were; 40, 80, 120, 160, 200 and 240. The results from using the *GA* model were more accurate and faster when comparing it with the existing sewer network system. Moreover, the study approved that the method is capable of solving large problems.

(Duarte Zeferino, 2011) applied an optimization model to determine the least – cost solution of the wastewater system of a region that has several population centers. The wastewater that produced from the community was discharged into a river. The objective function was to minimize the cost of installation and maintenance of the sewers and installation, operation (including energy), and maintenance of the treatment plants and pump stations. The objective function constraints were: (1) continuity constraints (inflow and outflow from the system and all nodes are in equilibrium), (2) the treated flow processed should not exceed the treatment unit capacity and the flow in the network should be within minimum and maximum allowable values, (3) environmental constraints (specify limit values for the parameters used to characterize river water quality), and (4) non-negativity and integrity constraints. The non – linear optimization model was solved by implementing a simulation annealing (SA) algorithm. The model was applied to three case studies each of them had same dimension (48.4 km × 28.0 km) and crossed by the same river with the same hydraulic and environmental characteristics. Each case had a different land elevation but they had the same population centers. Four Scenarios related to the constraint were applied. The results showed that the lowest cost was for the scenario of no constraint applied to the river water quality. The highest cost was for the case where the land was flatter than the other two case studies.

2.6 Wastewater Reusing for Irrigation

Municipal wastewater reclamation and reuse effectively provides ways to solve water resource problems in barren and semi-barren regions and irrigation is the major reuse for reclaimed water. With developing in technology, wastewater may be treated to meet the most restricted quality requirements and be used for any purpose willingness such as drinking water supply (Chen, 2013). There are a number of regulations that should be followed when the treated wastewater used for irrigation to protect the environment and human health. The major concerns of reclaimed water are the constituents remaining after treatment. These constituents are classified as conventional and nonconventional parameters and emerging constituents. The conventional parameters are pH, BOD, TSS, nitrogen, phosphorus, and organisms. The non-ordinary parameters are TDS, pesticides and refractory organics, surfactants, and metals (Qasim & Zhu, 2018).

(Hatami, 2018) assessed the wastewater quality produced from the EA wastewater treatment plant in Bojnoord city to be reused for agriculture and irrigation purposes. The parameters that measured were, EC, BOD, COD, TSS, VSS, TDS, SAR, and concentrations of sodium, magnesium, calcium, potassium and chloride. The results showed that the percentage of removal of BOD and COD are 88% and 89% respectively. The efficiency of removal of TSS and VSS were > 85 %. According to the results of it was concluded that the effluent is suitable for irrigation and agricultural purposes.

(Barbagallo, 2012) evaluated and analyzed the treated wastewater that produced from different wastewater treatment plants WWTPs in Sicily in Italy to be reused for irrigation. The maximum irrigation area in Sicily was 180,000 ha. The total number of WWTPs in the study area was 523 units, of which 259 were actually in operation, 89 not in operation, 32 were discarded, 47 were under construction and 96 were just planned by the public administration. GIS was integrated to locate the WWTPs in the study area with all information regarding the treatment units. Moreover, the characteristics of the treated wastewater, data about irrigation areas and the required irrigation volume were applied. The standards and restrictions of Italy's regulations and WHO for unrestricted irrigation water quality specifically for chemical compounds and microbiological parameters were considered in the research. A number of WWTPs were selected for effluent reusing based on the criteria related to; (1) the population equivalent PE (based on organic load) of each plant, (2) the elevation difference between the WWTP's location and the nearest irrigation district, and (3) the maximum distance from the plants to the irrigation districts based on the treated volume. The results showed that the total numbers of district irrigation areas were 24 out of 37 who were capable of receiving treated wastewater from 59 WWTPs. A quantitative microbial risk analysis was used for three WWTPs with different PE)to determine the numerical values of health risks. The study showed that the municipal treated wastewater could be used safely for irrigation of crops that eaten raw. The total amount of reusing water from the suitable WWTPs was 87 x 10^6 m^3/yr while the water deficit was 65 x 10^6 m^3/yr (water deficit = annual water required for irrigation – annual water released for irrigation).

(Afferden, 2010) prepared parameters for utilizing DWWTUs for reusing purposes in Lower Jordan Rift Valley area, which suffers from lack of water. The Jordanian Government was planning to utilize the treated wastewater for reusing. The authors forecasted the population of the cities that located in the study areas based on real data for population census from 1994 to 2004 and the population growth rate was considered uniform with a constant rate equal to 2.5%. Moreover, the study area was classified into two categories: (1) rural area for communities of population less than 5000 capita and (2) urban areas for communities that having population of more than 5000 capita. Data related to wastewater flow per capita was not available in Jordan. Therefore, the calculation of wastewater flow amount was based on the daily water demand per capita multiplied by a return factor of 0.825. The degree of connection of the flow produced from the community to the existing WWTP (13 treatment units) in the study areas were calculated from actual load of the treatment units and it was found that 75% of the urban area was connected with sewer network and only 5% of the rural area had sewer network. Therefore, the recommendation was to install DWWTUs in rural areas and the reclaimed water should meet the restrictions of Jordanian limitations.

(Adewumi, 2016) presented basic information about wastewater reusing and they showed many examples of reusing projects in 30 countries all over the world. They displayed the treatment level of each example and the reusing applications, which were mostly for irrigation, toilet flushing, industrial uses and for groundwater recharging. Moreover, treatment plant type was specified based on the reusing application and effluent required quality. The authors present the sanitation situation in Nigeria, which was very poor as there was only one industrial treatment plant in the northern part of the country. Most of Nigerian cities discharged untreated wastewater into water bodies, which were extremely polluted.

2.7 Wastewater Sludge

Sludge produced from wastewater treatment plants disturbs communities and it is a source of environmental contamination of the existing of various contaminations. Innovative and effective sludge treatment passages are fundamental for the clean and protected environment disposal (Abdul Raheem, 2017). Sludge handling and disposal includes collection, transporting, processing of the sludge to convert to a suitable form for disposal and final disposal of sludge. Moreover, the produced sludge could be reused in composting, energy recovery or even as a construction material. (Kelessidis, 2011) outlined the current situation and discussed future vision for sludge treatment and disposal in European Union (EU) countries based on available European Commission and Eurostat reports. The study showed that sludge management issued a big challenge in Europe. They mentioned that there are three main types of sludge treatment methods used in European countries; stabilization, conditioning and dewatering. The most common type of sludge treatment was sludge stabilization (aerobic and anaerobic digestion). Moreover, the common sludge disposal methods in EU were: agricultural uses for composting, incineration and landfills. In some EU countries it is not allowed to use landfills for their sludge disposal and they were forced to select between agricultural use and incineration. According to the research, it was expected that the percentage of bio-solids reuse in lands would reach 50 % by 2020 in EU. Based on reports it was realized that the percentage of landfilling decreased from 33% to 15 %, while incinerating sludge was increased from 11% to 21% and the reusing rate for agricultural utilization and composting increased by rate of 12.5%.

(Radaideh, 2010) collected a range of activated digested sludge samples from two different full scale municipal wastewater treatment plants in Jordan. One of the plants consisted of two EA tanks and the other plant consisted of a trickling filter followed by a conventional activated sludge processes. Moreover, two-lab scale digested sludge tanks were also used and samples were taken from there as well. One of the tanks used aerobic/anoxic digested EA and the other used anaerobic digestion. Comparisons were made between all samples (for the two full scale plants and for the two lab scale tanks) after 30 days of digestions. The following parameters were measured; (1) % of removal of volatile solid, (2) SVI and (3) CST. The results showed that the percentage of volatile removal of the aerobic digested for both lab scale and real plants were higher than the anaerobic digested sludge. The SVI and the CST were higher in the anaerobic digested sludge for both the real plant and the lab tank. The sludge drainability time through lab sand drying beds for both aerobic and anaerobic digested sludge were measured and the results for the EA digested sludge gave better results.

(Al-Muzaini, 2003) evaluated the performance of the sludge sand drying beds that used for dewatering the produced sludge of a wastewater treatment plant in Kuwait (Jahrah). The influent wastewater was from many sources such as; domestic, industrial sectors, petroleum stations, and car garages. The treatment plant had three large drying beds and each bed was divided into further 10 cells. The sand layer was 40 cm thick placed over 20 cm graded gravel. A network of pipes used to collect the percolated sludge through sand and gravel layers. The sludge dried in 9 days in summer and in 15 days in winter produces a cake of up to 40% solids. Samples were collected on monthly bases from the drying beds for a period of one year and bacteriological tests were done such as; total coliform, fecal coliform and salmonella. The results of the bacteriological test were very low and that indicated the effectiveness of the treatment to produce a good sludge quality. Moreover, the author also focused on another point related to the amount of produced daily sludge, which was 278 m^3/d, and that value was very high in compare to other plants all over the world and that was because of the hot weather of Kuwait.

(Radaidah, 2011) modified a sludge sand drying bed of Central Irbid Wastewater Treatment plant in Jordan by applying concentrated solar energy. The solar energy was used to heat water that was passed through a galvanized pipe network, which was installed at the bottom of the drying bed. The sludge in the modified drying bed was heated and samples were taken regularly from both modified and non-modified beds for a period of 18 months. The mean annual temperature of the atmosphere was taken to be 18 ˚C. Physical, chemical and biological analyses for both sample types were done. The results showed that when using the modified drying bed, the time required for dewatering was decreased by 60 %. Moreover, for the heated drying bed the microbiological contents of the sludge were decreased and for some pathogens 100 % removal were obtained. In addition, the results showed that pathogen content of the dried sludge of the heated drying bed had no risk on public health. In conclusion, the produced sludge from the modified drying bed had properties better than the conventional type in terms of pathogenic and organic content and that make it suitable to be used for land application practices.

2.8 Summary

This chapter of review of literature was performed to identify the main aspects related to DWWTUs during the last decades in terms of design, reusing the treated water for irrigations, selecting the best locations and finding optimum sizes using different optimization models. The review covers many prospects and the main findings were;

1. From the limited studies regarding the wastewater quality in Sulaimania City a basic idea was conducted. The available study covered some places in the city and it focused mainly on the discharge outlet points.
2. Different methods were adopted in using optimization methods for minimizing the cost of construction and operations of the treatment units, pumping and conveying pipes. The models that used were; GA, net benefit value (NBV) model, Multi objective optimization model, statistical model using data from existing treatment plants , and models using adaptive GA.
3. Using GIS and AHP with different suitability criterion related to social, economic and technical aspects. Moreover, many restriction layers were used to find the best locations of the DWWTUs.
4. The evaluation of treated wastewater, specialty from EA plants, for reusing was done in terms of the water quality for irrigation and that was by measuring parameters such as; , BOD, COD, TSS, VSS, TDS and SAR. It was found that the effluent was suitable for irrigation and agricultural purposes. Moreover, some researches focused on assessing the available amount of wastewater in order to get benefit from the reclaimed water quantities.
5. To evaluate the performance sludge drying beds that used for dewatering the produced sludge from WWTP, samples were taking on monthly bases for a period of one year and bacteriological tests were done and the results showed that the sludge had a good quality. While other researches focused on the design parameters and modified methods to enhance the dewatering value and pathogenic removals.

CHAPTER THREE

THEORETICAL CONCEPTS

Chapter Three
Theoretical Concepts

3.1 Introduction

This chapter describes the theoretical background of this research related to utilizing DWWTUs in a city and using the treated wastewater (reclaimed water) for irrigation. The first step in this research is to find the optimum locations of the DWWTUs inside the study area using Multi – Criteria Decision Model (MCDM). Moreover, based on the main objectives of this study, the cost equation of the DWWTUs and the conveying piping system costs of the reclaimed water that used for irrigation will be found. A transportation model is developed and GA in a matrix form is used to find the optimum amount of treated wastewater from each DWWTU to be reused for irrigation. The green areas could be irrigated from more than one DWWTU and the optimum solution will specify the source of water of each green area. Furthermore, GIS network analysis model is used to find the optimum pipe lengths and destination of the reclaimed water.

The selected DWWTUs type are extended aeration package plant (EA) which is recommended for small residential communities (Eddy, 2014, p. 1081). Drying beds are also used for the disposal of the digested sludge that produced from the DWWTUs. In the following sections the details of the optimization models, GIS models, package unit details and drying beds design are explained.

3.2 Selection of Suitable Locations of the DWWTUs Using GIS Models

Nowadays GIS technology is used widely in many environmental fields and it is one of the effective tools that used to deliver and support information to the environmental managers. GIS solution utilized to improve decision making, professional data analysis and interpretations, create analytical scripts for EIA studies. In addition, it increases productivity with streamlined work processes and pattern environmental incidents (Khandve, 2011, p. 244) . In this research GIS is used in organizing data, creating maps and developing models. MCDM for suitable land selection of the DWWTUs locations is used, the details are shown in the following paragraphs;

3.2.1 Multi-Criteria Decision Model using GIS

MCDM is concerned with forming and solving decision and forecasting problems involving multiple criteria. The aim is to help decision makers to solve problems. It is necessary to use decision maker's preferences to differentiate between solutions. The decision making process involves many steps; (1) showing the case, (2) criterion identification, (3) selection of the weight method like AHP and (4) show the method of accumulation which should be represented as a function (Majumder, 2015, p. 31).

MCDM is one of the methods that utilized to select the suitable locations of facilities like DWWTUs and sludge drying beds. The model's components consist of a set of suitability criteria related to environmental, social, hydrological and economical properties. Weighted Linear Combination (WLC) algorithm is used to find the land suitability index as shown in Eq. (3.1) used by (Sharma, 2012, p. 56):

$$S_{index}= \sum_{i=1}^{n}(Wi\,.\,Ci)\ \prod_{j=1}^{m}\ r_j \tag{3.1}$$

<u>Where</u>

S_{index} :	Land suitability index.
Wi:	Weight of the criteria
C_i :	Suitability of criteria
r_j:	The restrictions criteria
n, m:	Number of criteria and restrictions, respectively.

Eq.(3.1) is applied into ArcGIS software by creating three GIS models which are: (1) Suitability Model, (2) Restriction Model and (3) Suitability Classification Model of the land locations. Fig.(3.1) shows the flow diagram of the main steps of the process.

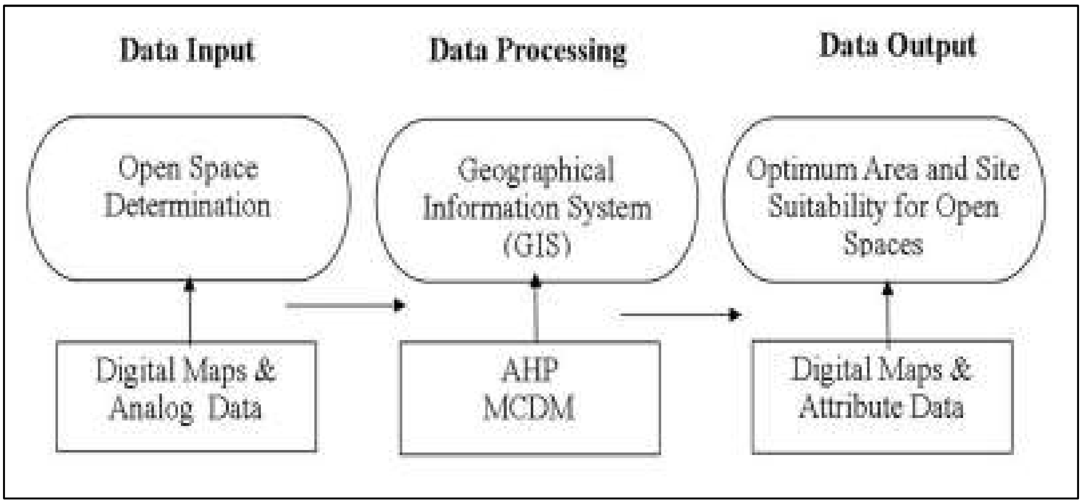

Fig.(3.1): Flow Diagram of the Main Steps of the Suitable Areas' Location Model in GIS, (Researcher)

3.2.2 Analytical Hierarchy Process (AHP)

GIS software is not capable of finding the weights (Wi) of the criteria; therefore, AHP is used which is one of multi criteria decision making methods that was originally developed by (Saaty, 2012, pp. (6 -8)). In this method each criterion is evaluated by using pairwise matrix **A** of size [*m x m*], where m is the number of selected criteria. Each element a_{jk} of the matrix represents the importance of the *j*th criterion relative to the *k*th criterion. If $a_{jk} > 1$, then the *j*th criterion is more important than the *k*th criterion, while if $a_{jk} < 1$, then the *j*th criterion is less important than the *k*th criterion. If two criteria have the same importance, then the entry a_{jk} is 1. The entries a_{jk} and a_{kj} satisfy that $a_{jk} \cdot a_{kj} = 1$, and the value of $a_{jj} = 1$ for all *j*. The relative preference between two criteria is measured according to a numerical scale from 1 to 9, as shown in Table (3.1).

The input can be obtained from real magnitude such as height, cost, or from individual judgment. After creating the matrix **A**, normalized pairwise comparison matrix $\mathbf{A}_{\mathrm{norm}}$ is derived by doing the sum of the entries on each column equal to one, Eq.(3.2) shows the process of calculating $\bar{a}_{jk}$ of the matrix $\mathbf{A}_{\mathrm{norm}}$:

$$\bar{a}_{jk} = \frac{a_{jk}}{\sum_{l=1}^{m} a_{lk}} \qquad (3.2)$$

Table (3.1): Scale for Pairwise Comparisons, (Saaty, 2012, p. 6)

Importance Value of a_{jk}	**Definition**
1	*j* and *k* are equally important
2	*j* is equally to moderately important than *k*
3	*j* is moderately important than *k*
4	*j* is moderately to strong important than *k*
5	*j* is strongly important than *k*
6	*j* is strongly to very strongly important than *k*
7	*j* is very strongly important than *k*
8	*j* is very to extremely important than *k*
9	*j* is extremely important than *k*

The criteria weight Wi is created by finding the average of the entries on each row of $\mathbf{A}_{norm}$ as shown in Eq.(3.3),

$$W_j = \frac{\sum_{l=1}^{m} \overline{a_{jl}}}{m} \tag{3.3}$$

Checking the Consistency – Consistency Ratio CR

The AHP includes an effective technique for checking the consistency of the evaluations made by the decision maker when building the pairwise comparison matrix **A**. The Consistency Index (CI) is obtained by first computing the scalar λ as the summation of $\bar{a}_{jk}$ multiplied by W_j of each criterion. CI is found from Eq.(3.4):

$$CI = \frac{(\lambda - m)}{(m - 1)} \tag{3.4}$$

CR is calculated as in Eq.(3.5), (Saaty, 2012, p. 9).;

$$CR = \frac{CI}{RI} \tag{3.5}$$

Where

n: Number of criteria
CI: Consistency Index
RI: Random Index value referred, table (3.2)
λ: Scaler Factor

Table (3.2) Random Index Value *RI* (Saaty, 2012, p. 9).

n	1	2	3	4	5	6	7	8	9	10
RI	0	0	0.58	0.9	1.12	1.24	1.32	1.41	1.45	1.49

The value of *CR* is an indicator shows the scales that has been allocated to each criterion weather it was a good judgment or not and it should be less than 10%.

3.3 Cost Optimization Model

The cost calculations of a wastewater system includes a number of elements related to the collection system and the treatment procedures. Moreover, there are other cost measurements that produced from the reusing process. The cost calculation of the treatment units includes the costs of investment (materials, labors, construction, installations and others), and operation and maintenance cost (energy, operation staff, materials, administration and others). There are factors that have effects on the mentioned elements as; the treatment capacity, location, whether it is centralized or decentralized, treatment methodology, the reusing purpose (which will specify the effluent quality and amount) and environmental restrictions. Regarding reusing and conveying the reclaimed water from the treatment units to the end users, it is also an essential part in the cost calculation. Many factors will influence the cost assessment of reusing such as; the reusing purpose, the amount of reclaimed water, and the destination points' locations. The wastewater collection sewer network is also one of the elements of the wastewater system, but in this research it will not be included in the cost optimization model as it is already existed in the study area.

In this study the main objective is to create a model to find the optimum capacity and best locations of each DWWTUs that gives the minimum cost and maximum benefit of water reusing for irrigation. The cost model will be for the treatment units and for the conveying of the reclaimed water as explained below:

3.3.1 The Treatment Plant Cost

The cost of the treatment plant includes the construction cost and cost of operation and maintenance (O&M). The cost equations should be created from real existing plants, which could be for the whole package treatment unit or, for each individual part of treatment units separately. The cost formulas are either as a function of unit capacity (treated flow) or as a function of population. As there are no treatment plants in the study area, formulas from other countries are used. Although the formula is for another region but the optimization process is a relative comparison of the costs of the treatment plants and the same equation is used for all the treatment units together. In conclusion, the results of the optimization will not be affected. The details of the objective function are shown in chapter four.

3.3.2 Cost of Conveying the Reclaimed Water

The treated wastewater is stored in a tank T_1 in the treatment plant area to control the flow fluctuation during different periods of flow (Viessman, 2009, p. 139). The reclaimed water will be conveyed via piping networks to the green areas to be used for irrigation. Two types of pipes are used: gravity pipes and pressurized pipes using pumps. The elevation differences between the location of tank T_1 and the green areas and the head loss value of the conveying pipes will specify whether gravity pipe or pressurized pipes will be used. The cost of conveying will include; (1) cost of the pipes, (2) cost of the pipe installation, (3) cost of the pump station construction, and (4) cost of O&M of the pump station. All those costs are functions of the conveyed flow (K. Swamee, 2008, p. 80). The amount of flow from each DWWTUs to each green area should be quantified in a manner that minimizes the total cost. The process of conveying the reclaimed water to the green areas is considered as a transportation problem; therefore, it is utilized for creating the relation between the treated flow and the demands of the green areas. GA in a matrix representation form is used to solve the model to compute the optimum amount of treated flow at each DWWTU. Matlab2018a software program code is applied to solve the GA.

a. The Transportation Optimization Model

It is an optimization method in which the objective is to minimize the cost of transporting a certain product from a number of origins to a number of destinations. In this research the amount of reclaimed water from the DWWTUs (origin) is transferred to a number of GR (destination).

This method was explained by (S. Rao, 2009, pp. (220 -222)) , and that is by assuming n origins (the DWWTUs) and m destinations (the green areas). Let a_i be the amount of supplied water from origin i ($i = 1, 2, \ldots, n$) and bj be the amount required at destination j ($j = 1, 2, \ldots, m$). Let f_{ij} be the cost per unit of transporting the reclaimed water from origin i to destination j. The objective is to determine the amount of water (Q_{ij}) transported from origin i to destination j such that the total transportation costs are minimized. This problem can be formulated mathematically as:

$$\text{Minimize } f = \sum_{i=1}^{n} \sum_{j=1}^{m} f_{ij} \quad (3.6)$$

Subjected to:

$$\sum_{i=1}^{n} Q_{ij} = b_j, \qquad j = 1,2,\ldots\ldots,m \quad (3.7)$$

$$\sum_{j=1}^{m} Q_{ij} \leq a_i, \qquad i = 1,2,\ldots\ldots,n \quad (3.8)$$

$$Q_{ij} \geq 0, \quad i=1,2,\ldots\ldots,n, \quad j=1,2,\ldots m \quad (3.9)$$

The transportation problem have (n x m) variables and ($n+m$) constraints. Eq.(3.7) shows that the total amount of the water transported from the all origins i to destination j must be equal to the amount required at destination j ($j = 1, 2, \ldots, m$). Eq. (3.8) shows that the total amount of the water received from origin i to all destination j must be ≤ to the amount available at the origin i ($i = 1, 2 \ldots n$). Eq. (3.9) added the non-negativity since negative values for any Qij have no meaning. It is assumed that the total demand equals the total supply, that is,

$$\sum_{i=1}^{n} a_i = \sum_{j=1}^{m} b_j \quad (3.10)$$

Eq.(3.10), is called the *consistency condition* and must be satisfied if a solution is to exist. This can be seen easily since

$$\sum_{i=1}^{n} a_i = \sum_{i=1}^{n} \left(\sum_{j=1}^{m} Q_{ij}\right) = \sum_{j=1}^{m} \left(\sum_{i=1}^{n} Q_{ij}\right) = \sum_{j=1}^{m} b_j \quad (3.11)$$

The transportation matrix can be represented as shown in Fig.(3.2)

To / From		Destination j					Amount Available a_i
		1	*2*	*3*	…	m	
Origin i	*1*	Q_{11} f_{11}	Q_{12} f_{12}	Q_{13} f_{13}	…	Q_{1m} f_{1m}	a_1
	2	Q_{21} f_{21}	Q_{22} f_{22}	Q_{23} f_{23}	…	Q_{2m} f_{2m}	a_2
	3	Q_{31} f_{31}	Q_{32} f_{32}	Q_{33} f_{33}	…	Q_{3m} f_{3m}	a_3
	⋮	⋮	⋮	⋮	⋮	⋮	⋮
	n	Q_{n1} f_{n1}	Q_{n2} f_{n2}	Q_{n3} f_{n3}	…	Q_{nm} f_{nm}	a_n
Amount Required b_j		b_1	b_2	b_3	…	m	

Fig. (3.2): The Transportation Array, (S. Rao, 2009, p. 222)

b. Theory of GA Optimization Technique

It is a simple method applied for complex problems and it is one of the nontraditional stochastic optimization methods used to solve nonlinear objective functions. GA is applied in this research to solve the objective cost problem because of the big and complicated data. Moreover, GA is widely applied into wastewater and pipe networking problems. In this study a matrix representation of the GA is used which can display data structure of the elements so they can have better relations to surrounding locations dataset (Chen, 2017, p. 2). Using a matrix form of GA in the optimization of DWWTU and pipe network costs is a genuine work and there is no previous researches about that. GA is based on the principles of natural genetics selection and the method adopts random selections from a population guesses. Continuous GA is used as there is no need for accuracy in the variable values also because of the big amount of data, which make it difficult to use binary GA. The components of the GA are explained by (Sastry, 2006, pp. (97 -99)) as in below:

i. **Initialization**: The process starts with random generation of a number of solutions. Each solution represents a chromosome, with the variables as genes. The initial population size is *Np*, which is also the number of chromosomes.
ii. **Evaluate The Fitness**: It is applying the random generated chromosomes (parents) in the fitness function (objective function).
iii. **Selection**: Is to select the best solution among the worst and that could be done using many methods such as roulette-wheel selection, stochastic universal selection, ranking selection, tournament selection and the whole parents could be selected for mating. The results will arranged in descending or ascending order based on the type of the optimization problem. For maximization problem, descending order is used. Ascending order is used for minimization problem.
iv. **Crossovering:** The crossover process is to produce offspring as a new solution population from the parent populations. In this step, parts of two or more parental solutions are combined to create new, possibly better solutions (i.e. offspring). There are many ways of accomplishing this, and the best solution depends on a properly designed recombination mechanism. The offspring under crossovering will not be identical to any particular parent and will instead combine parental traits in a different manner.
v. **Evaluation**: The crossovering process is done for the Np initial parent population then they will produce an Np offsprings. These two populations (parent and offsprings) are mixed together and (*2 x Np*) solutions will be produced. The (*2 x Np*) solutions will applied into the objective function to find the fitness values which is the cost *F*.
vi. **Iterations:** The crossovering process could be repeated many times using the obtained population instead of the randomly generated one. The process of repeating is called iteration and the number of iteration is selected and it could be 1, 2, 3 …etc.
vii. **Mutation:** This process is done after the crossovering and iterations are finished where at the end of the last iteration the best three solutions (optimum) are the first, second and third. In this stage, a process called mutation is to be done by selecting some solution variables and start to increase or decrease their values and check if this will enhance the obtained optimum solution by the iterations of the crossovering process.

In this study the initiated population is presented in a matrix of size equal to [*n x m; Np*] and each solution represents a chromosome with variables equal to genes. The random values are generated by *rand (n x m ;Np)*. The chromosome will be in a matrix form and as a function of *Q* :

$$Chromosome = \begin{bmatrix} Q_{11}, Q_{12}, Q_{13}, \ldots Q_{1m} \\ Q_{21}, Q_{22}, Q_{23}, \ldots Q_{2m} \\ Q_{31}, Q_{32}, Q_{33}, \ldots Q_{3m} \\ . \\ . \\ Q_{n1}, Q_{n2}, Q_{n3}, \ldots Q_{nm} \end{bmatrix}$$

Where *n* is the number of the *(*DWWTUs*)* and *m* is number of green areas *GRs* that will be irrigated. The process is applied to find the optimum amount of treated wastewater delivered to each green area that gives the minimum cost *F*.

$$Cost = f\,(chromosome) = f \begin{bmatrix} Q_{11}, Q_{12}, Q_{13}, \ldots Q_{1m} \\ Q_{21}, Q_{22}, Q_{23}, \ldots Q_{2m} \\ Q_{31}, Q_{32}, Q_{33}, \ldots Q_{3m} \\ . \\ . \\ Q_{n1}, Q_{n2}, Q_{n3}, \ldots Q_{nm} \end{bmatrix}$$

There are many types of crossovering process in matrix form GA such as, block crossovering, self-crossovering, row crossovering, two point crossovering and others. In this study the crossovering process is done for columns and with one point of crossover (*PCO*) which is specified as shown in Fig.(3.3); Crossovering between the two parents will occur and the variables will exchange to produce offspring1 and offspring 2.

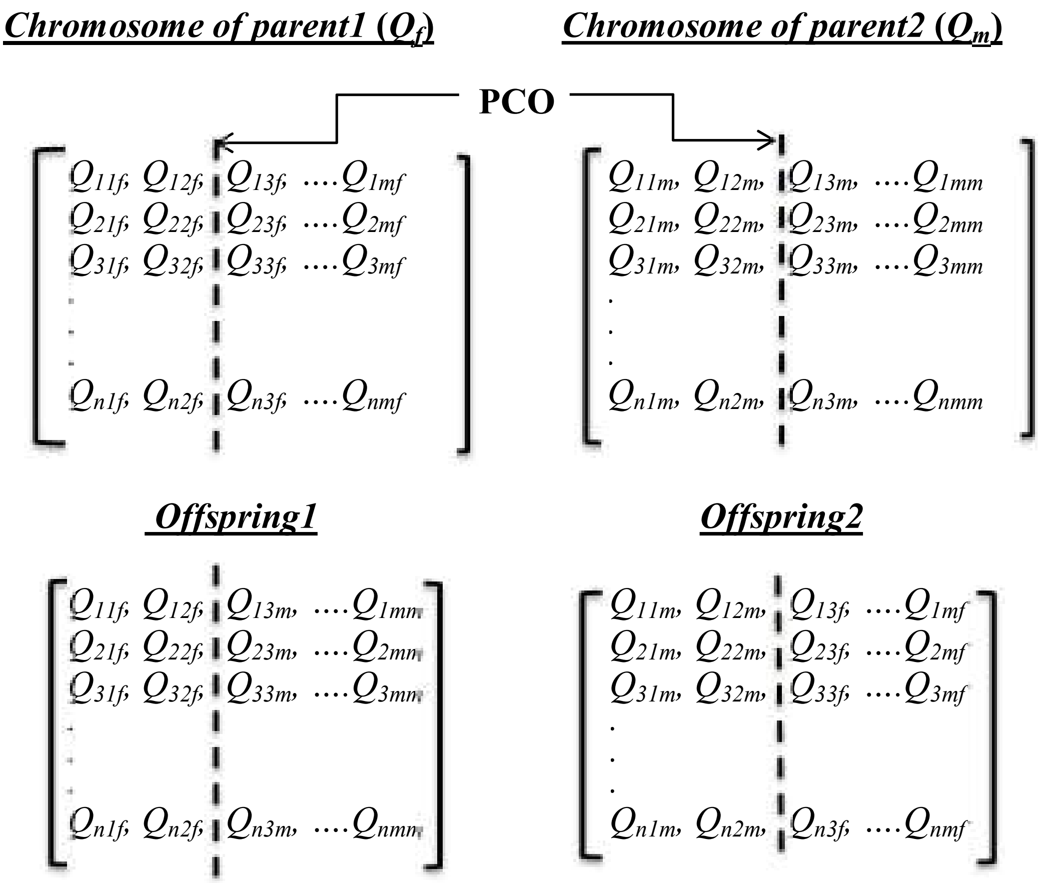

Fig. (3.3) : The Crossovering Process between the Two Parents, (Researcher)

c. GIS Network Analysis – OD Cost Matrix

Network Analysis - OD Cost Matrix method in GIS is a model used to measures the least-cost paths along a network (the drive time and drive distance) from multiple origins to multiple destinations. This technique was used for many transportation researches to find the optimum cost of reaching to the closest certain facility. In this research, it is the first time to use this tool to find the optimum water network routes and lengths of the pipes that connect the DWWTUs and the GRs. The origins are the centroids of the nominated areas of the DWWTUs and the destinations are the centroids of the green areas. The steps in this method are illustrated in Fig. (3.4). The road layer of the study area is used as a best route network in the process and the pipe network layout is considered to follow the same path of the road. The output shape type is a set of straight lines. Even though the OD - cost matrix solver does not output lines that follow the network, the values stored in the lines attribute table reflect the network distance, not the straight-line distance. This method is fast in solving large data space more than the other types of GIS network analysis processes and that will save computation time (ESRI, 2013).

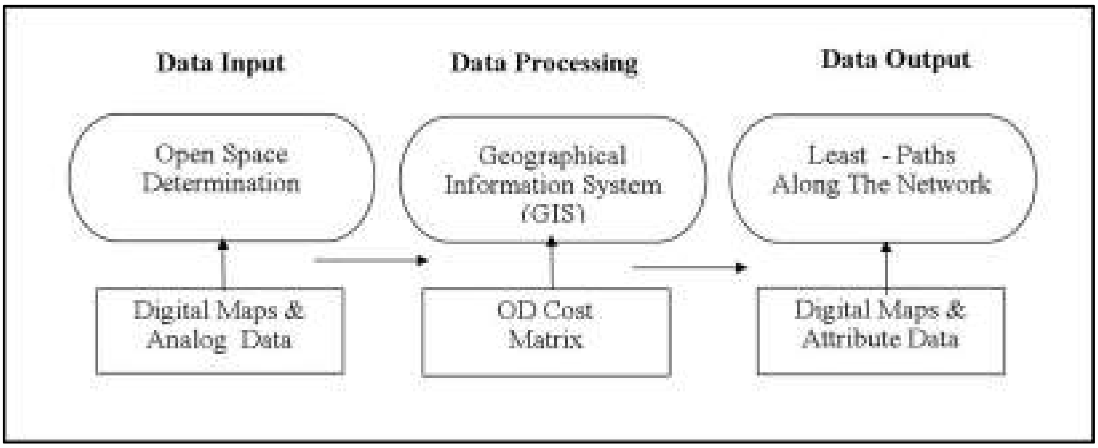

Fig. (3.4) : Flow Diagram of the Main Steps of the OD - Cost Matrix in GIS, (Researcher)

d. Elevation Difference between the DWWTUs and the GRs Using GIS

The calculation of the elevation difference between the DWWTUs locations and the GRs are important to specify the pipe network type if it is gravity pipe or pressure pipe. Elevations of the study area are found using Digital Terrain Model (DTM) map in GIS. The elevations of the centroid points from each DWWTUs locations and green area are found using Point Extraction Tool (Yuji, 2011, p. 6).

3.4 Decentralized Treatment System

DWWTUs units could be defined as small treatment units that installed close to the sewage generation areas. The treated sewages could be reused for many purposes like irrigation, groundwater recharging, firefighting and others. There are many sizes and methods of treatments, which depend on the amount of flow, effluent quality, reusing purposes, and it depends on the location of the treatment units. Prefabricated plants (package plants) are one of the technologies that used to treat wastewater from small communities with flow amount ranged from (38.50 – 3800 m^3/day) (Eddy, 2014, p. 1080). Sewage treatment package plants are cost effective, have good treatment employments, are built -in, require small footprints, easy to install and are highly docility to environment.

There most common types of wastewater treatment package plants are; extended aeration plants, sequencing batch reactor, oxidation ditches, contact stabilization plants, rotating biological contactor and physical/chemical treatment (Eddy, 2014, p. 1082). In this research EA is used as this type is mainly utilized for wastewater treatment of residential and small communities. EA treatment process has excellent effluent quality, produces relatively low sludge amount, not complex and it has a simple operation process (Eddy, 2014, p. 1081).

3.4.1 Extended Aeration Treatment Method

Extended aeration method is a modified activated sludge process used to remove biodegradable organic wastes under aerobic condition. In this study extended aeration package plant is used and it consists of the followings; (1) pretreatment units such as screens and grinders, (2) flow equalization basin, (3) aeration tank,(4) secondary clarifier, (5) disinfection tank, (6) storage tank for the reclaimed water, (7) pumping station, and (8) aerobic digester. The tank could be installed underground but the tank walls should extend 0.15 m above the ground to prevent surface runoff to inter the plant (EPA, 2000, p. 1) . The details are explained below:

1. **Pretreatment units**: Bar screens and commutators are usually installed at the entrance of the treatment plant to get rid of all solid wastes such as; silts, sand grains, leaves, seeds and other materials that exist in the sewage which cannot be spoiled.

2. **Flow Equalization Basin:** It is a flow variation controlling tank and it is used to control and regulate the flow during peak periods which located at aeration tank influent. The process comprises providing storage capacity and adequate aeration and mixing duration to prevent odors and waste settlements. The required capacity for flow equalization is found by using an inflow mass diagram and a detailed data of hourly flow amount for the city is required (Qasim, 1985, p. 38).

3. **The Aeration Tank:** At this stage, the biological treatment is occurred in which the flow is completely mixed with oxygen that is supplied mechanically or by air diffusors. The microorganisms will be supplied by oxygen and will feed on the organic matter in the sewage. The wastewater in the aeration tank is called mixed liquor suspended solids

(*MLSS)*. The capacity of the aeration chamber should be enough to provide aeration for a retention time equal to 24 hr during the average flow and *BOD* loading of *0.1* Ib BOD_5 /Ib *MLVSS*. The required air in m^3/day is calculated from Eq. (3.12), (Eddy, 2014, p. 1088):

$$\text{Air required } (m^3/d) = \frac{\text{Peak daily BOD (Kg/d)}}{O_{teff}\ \% \ x\ \rho_a \ x\ O_2\%}, \qquad (3.12)$$

Where:

O_{teff}: oxygen Transfers Efficiency %

ρ_a: specific gravity of air = 1.21 Kg/m^3

$O_2\%$: oxygen content in air %

Peak daily *BOD* (Kg/d) = [No. of capita x 2.5 x Kg *BOD*/Capita. Day]

4. **Secondary Clarification Tank**: It is an essential part of the activated sludge process and it follows the aeration tank. In this basin a large amount of the *MLSS* that comes from the aeration tank will be separated. Part of the mixed liquor will be returned to the aeration tank (Q_R) through a sludge return pipe. The effluent Q_{eff} has a low concentration of *BOD* and suspended solid (*SS*) which comply with allowable environmental limits.

5. **Disinfection:** The treated wastewater is then disinfected with chlorine in the chlorination chamber, and the chlorine is removed by dechlorinating unit. The detention time should be at least 30 min at peak flow with a typical dose of 25 mg/L**.**

6. **Storage Tank for the Treated Water (T_1) :** A storage basin is also used to collect the reclaimed water that will be delivered to the green areas.

7. **Pumping Station**: Pumps are used to convey the reclaimed water to the green areas whenever required with different capacities and heads.

8. **Aerobic Digester:** It is used to treat the sludge that produced from the extended aeration plants of sizes less than (0.2 m^3/s) (Eddy, 2014, p. 835). The details are explained in paragraph 3.5.3.

The design limitations and criteria of the package plant extended aeration activated sludge process are shown in Table (3.3). Fig.(3.5) and Fig.(3.6) show the typical details of an extended aeration package plant.

Table (3.3) : Typical Design Limits of Extended Aeration Package Plant
(Eddy, 2014, p. 1084)

Design Parameter	**Value**	
	Range	**Typical**
Pretreatment - Bar Screen		
Aeration Tank		
Retention time (aeration tank) , hr	18 - 36	24
BOD_5 loading ,Kg BOD_5 /kg MLVSS	0.05 - 0.15	0.10
MLSS (aeration tank) , mg/L	2,500 – 6,000	3,500
Sludge Age, θ_c , day	20 - 30	25
Oxygen Required		
Average at 20 ˚C , Kg/Kg BOD_5 applied	2 - 3	2.5
Peak at 20 ˚C , (value) x (av. flow)	1.25 – 2.0	1.5
Oxygen Transfers Efficiency		6%
Secondary Clarifier		
Settling tank overflow rate Based on peak hourly flow , m^3 /m^2.day	24 - 40	33
Waste Sludge		
Dry Solid , Kg / 10^3 gal	0.32 -0.45	0.36
Excess Sludge , Kg/Kg BOD_5 removed	0.3 – 0.75	0.4
Specific gravity of sludge solids		1.30
Specific gravity of Sludge		1.015
Chlorination		
Dosage at peak flow, mg/L	15 - 40	25
Detention time at peak flow , min	15 - 45	30

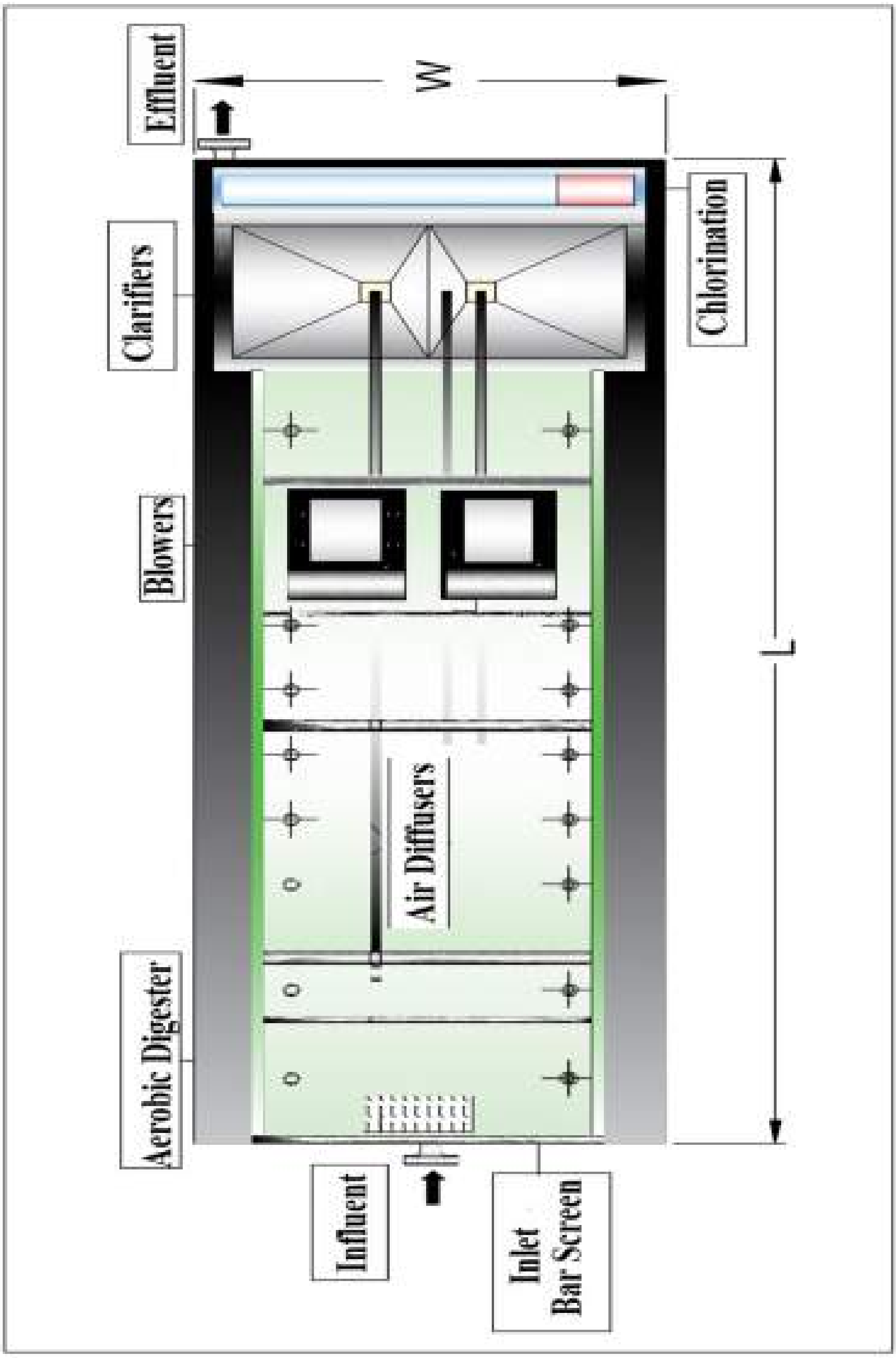

Fig.(3.5):Typical Extended Aeration package plant – Plan View, (Researcher)

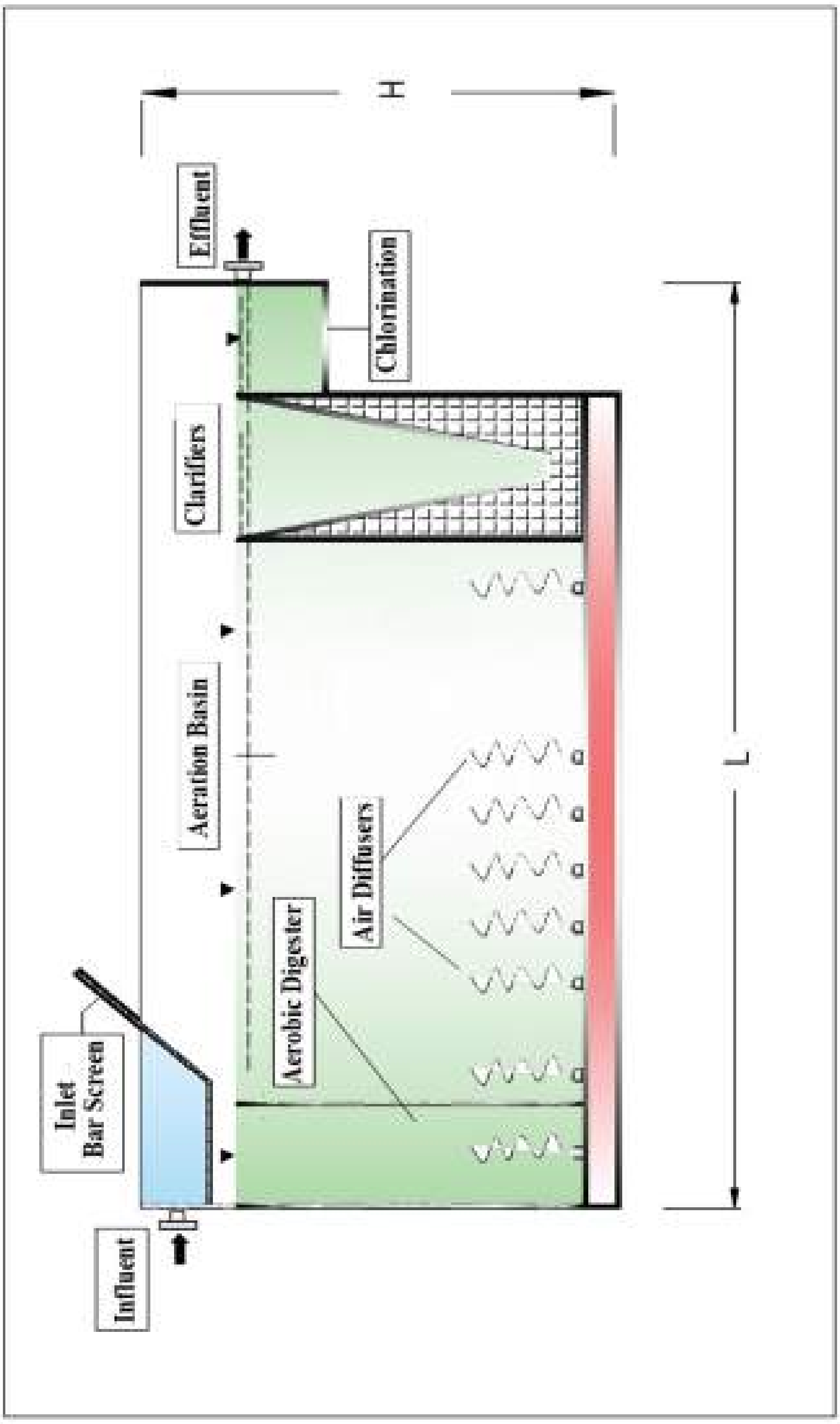

Fig.(3.6):Typical Extended Aeration package plant – Side View, (Researcher)

3.4.2 Wastewater Flow Calculation

Wastewater in cities produced from many sources such as; domestic, commercial, public, industrial activities and from groundwater infiltrations (J.McGhee, 1999, p. 7). Sewer pipes are used to collect the sewage to be conveyed to treatment facilities to get clean water with no pollutants. Specifying the amount of produced wastewater is essential in the design of the piping network, pumping system and the treatment plant units. The amount of discharged wastewater is calculated either practically at site using specific devices or from some theoretical methods. The theoretical method to calculate the amount of wastewater flow for each individual source is shown in the following sections:

1. **Domestics Wastewater Flow:** The main sources of domestic wastewater in a city are from residential areas, commercial district, institutional facilities and recreational areas.

(a) **Residential Buildings:** Residential buildings in a city are individual houses, apartments, hotels and motels. The amount of wastewater flow is commonly determined on the base of population density and the average per capita flow values. The amount of wastewater flow from the residential areas could be estimated from the water supply consumption per capita per day as shown in the equation below. (Eddy, 2014, p. 186) :

$$Q_{av} = Q_{avw} \ x \ R \ x \ Capita \tag{3.13}$$

Where:

Q_{av} : average wastewater flow per day, in m^3/day

Q_{avw} : average water supply flow per capita per day, m^3/cap. day

R: percentage of municipal water supply discharged into the collection system as wastewater and it is usually from 60 – 85 % (Eddy, 2014, p. 187)

Capita: number of populations

(b) **Commercial Districts:** Commercial buildings in a city includes many shops, handicrafts, business buildings, and malls. The wastewater flow for commercial areas is measured in m^3/ha.day. Average flow volume per day for commercial area rang from (7.5 to 14 m^3/ha.day) (Eddy, 2014, p. 187).

(c) **Institutional Facilities:** Wastewater flow from institutional buildings changed according to the area and structure type, the following are examples of institution building; hospitals, schools, universities, jails, and others (Eddy, 2014, p. 187).

(d) **Recreational facilities:** The amount of flow from recreational facilities changed within seasons and such as; swimming pools, cafeterias, resort, hotels, clubs, restaurants, etc. The amount of flow is measured in m^3/unit.day.

2. **Industrial:** Effluents from industrial facilities changed according to its type and size, the water reuse phase and the wastewater treatment methods. The produced wastewater volume is measured in m^3/ha. day. Another method for estimating the amount of produced wastewater, is by multiplying the amount of used water with 85 – 95 % of (Eddy, 2014, p. 187).

3. **Infiltration/Inflow:** It is defined as the water that entered into the sewerage network through the cricks in connections, pipe joints, and manhole walls. There are many types of inflow such as; groundwater, from building drainages, seepage from springs and wetlands. Calculating the amount of groundwater inflow relied on lengths and diameter of the sewer pipe (m^3/day. mm – Km) other methods depends on the amount of served area (m^3/ha. day). The volume of inflow could range from ignored amounts to obviously highly quantities and that will depend on many factors like the groundwater altitude, the climate, the soil permeability, the season and other factors.

4. **Wet Weather Flow (WWF):** Storm water is collected through street inlets to be conveyed by separate or combined sewer networks. In separate network usually the storm water is discharged into water bodies or open areas, while in combined sewer system it will be transported to wastewater treatment plant. In combined system flow during wet weather will affect the design of the DWWTUs in terms of the quality and amount of influent.

3.5 Sludge Amount Calculation and Treatment

Sludge treatment is one of the complex issues that face engineers and that refers to its large volume in compare to the other removed constituent during treatment. Moreover, sludge contains substances that are very annoying to people especially in case of DWWTUs which installed close to residential areas (Eddy, 2014, p. 765).

In this study the sludge is produced from the final clarifier and is digested in an aerobic digester. The digested sludge is stored in a holding tank and it is transported by trucks to the drying bed to be reused for composting. It is essential to calculate the amount of the produced sludge to specify the size and location of the sludge drying bed. The quantity of sludge that produced could be measured in Kg/day or in m^3/day (Andreoli, 2007, p. 55).

3.5.1 Calculation of the Generated Sludge Q_w

The generated sludge Q_w is the produced from the clarifier and it is separated to be conveyed to the aerobic digester. In this study the amounts of the sludge are calculated by using the mean cell - resident time θ_c equation as shown below (Viessman, 2009, p. 585) :

$$\theta_c = \frac{VX}{Q_w\ X + (Q_{in} - Q_w)X_e} \tag{3.14}$$

$$t = \frac{V}{Q_{in}} \longrightarrow V = t\ x\ Q_{in} \tag{3.15}$$

Where:

V	Volume of reactor, m^3
X	Concentration of biomass in aeration tank (MLVSS), mg/L
Xe	Concentration of biomass in effluent , mg/L
Q_w	Rate of excess sludge (wasted sludge), m^3/day
Q_{in}	Influent flowrate of the treatment plant, m^3/day
$Q_{e.}$	Rate of effluent flow, m^3/day
t	Mean hydraulic retention time for the reactor, hr
θ_c	Mean cell - resident time, day

Values of θ_c , X , t [table (3.3)] and Xe [will be assumed] are applied into Eqs. (3.14) and (3.15) to find a relation between Q_w and Q_{in} to calculate the rate of produced sludge as a function of the treated flow in the package. The details of the results of the calculation are shown in chapter 4.

3.5.2 The Sludge Treatment Methods: Many methods could be selected for the treatment, which depends on the sludge amount, the sludge type and on the reusing purpose. The methods that used for sludge treatment are; sludge thickening, dewatering, and digestion (aerobic and anaerobic). In this study aerobic digestion is used.

3.5.3 Aerobic Digestion: It is a biological process that occurs in the presence of oxygen and it could be used for treating: (1) waste activated sludge, (2) mixtures of waste activated sludge or trickling filter and primary sludge, (3) waste sludge from extended aeration plants, or (4) activated – sludge treatment plants designed without primary settling. Mainly aerobic digestions is used in plants of size less than 18,925 m^3/d and in recent years it is utilized for larger treatment units. The advantages of aerobic digestion are; (1) BOD concentration is within the allowable limit, (2) the produced sludge is odorless and stable, (3) the operation is not intricate, (4) affordable capital cost. In spite of the mentioned advantages there are some disadvantages such as high operation cost and the process affected by temperature therefore it is required to be covered, (Eddy, 2014, p. 835). Table (3.4) shows the design criteria for aerobic digesters.

Table (3.4): The Design Criteria for Aerobic Digesters,
(Eddy, 2014, p. 837)

Parameter	**Value**
Hydraulic retention time, at about 20 C° , day	12 - 18
Solid Loading, Kg volatile solids/m^3.day	0.16 – 0.48
Oxygen requirements, KgO_2/Kg solids destroyed cell tissues	1.045
Energy requirements for mixing Mechanical aerators hp/10^3 ft^3 Diffused – air mixing, $m^3/10^3$ m^3 . min	 27 – 53 20 - 40
Dissolved – oxygen residual in liquid, mg/L	1 - 2
Reduction in volatile suspended solids %	40 - 50

(1)The Tank Volume: The digester tank volume can be calculated by applying Eq.(3.16) (Eddy, 2014, p. 841), as shown below:

$$V_d = \frac{Q_W \ X_i}{X\,(K_d\,P_v + 1/\theta_c)} \tag{3.16}$$

Where:

V_d	Volume of aerobic digester, m^3
Q_w	The digester influent average flowrate, m^3/d
X_i	Influent suspended solids concentration, mg/L
θ_c	Solid retention time , day
X	Digester suspended solids concentration, mg/L
K_d	Reaction rate constant, d^{-1}
P_v	Volatile fraction of digester suspended solids

(2)The Oxygen and Energy Requirements for Mixing: The required oxygen (Kg of O_2) of the aerobic digestion is measured based on the Kg of complete oxidation of destroyed cell tissues as shown in table (3.4). To achieve the required oxygen amount proper agitation should be provided and mixing power requirements should be checked as shown in table (3.4). The required oxygen Kg O_2/ day are calculated as in equation (3.17):

Kg O_2/ day = Total mass of volatile solid (VSS) x oxygen required (Kg O_2/Kg destroyed cell tissues), (from table 3.4)

Kg O_2/ day = VSS x 1.045 Kg O_2/Kg cell tissue destroyed (3.17)

3.5.4 The Sludge Storage: Long-term storage may be accomplished in sludge stabilization process with long detention period such as aerobic digestion or in a separate tank. In small treatment units, usually the sludge is stored in the settling tank or in the digester.

3.5.5 Drying Bed: It is a natural drying process in which dewatering is occurred by losing water to the atmosphere through evaporation and filtration through the filter media and the drain pipes at the base of the beds (Ifeanyi, 2008, p. 6)**.** The produced sludge is usually dumped of in landfills or it reused in composting and soil conditioning. This method is recommended because of its low cost, it does not need a regular responsiveness and the solid content is high in the dried sludge. The factors that considered in the design of drying beds are; (1)weather conditions, (2)sludge properties, (3)land values and availability, and (4) closeness of residential areas. In this research, conventional sand drying bed is adopted as it is used most commonly. This type of drying bed is restricted to digested sludge. Fig.(3.7) and Fig.(3.8) show the typical details of sludge sand drying bed which consists of the following details**:**

(1) The Sand Layer: Is placed on the top of the drying bed in which the sludge from the truck will be placed over it. The depth of the sand layer is (230 – 300) mm. The sand has an effective size of (0.3–0.75) mm and a uniformity coefficient of less than 4, (Eddy, 2014, p. 871).
(2) The Gravel Layer: The graded gravel or stone layer is used to support the sand layer and it has a depth of (20 – 46) cm. It is placed under the sand layer and over the underdrain pipes, (Techobanoglous, 1998, p. 959).
(3) The Underdrain Pipes: There are underdrain pipes that used to collect the drained water, their diameters are not less than100 mm size, are placed in a distance from (2.4 – 6.0) m, and have a minimum slope of 1 %, (Eddy, 2014, p. 871).
(4) The Drying Bed Area: The Area is divided into smaller beds with dimensions of (4.5 – 18) m wide and (15 – 47) m length. The sludge is added on the bed in many layers of (20 - 30) cm thickness per each. This type could be covered and that will be preferred to protect the sludge from weather changes. The sludge drying time is important and it is affected by the initial concentration of the solids in the sludge and on the depth of discharged sludge over the sand (Shammas, 2007, p. 404). The focus will be on sizing the drying beds, which is based on the amount of transferred sludge from the DWWTUs.

(5) Sizing the Drying Beds and Land Requirements: Sizing of drying beds is a function of the sludge type, solid content and the sludge volume. For optimum drying bed size the sludge loading rate is ranged from (100 – 300) Kg dry solid /m^2.year (uncovered beds) and from (150 – 400) Kg dry solid /m^2.yea (for covered beds). The recommended uncovered and covered sand drying bed's areas are calculated in Eq. (3.18) and Eq. (3.19) respectively, (Qasim, 1985, p. 295);

$$A= (0.14 - 0.28)\ m^2/\text{capita} \times \text{No. of capita (Uncovered Beds)} \quad (3.18)$$
$$A= (0.10- 0.20)\ m^2/\text{capita} \times \text{No. of capita (Covered Beds)} \quad (3.19)$$

The dimensions of the drying bed cells are calculated as in below:
$$\text{Cell Area } A_c = L\ (\text{length}) \times W\ (\text{width}) \quad (3.20)$$

Number of cells $N_C = \frac{A}{Ac}$ (3.21)

(6) Locations of the Drying Beds: The best location for the drying beds depends on many factors such as;

1. The amount of produced sludge which will specify the required area also the land availability is an important factor that should be considered.
2. It should be far from any residential areas minimum 100 m to avoid odor problems.
3. It should be far from any water bodies.
4. The bottom needs to be sealed to prevent groundwater pollution and the drained sludge must be treated (Spuhler, 2010).
5. It is preferred to be at the end of the city and close to agricultural areas to be used as fertilizer.
6. Wind direction should not be toward the residential areas.

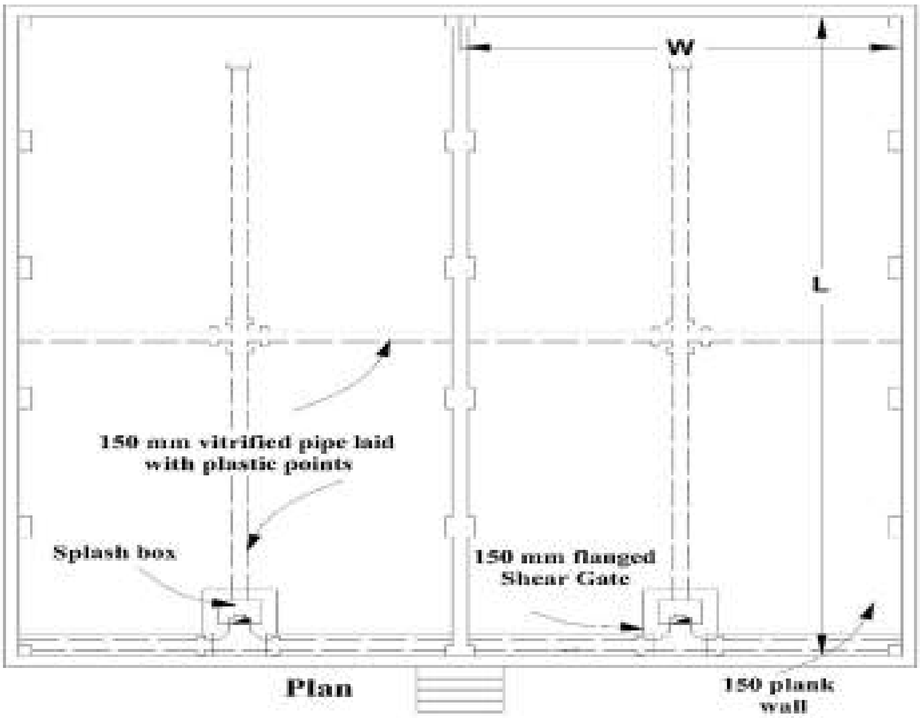

Fig.(3.7) : Typical Sand Drying Bed – Plan, (Eddy, 2014, p. 872)

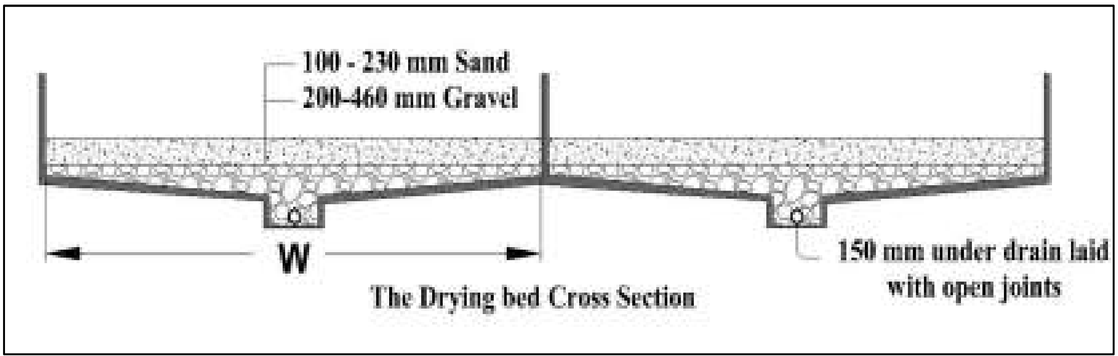

Fig.(3.8) : Typical Sand Drying Bed – Cross Section, (Eddy, 2014, p. 872)

3.6 Estimation of Landscape Irrigation Demand

Estimating the water demand for irrigating the landscapes (green lands) of the study area is one of the important parts in the work and it is directly integrated in the optimization model. Landscapes usually consist of a mixture of different plants and that make it difficult to find a single algorithm that produces accurate irrigation demand for the whole area. The value of the irrigation demand could be found using evapotranspiration (*ET*) method, which is based on the amount of water that evaporated and transpired from the plants (Stoughton, 2010). The daily water demand of the crop (*ETc*) could be calculated from Eq.(3.22), (Stryker , 2018, p. 1) as shown below:

$$ETc = Water\ Duty = \frac{ET_O\ x\ PF\ x\ SF}{IF} \tag{3.22}$$

<u>Where:</u>

ETc: water requirement for irrigation (Water Duty), m^3/day

ETo: referenced evapotranspiration, mm/day

PF: the plant factor, use 1.0 for lawns, 0.8 for shrubs and 0.5 for average shrub water use and 0.3 for low shrub water use.

SF: the area to be irrigated, m^2

IF: irrigation efficiency, it is the percentage of irrigated water that used by the plants and it depends on the type of irrigation system. For instance; *IF* = 0.80 for sprinklers and *IF* = 0.90 for drip irrigation system. It is recommended to use *IF* = 0.75.

The process of finding the required water demand for irrigation of landscapes is shown in Fig.(3.9), (Stoughton, 2010)**;**

(1) Estimate the Irrigation Area	(2) Identify Landscape Type	(3) Find the Evapotranspiration Value (*ETo*)	(4) Landscape Irrigation Use = Irrigation Area x *ETo*

Fig.(3.9) : Flow Chart of the Process of Finding the Water Demand of Irrigation of Landscape, (Researcher)

The First step in calculating the water demand is estimating the irrigation areas which could be found easily from GIS maps. Also it is necessery to identify the landscape types as each type requires different amount of water such as grass, trees , flowers and other vegitations.

CHAPTER FOUR

RESEARCH METHODOLOGY

Chapter Four
Research Methodology

4.1 Introduction

The main aim of this work was to find the optimum number, locations and capacities of the DWWTUs in Sulaimania City for reusing for irrigation and to eliminate the effects of untreated wastewater that discharged to Qilyasan stream without treatment. The work was divided into four major parts, which were;

1. Finding the optimum locations and numbers of the DWWTUs inside the city.
2. Finding the optimum capacity of each DWWTUs and find the optimum cost of reusing the reclaimed water from the treatment units for irrigating the green areas in the city.
3. Design of the DWWTs
4. Design the sludge disposal sand drying beds.

4.2 Methodology

The research methodology consisted of theoretical and practical parts. Many site visits to all city zones and villages of the study area, residential complexes, and green areas especially main green parks were done. Information about the population, the sewerage flow and water system, groundwater and wells were gathered. AutoCAD and GIS maps and data about the study area from related official authorities were collected.

Regarding the theoretical part, the first step in the work was to find the best locations and optimum numbers of the DWWTUs in the city using GIS and AHP. The next step was to find the optimum sizes of the DWWTUs using GA in a matrix form combined with GIS and the process was implemented in Matlab 2018a coding program. Many GIS models were created in the work such as land suitability model, Network Analysis OD Matrix to find the best cost piping routes. An objective function was derived based on the cost of the DWWTUs, the piping and pumping.

Moreover, a preliminary design was done to find the details of the components of the optimized DWWTUs. In addition, a sand drying bed was designed outside the city to collect the produced sludge from the DWWTUs. Fig.(4.1) shows the flowchart of the research methodology.

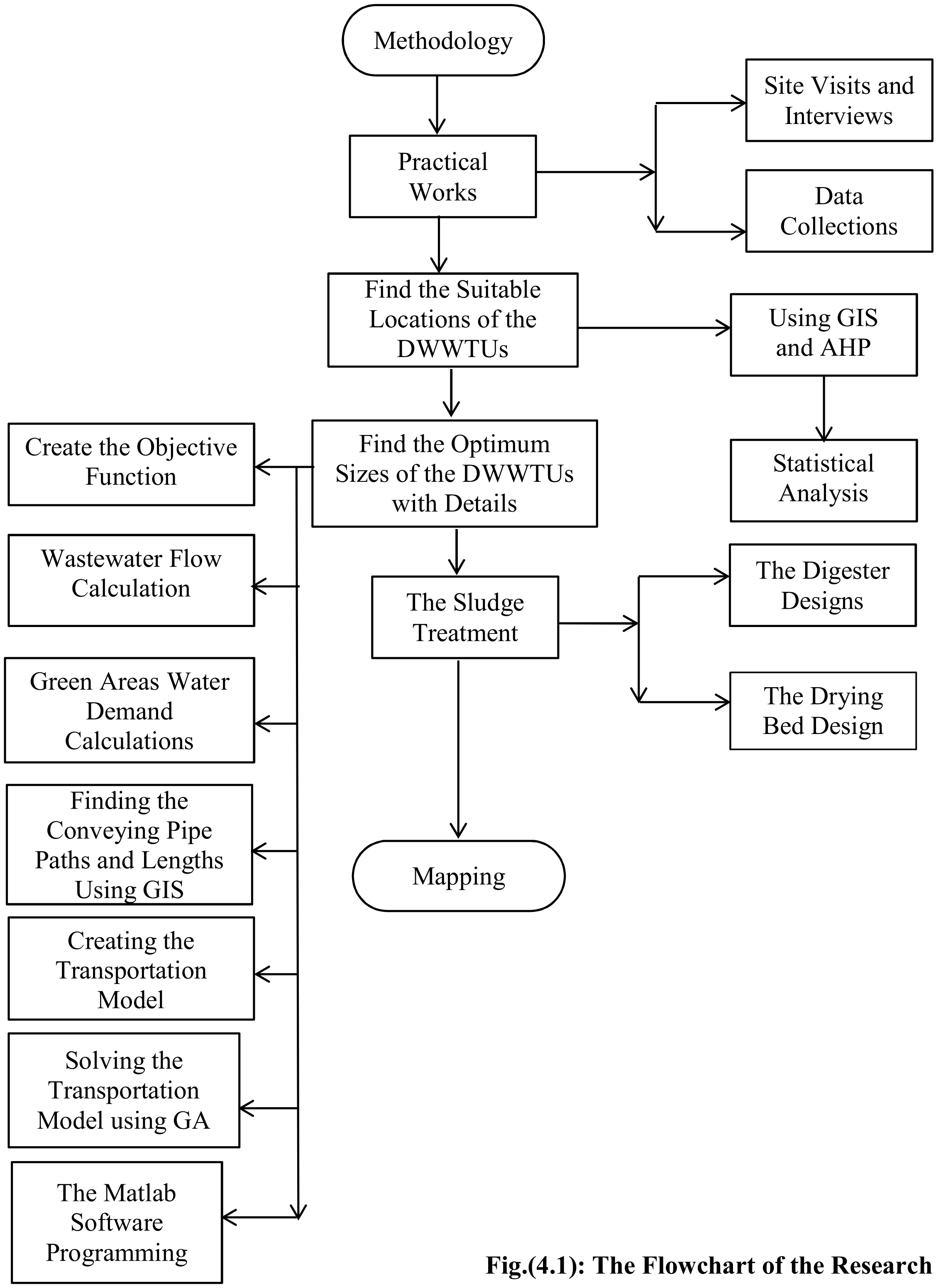

Fig.(4.1): The Flowchart of the Research

4.3 Site Description

This study is carried out in Sulaimania city, Kurdistan - Iraq. Sulaimania has a mountainous topographic area with elevation ranges between (645m to 1075m) amsl, the latitudes are between (35° 36′ 07″ N - 35° 31′ 35″N), and the longitudes are between (45°22′ 23″E - 45° 28′ 23″E) (GDOSM-GIS, 2017). Sulaimania city is divided into four suburbs which are: Main suburbs, Bakrajo, Rapareen and Tasloja (GDOSM-GIS, 2017). This research focused on Sulaimania Main suburbs only which has 156 districts as shown in Fig.(4.2). The study area suffers from lack of water for domestic demands, irrigation and industrial uses. Water is supplied to residential areas each three days and for a durations of 3 hours only (DOWS, 2017). In addition, the green areas are facing water shortages and the available water is not covering the water demand (GDOSM-Gardens, 2017). The water scarcity in the city is due to the rapid expansion of the city, climate changes and immigration from the surrounding areas. The main water sources of Sulaimania city are from Dukan and Sarchnar water treatment plants (Wash Cluster, 2015, p. 1) and also there are number of wells in the city .The amounts of water from those sources are not sufficient to cover all the requirements of the city (DOWS, 2017).

4.4 The Existing Sewer System

The sewer system of the city is combined with concrete box conduits used as main trunk sewers. The arrangements of the main sewer networks consist of 10 separate groups named as: Lines A, B, C, D, E, F, G, H, I and J. Each group is divided into branches as shown in Fig.(4.3) and the details are shown in Table (4.1) and table (A.1) in appendix A. At the end of each main sewer box, the wastewater is currently discharged to open areas though separate outlets then to Qilyasan stream without treatment. Table (4.2) shows the details of the sewer outlets of the study area. The arrangements of the sewer networks of Sulaimania City are suitable to be used in decentralized wastewater treatment systems

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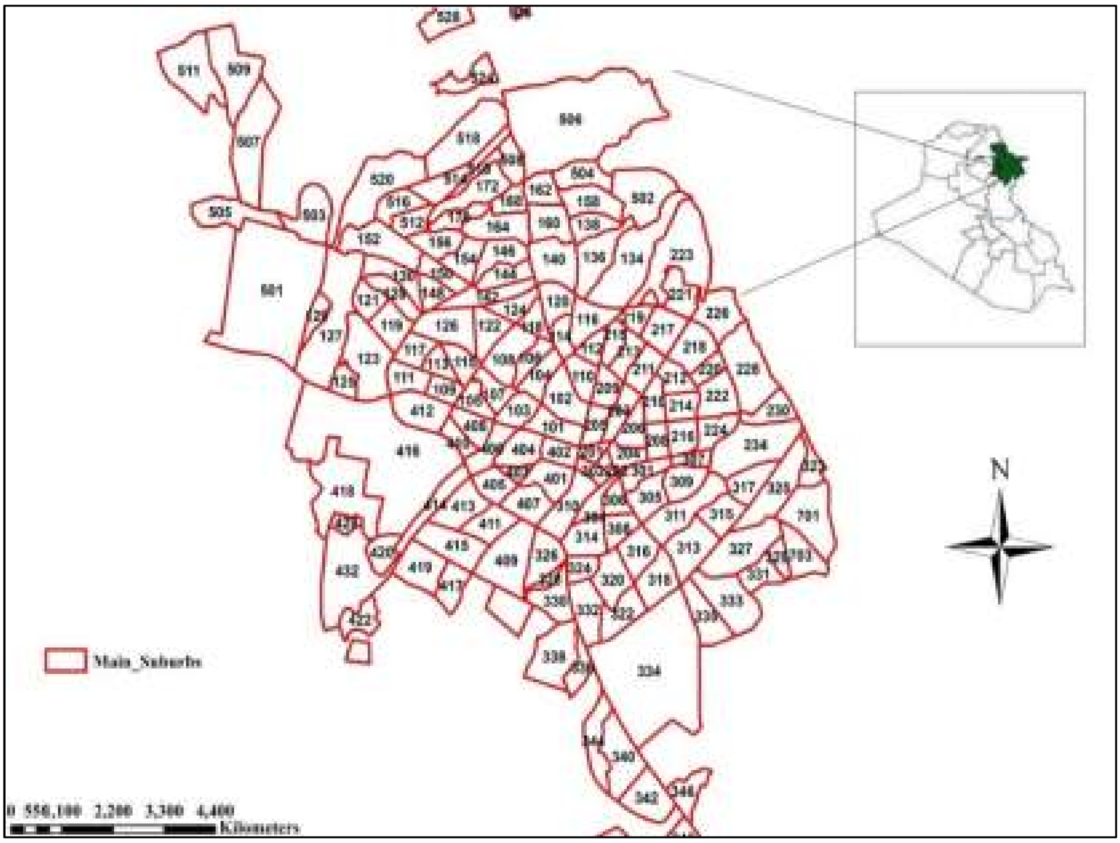


Fig.(4.2): The Districts of the Study Area (GDOSM-GIS, 2017)

Table (4.1): The Details of the Sewer Box Branches
(DOSWS, 2017)

Line	No. of Main Branches	Length, m	Line	No. of Main Branches	Length, m
A	7	7,186	F	7	11,022
B	16	18,579	G	28	25,171
C	25	21, 506	H	7	17,284
D	1	947	I	9	10,785
E	28	32,157	J	5	9,676

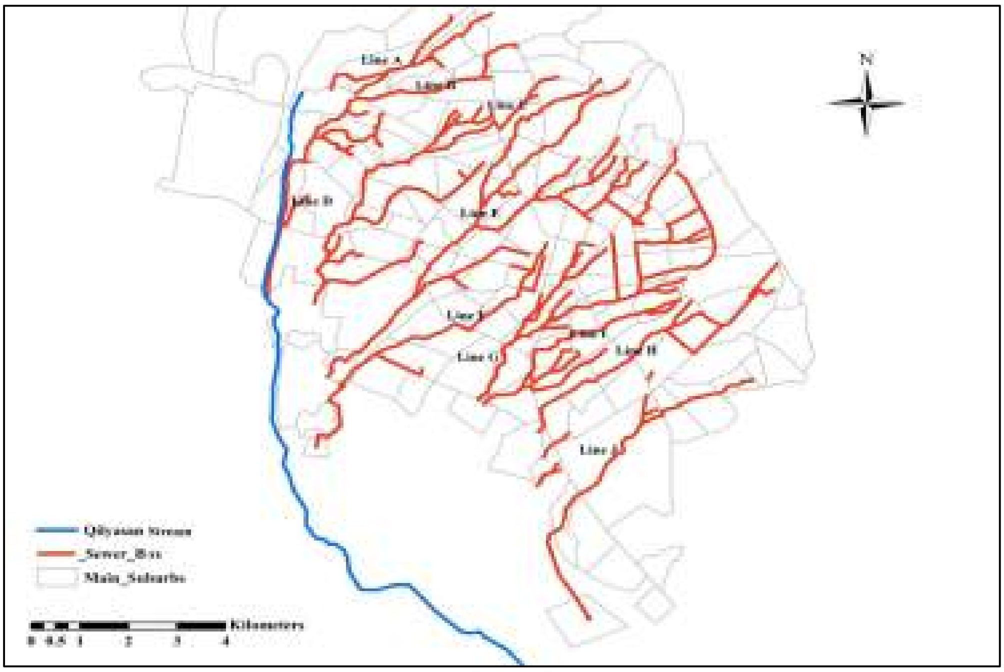


Fig.(4.3):The Main Sewer Box Layout of Sulaimania City, (DOSWS, 2017)

Table (4.2): The Details of the Sewer Box Outlets

Outlet No.	Size	Type of Box Sewer	Line
O1	2.50 m x 2.50 m	Single Box	A
O2	2.50 m x 2.00 m	Double Box	B
O3	3.00 m x 3.00 m	Double Box	C
O4	2.00 m x 2.00 m	Single Box	D
O5	3.00 m x 2.75 m	Single Box	E
O6	3.00 m x 3.00 m	Single Box	F
O7	2.50 m x 3.00 m	Single Box	G
O8	2.50 m x 3.00 m	Single Box	G
O9	2.00 m x 2.00 m	Single Box	I
O10	2.00 m x 2.50 m	Double Box	H
O11	1.00 m x 1.00 m	Single Box	J

4.5 The Existing Green Areas

There are many green zones (areas) in Sulaimania city like green parks with different sizes, green sectors in the road medians and green areas inside many residential compounds. Some of green zones are exist, others are proposed and some are under construction. The total green land size of the study area is about 17 Km^2, (GDOSM-Gardens, 2017) . In the study some green areas were excluded such as; (1) the green areas inside the residential compounds, (2) Hawari Shar park as there is a plan to have its own reusing system (Shar, 2018), (3) some green areas that located on the mountains and (4) Cemeteries. The total considered green areas are 4.74 Km^2, which consisted of different trees, flowers and grasses, as shown in Fig.(4.4). Water resources of irrigation of the existed green areas depend mainly on wells. Some of the wells are located at the same location of the green parks, some are far away, and trucks are used for conveying the water. The existed green areas are 85% of the total area, while 15 % are proposed and under construction (GDOSM-Gardens, 2017).

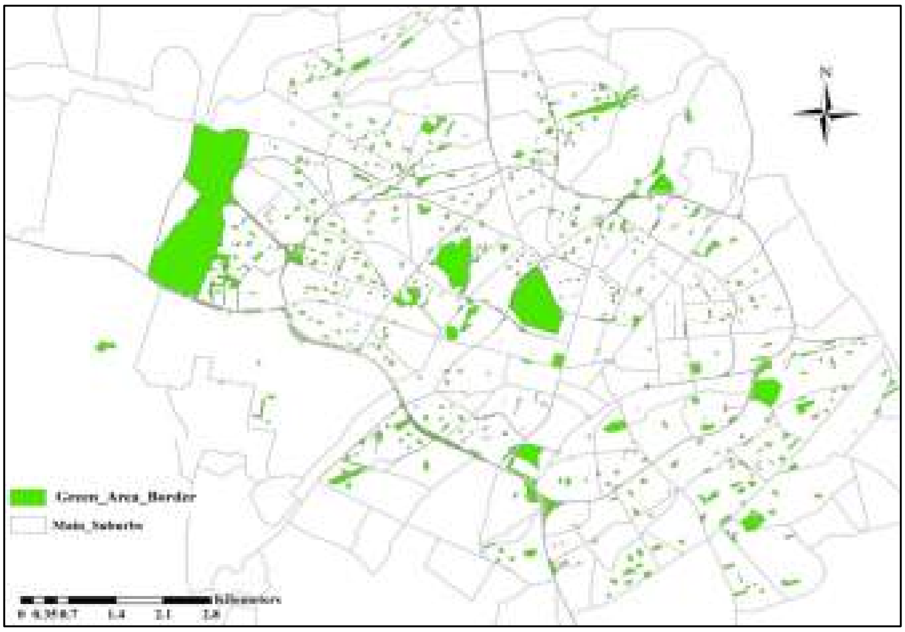


Fig.(4.4) : Green Areas of the Study area (GDOSM-Gardens, 2017)

4.6 Preliminary Selections of the Nominated Areas

A careful site study, visits and many interviews with authority representatives were made to collect information about the study area. The site visits to the districts were done during the research study and the visits were focused on data collection related to, (1) type of buildings, (2) sewer system, (3) populations, (4) available lands, (5) green areas and (6) the sewer outlets of each sewer line. Selecting the locations of the DWWTUs is considered as one of the essential elements in the work. The site locations of the treatment units have many effects such as; amount of reusing, cost of wastewater reclamations, amount of available water, and the cost of the sludge disposal. Selecting proper site locations of the DWWTUs can be affected by a number of factors such as; environmental parameters, economic considerations, social factors, technical aspects and reusing purpose.

From the site visit reports and the GIS map of Sulaimania City a preliminary selection of the site locations was done based on a number of criteria explained hereafter:

1. Size of the selected site location area is not less than 1,200 m^2
2. The site locations are not at the beginning of the sewer network and not far from the water networks.
3. The selected locations have accessibility to the roads
4. The selected lands are not located on a high leveled area in compare to the sewer box level
5. The selected site locations are located inside or close to the green areas

Based on the mentioned criteria, 134 nominated locations were selected and arranged into 10 groups which are; NA, NB, NC, ND, NE, NF, NG, NH, NI, and NJ located close to sewer lines A, B, C, D, E, F, G, H, I and J respectively. Fig.(4.5) shows some nominated areas located on lines A and B. The details of each nominated area are shown in table (A.2) in appendix A. The selected areas are evaluated and classified to find the suitability of each site location by applying Multi Criteria Decision Model (MCDM) using GIS integrated with Analytical Hierarchy Process (AHP). From the results of the suitability model, the best suitable areas are selected.

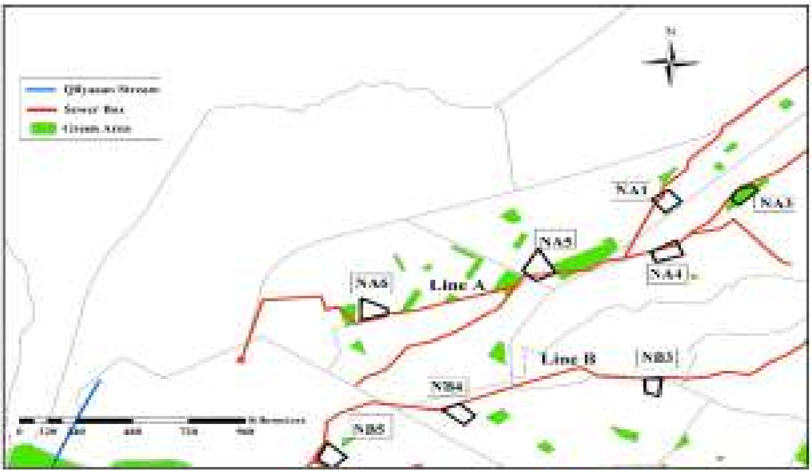


Fig.(4.5): Some Nominated Areas on Line A and Line B, (Researcher)

4.7 Multi-Criteria Decision Model

MCDM is used to select the suitable locations for the proposed DWWTUs. Five suitability criteria are used: (1) the size of the nominated areas, (2) distances from the nominated locations to the green areas, (3) slopes of the nominated areas, (4) population densities of the district where the DWWTUs will be placed and, (5) depth of the sewer box at the nominated area's location. Two restrictions are used in the model, which are: (1) the minimum distances of locations of the nominated areas are 30 m away from the surrounding buildings (EPA, 2000, p. 7) and, (2) the maximum distance of the main sewer box to the nominated areas is 50 m. Weighted Linear Combination (WLC) algorithm is used in the model. The suitability criteria are multiplied by the product of the area restrictions to find the land suitability index as in Eq.(3.1): $S_{index} = \sum_{i=1}^{n}(Wi \, . Ci) \prod_{j=1}^{m} r_j$ which applied into ArcGIS software by creating three GIS models, which are:

(1) Suitability Model
(2) Restriction Model
(3) Suitability Classification Model of the Nominated Areas

Figures (4.6), (4.7) and (4.8) illustrate the flowcharts of the structure of the three models respectively. The application of those three models to GIS is based on three main steps: data input and pre-processing, main processing, and output maps identifying the locations' suitability. In the Suitability Model, the value of $\sum_{i=1}^{n}(Wi \, . Ci)$ is calculated and the weights (Wi) of each criteria are measured from the AHP method which is explained in paragraph 4.5.3. The values of weights are applied in the Weighted Overlay Tool in the ArcGIS software. The Restriction Model is used to calculate the product of the area restrictions $\prod_{j=1}^{m} r_j$. The third model is performed by multiplying the Suitability Model times the Restriction Model. Six classes of the suitability of the nominated areas are obtained as shown in Table (4.3).

Table (4.3): Suitability Classifications of the Nominated Areas, m^2, (Researcher)

No.	Classification	No.	Classification
1	Restricted (R)	3	Very Suitable (V.S.)
2	Moderately Suitable (M.S.)	4	Highly Suitable (H.S.)
3	Suitable (S)	5	Extremely Suitable (E.S.)

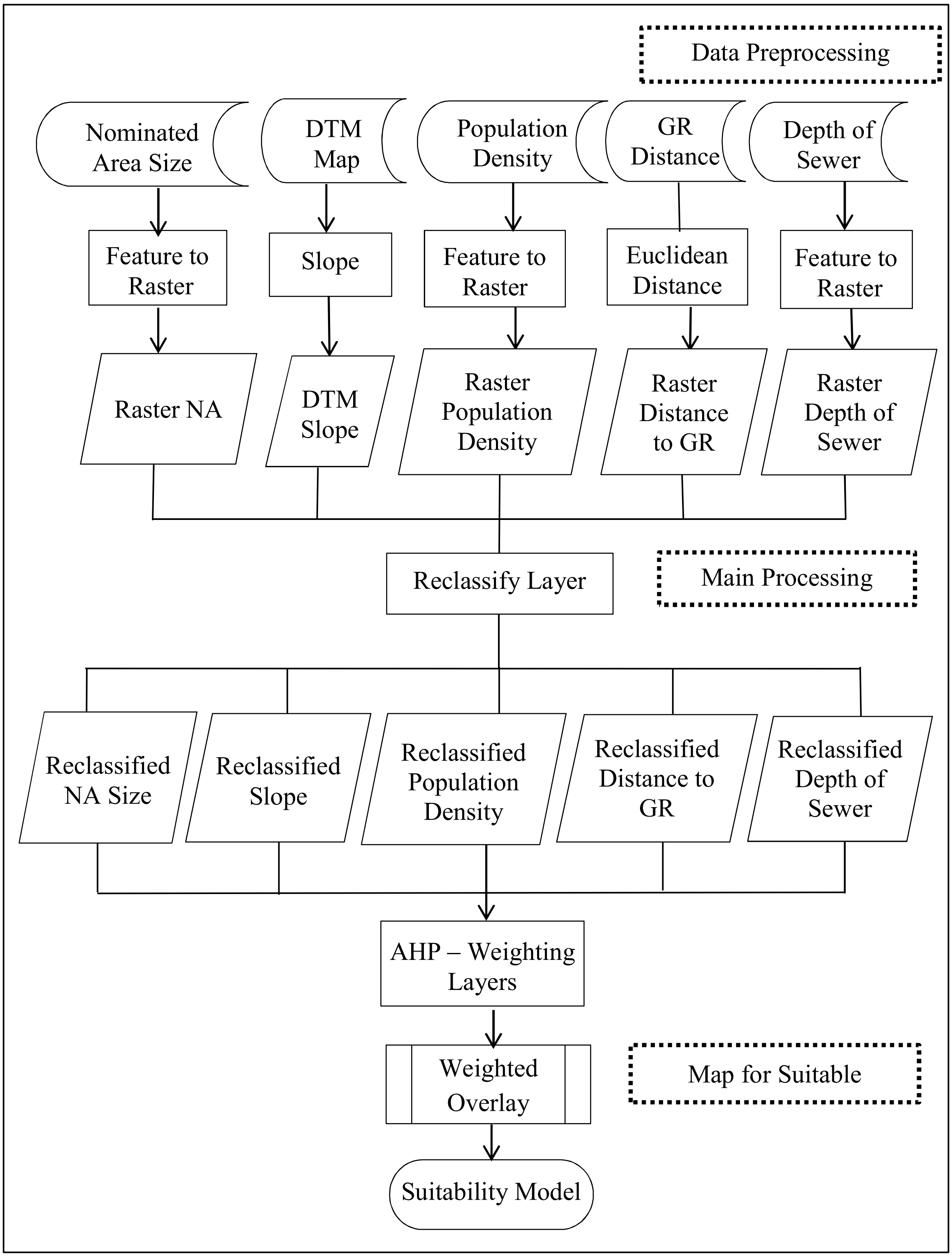


Fig.(4.6): The Flow Chart of the GIS Suitability Model Construction, (Researcher)

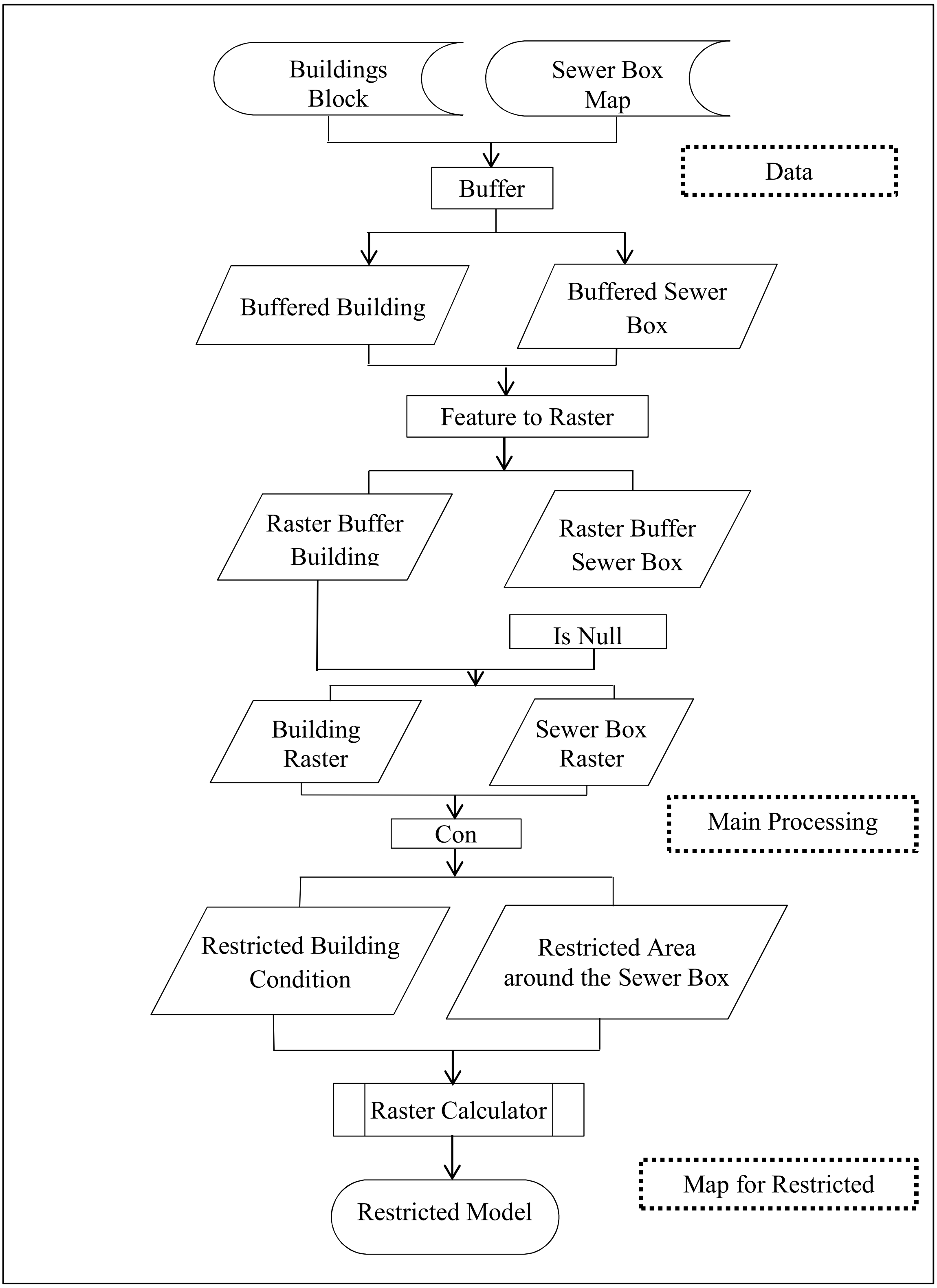


Fig.(4.7): The Flow Chart of Restriction Model Construction, (Researcher)

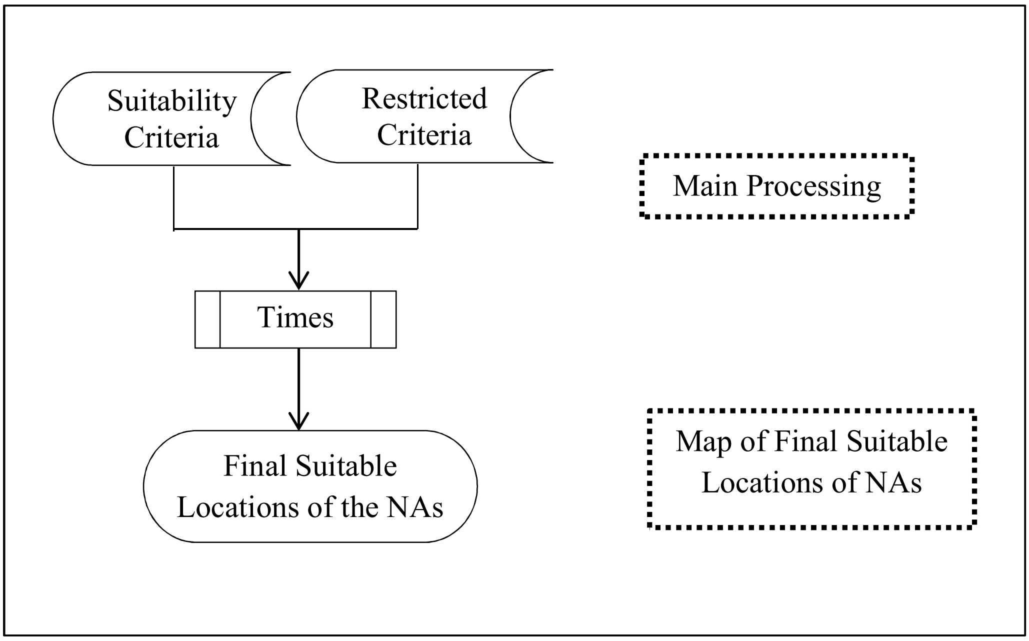


Fig.(4.8): Flow Chart of GIS Suitability Classification of the 134 Nominated Areas (Researcher)

4.7.1 Suitability Criteria

Five layers are used in the ArcGIS and each represents a suitability criterion. The criteria are measured in five different scales therefore; they are all classified in the GIS using Reclassify Tool. The reclassified layers will be weighted (multiplied by Wi and the results of the ranked layers are applied into the Weighted Overlay Tool of the ArcGIS to find Suitability Model's outcome. The details of each suitability criterion are shown below:

(1) Size of the nominated area (NA): The sizes of nominated areas that selected in Sulaimania city are different, ranged from small areas inside the city while large areas are located at the end of the city. The sizes of the nominated areas are classified into 7 ranks. The large areas will take higher rank as big areas will enable large DWWTUs to be installed and higher flow will be reused for irrigation. Fig. (4.9) shows the classified nominated areas according to the sizes.

(2) Distance to the green areas: The distances to the green areas from the nominated lands are calculated using Euclidean Distance Tool in ArcGIS and classified into 7 ranks. The closer distance will take bigger rank as once the distance is close it will be better and it will give less cost of conveying water to green areas. Fig.(4.10) shows the classifications of the nominated areas according to the distances to the green areas.

(3) Slope of the nominated areas: The natural land slopes of the study area (Sulaimania City) are classified into 5 classes. Less land slopes will take higher ranks in the model as it is more suitable for the installation of DWWTUs in terms of construction and operation. Fig.(4.11) shows the classification of the slope of the city land.

(4) Population density: Information about population of Sulaimania city from (DOSS, 2017) and from (GDOSM, 2017) was taken for each district of the city. Population density of the districts where the nominated lands were selected has been calculated individually. There is a significant difference in the population density in Sulaimania city between the districts. Old areas are crowded while new areas have small population density. Moreover, some places contain vertical building (residential complexes); they are also considered in the calculation of the population densities. In the GIS the population densities are classified into 7 classes. Areas with low population density will take high ranks, as it is not preferred to install DWWTUs in crowded areas. Fig.(4.12) shows the classifications of the population density of Sulaimania city. It was a big challenge to find the population of each district of the city. The available data of population of Sulaimania city from Directorate of Statistic was for 2009 and it was only for 101 districts. Population data from Sulaimania Municipality of 2002 was also taken and it was for 76 districts. The missed data was found by measuring the number of houses for new districts from AutoCAD and GIS maps and from site visits. The average number of capita per house was considered as 5.5 person (DOSS, 2017).

Geometric Population forecasting of 2018 is adopted in Sulaimania city and the annual rate of growth is equal to 3% as shown in Eq. (4.1 (GDOSM-GIS, 2017) and (Seureca, 2003, p. xi). Table (A.3) in appendix A shows the details of the populations of each district of Sulaimania city. Eq.(4.1) is used to find the population forecasting, (Gawatre, et al., 2016) ;

$$P_t = P_o \, x \, (1+ r \,\%)^{\,n} \tag{4.1}$$

Where

P_t forecasted population at year t ,(t = 2018)
P_o base population
r rate of growth, (r = 3%)
n no. of years

(5) Depth of the Sewer Box at Nominated Areas: The depth of the sewer box at the location of the nominated areas is very important, as it will specify the need of using pumping to lift the wastewater from deep sewer box to the treatment units. The depths are calculated for all main sewer boxes from the ground to the bottom of the sewer box. The depths are ranged from 2.10 m to 9.40 m and the calculations details of the ten main sewer lines are shown in tables (A.4a) to (A.4j) in appendix A. The nominated areas are classified into 7 classes based on the depths of the sewer boxes. Small depths are preferred and it will take higher rank as shown in Fig. (4.13).

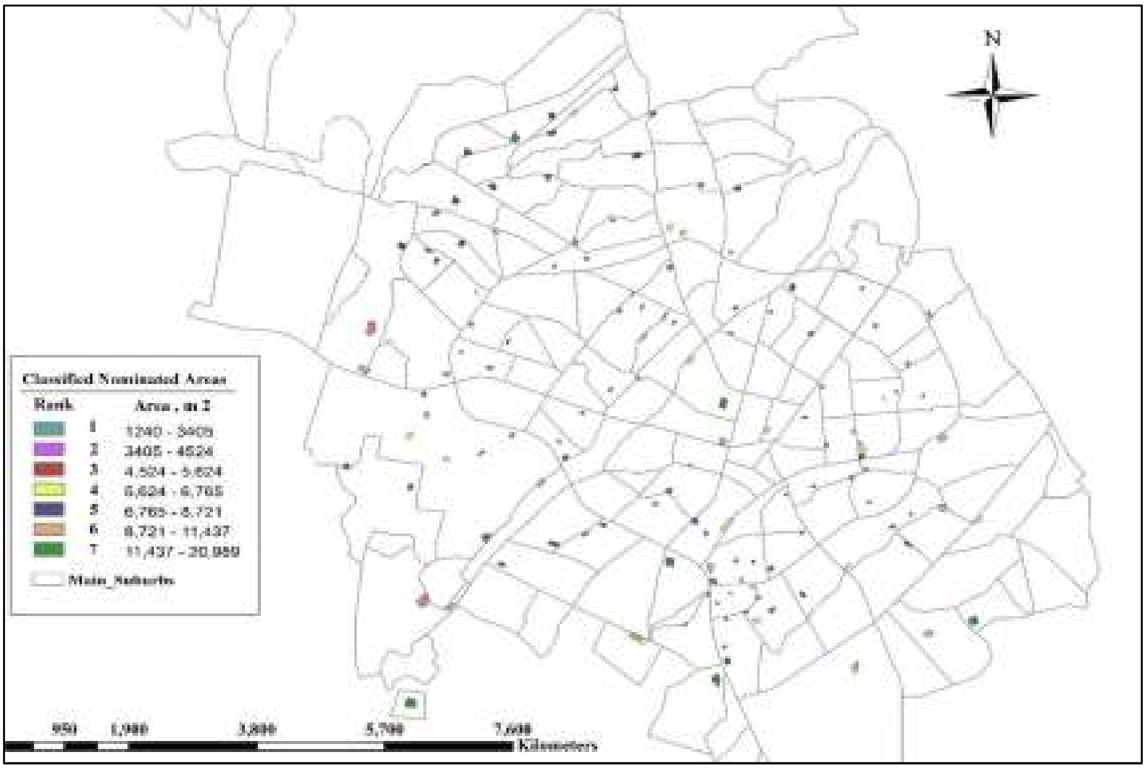


Fig.(4.9): Classified Nominated Areas (NAs)– Based on Size of Areas , (Researcher)

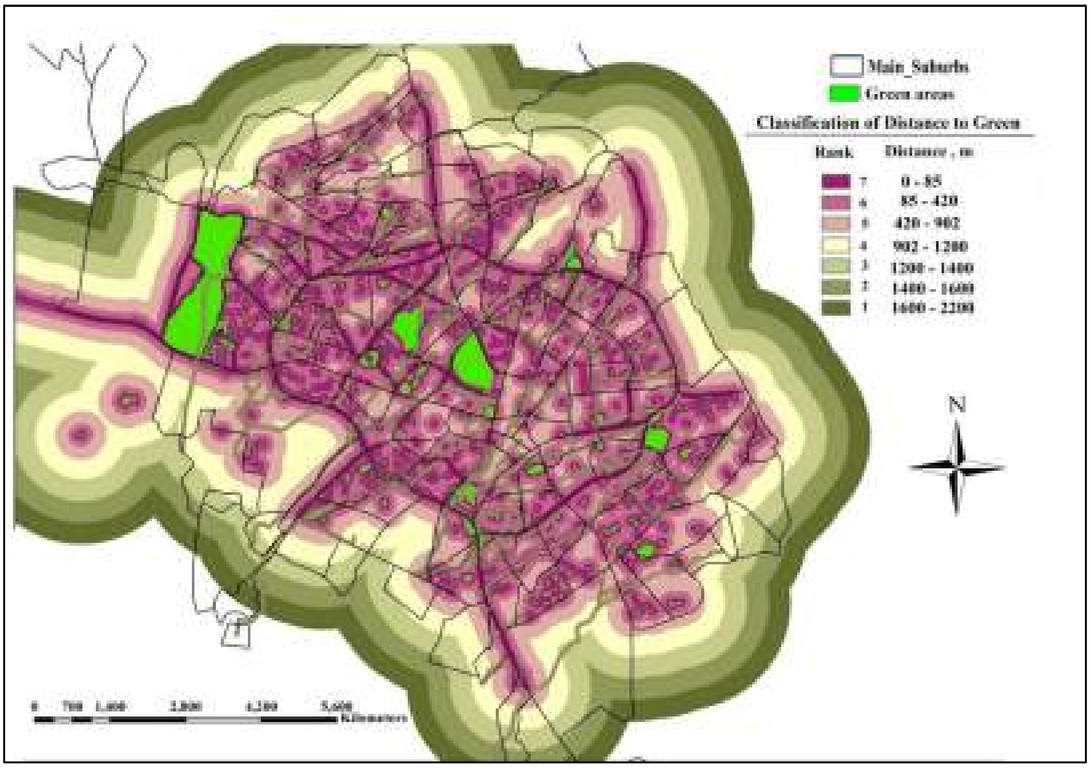


Fig.(4.10): Classifications of Distance to Green Area (GRs), (Researcher)

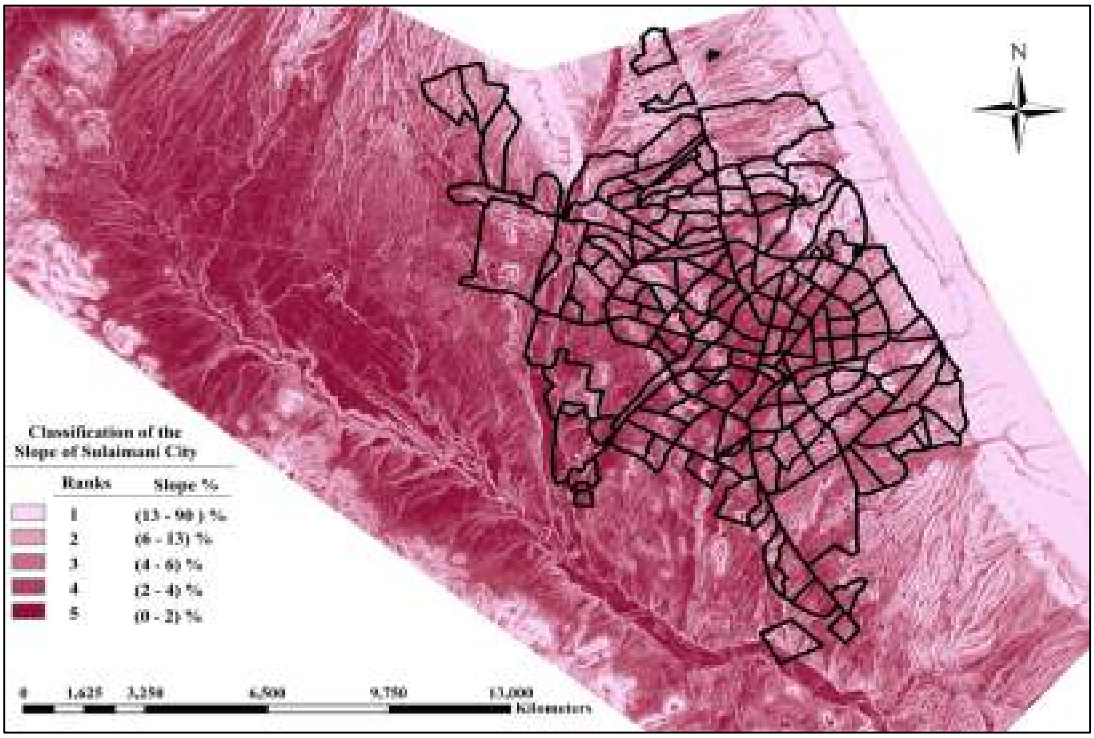


Fig.(4.11): Classification of the Slope of the Study Area, (Researcher)

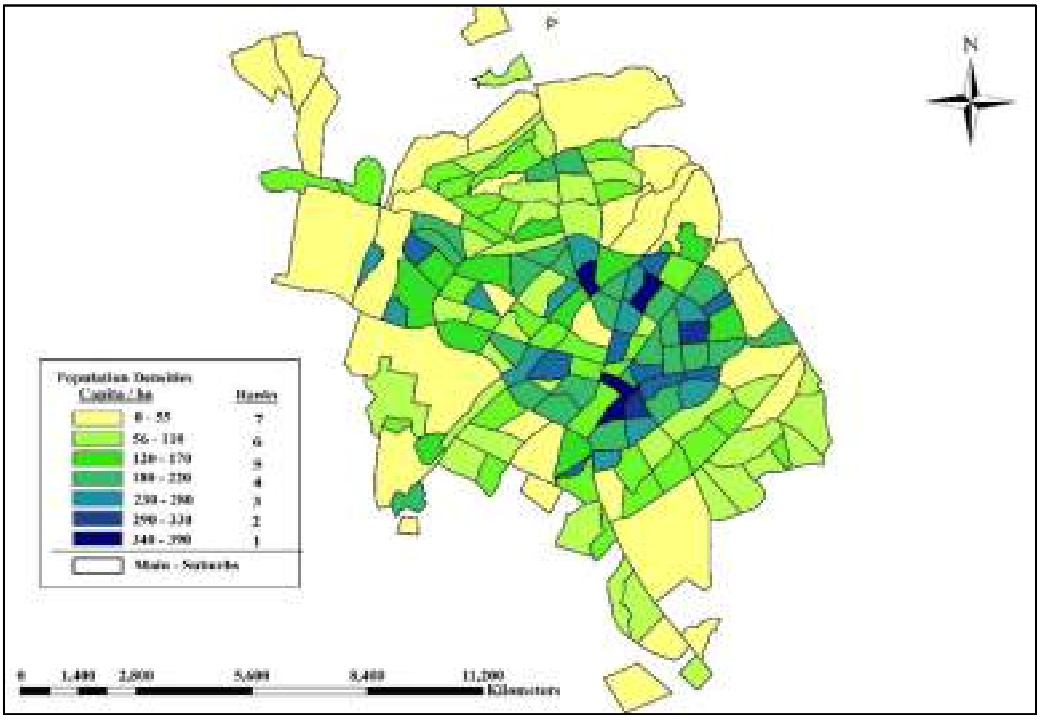


Fig.(4.12): Classification of the Population Density of Sulaimania City at each Suburbs, (Researcher)

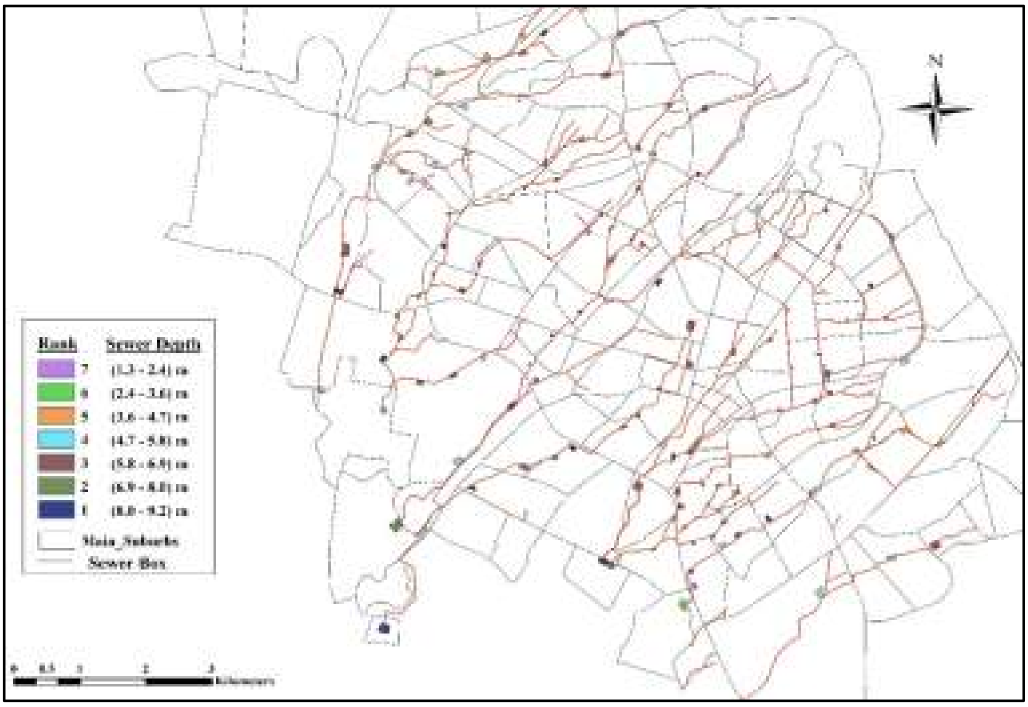


Fig.(4.13): Classification of the Nominated Areas Based on the Depth of the Sewer Boxes at the Nominated Area , (Researcher)

4.7.2 Restrictions criteria

The nominated areas should be close to the main sewer box to avoid high costs of connection works from the proposed DWWTUs to the sewer box and also to keep construction work far from the residential areas. The distance from the sewer box to the residential buildings are taken based on the characteristics of the area such as, average street widths and the distributions of the buildings. The width of the city's main street is 20 m, while the street widths inside residential areas are ranged from (5 -10) m or less in some places (GDOSM-GIS, 2017) and the buildings arrangement are close to each other. Therefore, a distance of more than 50 m will cause a big cost of excavation, construction and destruction of the surrounding area. In the GIS the sewer box line is buffered with a distance of 50 m from each side. The values within the buffer area (green color) will take a Boolean value of one while values outside the buffer area are the restricted area, and it will take a Boolean value equal to zero. Fig. (4.14) shows the restricted area around the sewer box. According to the environmental restrictions, the proposed DWWTUs should be far away from the residential buildings at least by a distance of 30 m (EPA, 2000, p. 6); the building layer is buffered with a distance of 30 m in the GIS program. The restricted areas are inside the buffer area and will take a Boolean value of zero (grey color). The area outside the buffered area is the allowable areas, and it has a Boolean value equal to one. Fig.(4.15) shows the details of the buffered areas around the buildings.

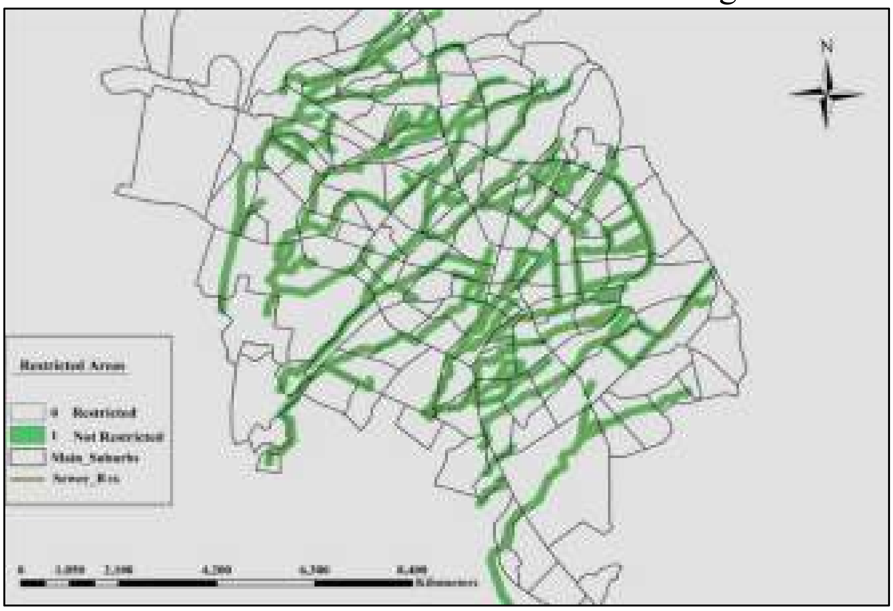


Fig.(4.14):Restricted Areas around the Sewer Box, (Researcher)

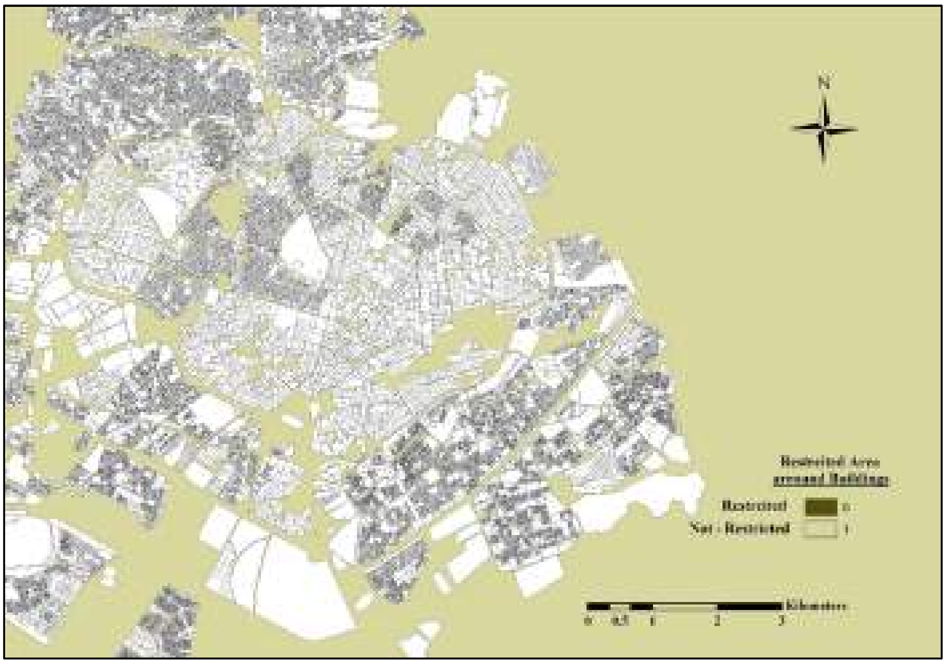


Fig.(4.15):Restricted Areas around the Buildings, (Researcher)

4.7.3 Analytical Hierarchy Process (AHP)

In this method, the magnitude of preference (the weight Wi) between factors is reflected. The influence of the factors is specified based on experience and wise judgment. The area size criterion is the preferred factor in comparison to the other factors as the land values are high inside the city. Moreover, obtaining lands inside the study area is difficult. The second preferred factor is the distance to the green areas as it has a significant effect on the cost of reusing the treated wastewater for irrigation. The city has a mountainous feature and far distances will need pumping to convey the treated wastewater, in addition, for long distances the lengths of the conveying pipes will be longer and that will be more expensive. The slope factor has less effect among the other suitability criteria as it is not difficult to change the nominated area's level and make it flat. The cost of leveling the area is less than the land value and less than the cost of water conveying. Population density also is important as treatment units in crowded areas may not be accepted by the people and it needs additional precautions and expenses. From practical experience, the additional precaution cost is still less than the cost of the land and cost of the distance to the green areas. Finally, the depths of the sewer boxes are evaluated also from practical experience and it is clear that for deep sewers, pumps will be required to lift the sewage to the treatment units which is not preferred. The costs of pumps are almost the same cost of conveying the treated wastewater to green areas but less than the cost of the lands and more than the cost of the land flatting. Table (4.4) shows the Pairwise Comparison Matrix for the five mentioned criteria.

Table (4.4): The Pairwise Comparison Matrix of the Five Criteria, (Researcher)

Suitability Criteria	Nominated Area size	Distance to GRs	Slope	Population Density	Sewer Box Depth
Nominated Area Size	1.0	2.0	3.0	2.0	2.0
Distance to GRs	0.5	1.0	2.0	2.0	1.0
Slope	0.33	0.5	1.0	0.5	0.5
Population Density	0.50	0.5	2.0	1.0	1.0
Sewer Box Depth	0.50	1.0	2.0	1.0	1.0
Column Sum	2.83	5.00	10.00	6.50	5.50

The normalized pairwise comparison matrix is derived by applying Eq.(3.2) by making the sum of the columns equal to one as shown in Table (4.5);

Table (4.5): The Normalized Pairwise Comparison Matrix of the Five Criteria, (Researcher)

Suitability Criteria	Nominated Area size	Distance to GRs	Slope	Population Density	Sewer Box Depth
Nominated Area Size	0.35	0.40	0.30	0.31	0.36
Distance to GRs	0.18	0.20	0.20	0.31	0.18
Slope	0.12	0.10	0.10	0.08	0.09
Population Density	0.18	0.10	0.20	0.15	0.18
Sewer Box Depth	0.18	0.20	0.20	0.15	0.18
Column Sum	1.00	1.00	1.00	1.00	1.00

The values of Wi is found by applying Eq.(3.3) and the detail is shown below:

W for Criterion number one (Nominated Area Size) is calculated as in below:

$$W1= (0.35 + 0.40 + 0.30 + 0.31 + 0.36) / 5 = 0.35 = 35\ \%$$

4.8 Cost Optimizing Model

After specifying the suitable locations of the DWWTUs the next step of this research is to create a mathematical model to optimize the cost of reusing the treated wastewater from the 31 DWWTUs for irrigation purpose in Sulaimania city. It is planned through this research to specify the amount of flow that could be treated by each DWWTU based on the required reclaimed water for irrigation. The objective optimization equation is a function of the cost of the treatment unit, the piping system, and the pumping cost and they are all functions of the treated flow. The capacity of each DWWTU will be calculated also as an output of this model. The amounts of available wastewater flow at each sewer box line and at each optimized nominated area that will be treated and reused are calculated in the following paragraph.

4.8.1 Wastewater Flow Calculation:

Sulaimania city consisted of residential, commercial, public and industrial areas. The commercial buildings in the city includes many shops, handicrafts, business buildings, malls, shopping centers, restaurants, hotels, motels ,cafes, oil stations, car services, warehouses .,etc. The locations of commercial buildings are distributed all over the city and have small effects on each individual DWWTUs. The car service buildings and maintenance areas are located mainly in a district called Peshasazi 416 as shown in Fig.(B.1) in appendix B. Therefore, no nominated areas are located in that area as the flow contains chemical that required advanced treatment. The details of the non – residential areas of the city are shown in Table(4.6). The sizes of the facilities mentioned in the table are small in compare to the total area of the city and they are scattered all over the city. Therefore, the considered contributing parts of the amount of wastewater flow will be for (a) Residential Buildings and (b) Residential Complexes.

Table (4.6): Area Sizes of the Non-Residential Districts of Sulaimania City
(GDOSM-GIS, 2017)

No.	Type	Area , ha	% of total area
1	Commercial (Shops and Handicraft)	503.37	4.9%
2	Administration Buildings	192.65	1.9%
3	Health Facility	66.37	0.6%
4	Schools and Universities	276.00	2.7%
5	Religion Buildings (Mosques)	24.74	0.24%
6	Religion Buildings (Churches)	0.93	0.01%
7	Sport facilities	21.85	0.21%

a. Residential Buildings:

Wastewater is collected from the residential areas through HDPE pipes with diameters ranged from 150 mm (house collecting pipe) to 1200 mm (Lateral and main pipes) which are connected to main concrete sewer boxes (DOSWS, 2017). The amount of wastewater from the residential areas that reach each DWWTU is estimated from Eq. (3.13). The results of the flow calculation of the residential areas are shown in tables (A.8a) to (A.8j) in appendix A. Sample of calculation for estimating the flow at optimized nominated area OA1 is shown below;

Sample of Calculation of Flow of Optimized Nominated Area OA1

The steps of the flow calculation are as in beneath;

1. Specify the nominated area's boarder (A_f), which is the part of district that their sewer system networks will discharge it's flow into the nominated area OA1 as shown in Fig.(4.16). The optimized nominated area OA1 will serve districts Qaiwan 514, Qaiwan 510, Hawari Shar 508 and Chnarok 172 as the A_f of each district are 686,973 m^2, 209,208 m^2, 381,694 m^2, and 485,364 m^2 respectively.

2. Find the population at A_f of the districts as in below:

$$\text{Capita} = \frac{A_f}{A_T} \text{ x Population of district} \qquad (4.2)$$

Where :

A_f: Nominated area boarder (area of flow), m^2

A_T: Total district area, m^2

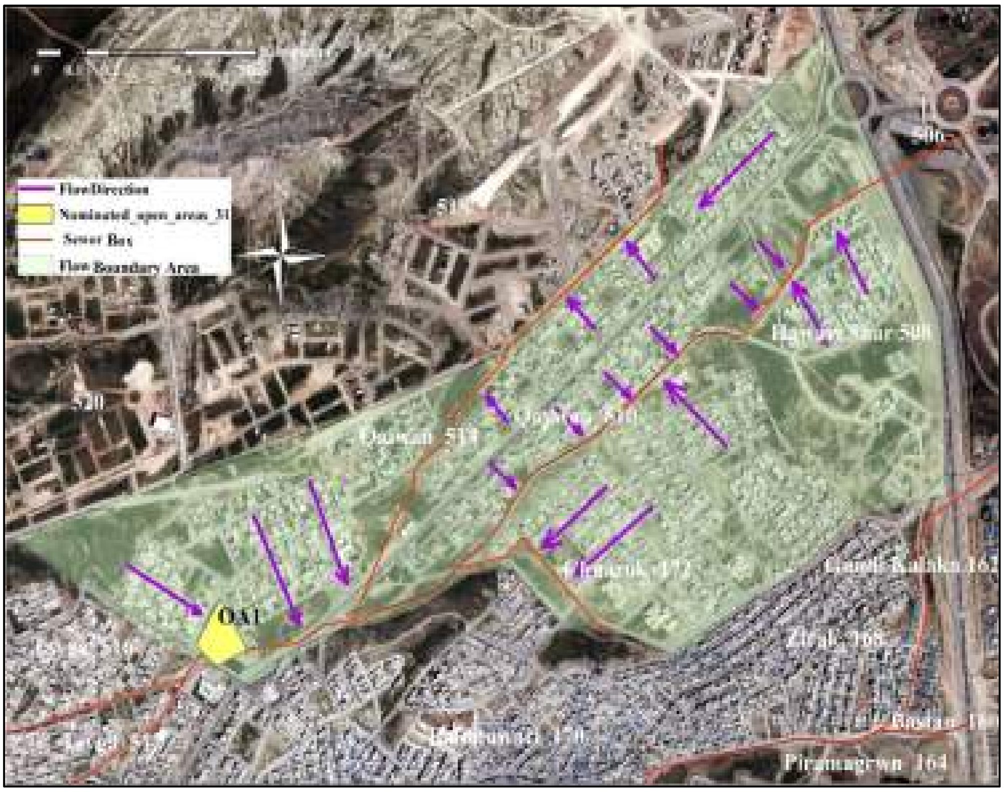


Fig. (4.16): Wastewater Collection at Optimized Nominated Area OA1, (Researcher)

3. Find the wastewater flow in the Sewer box at the nominated area Q_{av} by applying Eq.(3.13):

$$Q_{av} = (0.25\ m^3/Cap.day) \times 80\ \% \times Cap.$$

The results of the wastewater flow that reach the DWWTU named OA1 are shown in Table (4.7):

Table (4.7): The Results of Flow Calculation of DWWTU named OA1, (Researcher)

District Name	Population	A_T, m^2	A_f, m^2	A_f/A_T	Capita at Area of Flow	Flow, m^3/day
Qaiwan 514	4,932	686,973	686,973	1.00	4,932	986.49
Qaiwan 510	2,503	209,208	209,208	1.00	2,503	500.59
Hawari Shar 508	2,559	381,694	381,694	1.00	2,559	511.89
Chnarok 172	8,317	746,714	485,364	0.65	5,406	1,081.18
Average available total flow at optimized nominated area OA1						**3,080.15**

b. Residential Complexes

The study area includes 31 residential complexes located at different locations in Sulaimania city as shown in Fig. (B.2) in appendix B, (GDOSM, 2017).. The flow from each residential complex is calculated by applying Eq(3.13) as below:

Q_{avw} : average water supply flow = 0.20 m^3/Cap. day, % Return = 80 % , Capita = 5 cap/flat (DOWS, 2017)

The details of the wastewater flow produced from residential complexes are show in table (A.9) in appendix A. The total average flows through each sewer box during DWF that been calculated from both residential buildings and residential complexes are shown in Table (4.8).

Table (4.8): The Amount of Average Flow Calculation through each Sewer Box during DWF, (Researcher)

Sewer Box	Flow , m^3/day	No. of Nominated Areas
A	3,080	1
B	21,116	4
C	25,265	4
D	2,624	1
E	53,580	5
F	16,328	4
G	35,124	4
H	9,132	3
I	9,779	3
J	14,543	2
Total	**230,160**	**31**

c. Infiltration:

Is the water that enters the sewer system from the ground through defective pipes, pipe joints, connections, or manholes (EPA, 2014). The water table levels in the study area ranged from (650 m to 1025 m) amsl (Qaradaghy, 2015) and the depths of the water table are ranged from 10 m to 50 m. The sewer pipes are above the groundwater with a distance of more than 4 m at least even when the pipe depths reach 9 m then the infiltration is neglected in the study area.

d. Wastewater during Wet Weather Flow (WWF)

The sewer system of Sulaimania city is combined and in this research only Dry Weather Flow (DWF) is considered in the flow calculation as only a limited amount of the available wastewater will be taken to the DWWTU. The remaining amounts will pass through the sewer box. Only the wastewater quality will change during the storm time and that will be explained in the treatment details.

4.8.2 Green Areas' Water Demand

The total number of green areas that considered in the study is equal to 827 plots (GDOSM, 2017).The amount of irrigation demand will depend on the type of plants at each green area. Since the number of green areas are large, it will be difficult to know the details of type of vegetation. Also the available information does not explain the details of the contents of the landscapes.

Moreover, information related to the meteorological of Sulaimania City is not available to calculate ET_o. Therefore, the water duty value of the green area's irrigation is taken from an existing project in Sulaimania City which is the project of the irrigation system of New Sulaimania University Campus. The values that used in this project are 9 mm/day for grass and 3 mm/day for ground covers and trees (Tepe Construction Industry Inc, 2010).

In this research, one value of ET_o is taken as 10 mm/day for all the plants in the green areas. PF/ IF values are taken to be equal 1.0 and by applying Eq.(3.24) to find the water Duty;

$$ETc,\ (\mathrm{m^3/day}) = 10\ (mm/day) \times (1.0) \times SF\ (\mathrm{Area,\ m^2}) \times 10^{-3}\ (\mathrm{m/mm})$$

<u>Sample Calculation of Green Area (GR1)</u>

SF = 3,563 m^2 (The area size of GR1)

Located in Baranan 107 district area,
The demand = 10 (mm/day) x (1.0) x 3,563 (m^2) x 10^{-3} (m/mm) =35.63 m^3
Irrigation demand of each green areas in Sulaimania City (Q_d = ETc) are shown in table (A.10) in Appendix A.

4.8.3 The Objective Function

The aim of the optimization model is to calculate the amount of reclaimed water that will be reused from each treatment unit for irrigation of green areas and the capacity of the DWWTUs will be determined. Each DWWTU is surrounded by a number of green areas with different sizes and distances. In addition, some of the green areas are close to more than one DWWTUs. The optimum solution will also state the green areas that will be irrigated by each DWWTU. The developed objective function incorporates the cost of the DWWTU and the cost of the water pipelines to convey the reclaimed water to the green areas.

In this paragraph, the objective function (F) details and constraints are defined. The cost equation consists of the cost of the treatment units and the cost of conveying the reclaimed water to the green areas as shown in Eq.(4.3). The components of the cost equation are functions of the amount of treated flow (Q) that reach each green area from different DWWTUs. The amount of Q that gives the minimum cost value is obtained from the results of the model.

$$F = FT + FP + Fm \tag{4.3}$$

Where:

F: total cost function, \$

FT: treatment plant cost , \$

FP: piping cost, \$

Fm: pumping cost \$ (if pressurized pipe is used)

The details of the objective function are shown below:

1. The Treatment Plant Cost (FT)

The cost of construction and cost of operation and maintenance (O&M) of the treatment plant is considered. The general equation of the cost is as shown in Eq.(4.4), (Tsagarakis, 2003, p. 188):

$$F = a\ PE^{b} \tag{4.4}$$

Where

F: construction or O&M cost, \$

PE: population equivalent,

a , b: calculated coefficients

Calculating the values of the parameters (a and b) required real data related to the local market costs of existed extended aeration treatment plant. Unfortunately, there is no specific available data for the study area for such estimation. Hence, equations listed in literature are used as an alternative. As expected, this affects the estimated cost, but not affects the decision-making about the optimum sizes of the DWWTUs due to the relative effect, as the cost equation is used for all the treatment units. (Tsagarakis, 2003, p. 204) developed cost equations of the construction and the operation and maintenance (O&M) of a whole extended aeration plant in Greece as shown in Eqs.(4.5) and (4.6) respectively.

a. Construction Cost - FT1

$$FT_1 = (0.153)\ PE^{0.727} \tag{4. 5}$$

Where:

FT_1: Construction cost in 10^6 \$/1000 population equivalent,
PE: Plant size in 1000 population equivalent

b. O&M Cost - FT2

$$FT_2 = (0.0083)\ PE^{0.801} \tag{4.6}$$

Where:

FT_2: Annual O&M cost=10^6 \$/1000 population equivalent,
PE: Plant size in 1000 population equivalent

Cost of O&M was capitalized (FT_2') , from table project time life = 25 yr , i =10% (Interest Rate), P/A factor (Present Annual Payment) = 9.077, as follows, (Blank, 2012, p. 595) :

$$FT_2' = (0.07534)\ PE^{0.801} \tag{4.7}$$

The treatment plant's cost FT is found from Eq.(4.5) and (4.7) as in shown below:

$$FT = (0.153)\ PE^{0.727} + (0.07534)\ PE^{0.801} \tag{4.8}$$

Converting the *PE* to Q (flow m^3/sec):
Q = Population x (water demand 0.25 m^3/capita .day) x 80% (Return factor)
Q = Population (Cap.) x (2.32 x 10^{-6}) m^3/sec. capita ,
Q = Population /432,000
PE = Population/1000
PE = Q x (432) , [Q , m^3/sec] (4. 9)

Substitute into Eq.(4.8), The cost is multiplied by 10^6 to be in $;

$$FT = (12.61 \times 10^6)\, Q^{0.727} + (9.73 \times 10^6)\, Q^{0.801} \quad (4.10)$$

2. The Piping System Cost (FP)

The reclaimed water discharged to the surrounding green areas through pipe networks, and it could be by gravity or by pumping depending on the elevation differences between the locations of the DWWTUs and the green areas. The pipe head loss also considered in the calculation. Pipe lengths and land elevation differences are calculated using GIS as explained in a later paragraph. The general cost equation form of the pipe cost used in the research is shown below, (Swamee, 2008, p. 82)**:**

$$FP = K_m\ L\ D^m \quad (4.11)$$

Where;

FP:	the pipe construction cost [the pipe cost+ installation], $
L:	pipe length, m
D:	pipe diameter, m
K_m, m:	coefficients related to the pipe material

From the local market prices of HDPE – 100, PN16, values of K_m = 63.494, and m = 1.2616, the calculation detail is shown in table (A.11) in appendix A. By applying the values of m and K_m into Eq. (4.12), the cost equation of the pipe will be as shown below:

$$FP = Cost = 63.4.94\ D^{1.2616} \ x\ \ L \quad (4.12)$$

The treated flow will be stored in tank T1 in the DWWTU's location and discharged to tank T2 in the green area. The residual pressure at T2 assigned as a constraint to be ≥ 2 m. The residual head estimated due to the elevation difference between the locations of the DWWTU's and the green areas and the head losses of the conveying pipes as in the Eq. below:

$$Residual\ Pressure = (Z_o - Z_1) - (h_f \times 1.2) \tag{4.13}$$

Where

Z_o : the elevation of the DWWTUs locations, amsl

Z_1 : the elevation of the green area, amsl

h_f: the pipe head loss , m , [multiplied by 1.2 for minor losses]

if $(Z_o - Z_1) - (h_f \times 1.2) \geq 2$ Then the gravity pipe will be used

if $(Z_o - Z_1) - (h_f \times 1.2) < 2$ Then Pumping will be used

The Hydraulic Constraints:

$0.6 < v < 1.5, m/sec$, $v = \frac{Q}{A}$, $A = \pi \frac{D^2}{4}$, residual pressure at Tank T2 ≥ 2 m

From Darcy equation, h_f is found as in Eq.(4.14), (Swamee, 2008, p. 14);

$$h_f = \frac{8 f\ L\ Q^2}{\pi^2\ g\ D^5} \tag{4.14}$$

$$f = \{ (\frac{64}{Re})^8 + 9.5\ [ln\ (\frac{\varepsilon}{3.7\ D} + \frac{5.74}{Re^{0.9}}) - (\frac{2500}{Re})^6]^{-16} \}^{0.125} \tag{4.15}$$

$$Re = \frac{4\ Q}{\pi\ \upsilon\ D} \tag{4.16}$$

Where;

h_f: pipe head loss , m

υ: kinematic viscosity of fluid , m^2/s

L: pipe length, m

Q: treated effluent flow, m^3/sec

g: gravitational acceleration = 9.81 m/sec^2

ε: pipe roughness height, m

f : the pipe roughness coefficient, [for laminar and turbulent flow]

Re : Reynold Number

The values of the parameters are: υ *at 20 ° C= 1.012 x* 10^{-6} m^2/sec , ε *for HDPE 100 = 0.05 x* 10^{-3} *m*

Pump Head – Pressurized Pipe

$$h_o - (Z_1 - Z_0) - (h_f \times 1.2) \geq 2 , \quad (4.17)$$

Where

h_o: the pump head, m

3. The Pumping Plant Cost (Fm)

The cost equation of the pumping (Fm) consists of the costs of pumping house construction C_p and the operation cost Ae , as shown below (Swamee, 2008, p. 81):

$$Fm = C_p + Ae \quad (4.18)$$

Where:

Fm: cost of the Pumping system, $

C_p; cost of pumping plant construction, $

Ae: cost of pumping operation, $/yr

a. **Pumping Cost (C_p) in terms of Flow**

$$C_p = K_p \; P^{m_p} \quad (4.19)$$

Where:

K_p: coefficient

P: power in KW

m_p: an exponent

$$P = \left[\frac{(1+Sb)\rho \, g \, Q \, h_o}{1000 \, \eta} \right] \quad (4.20)$$

Where:

ρ: Density of water , Kg/m^3

Q: flow, m^3/sec

h_o: pump pressure head , m.

S_b: stand by fraction of the pump = 0.5 - 0.75 (use 0.5)

η: Pump efficiency = 0.68

The parameters K_p and m_p are related to the market prices and construction material type and it will be obtained from a known set of pumping capacities by plotting a cost curve. (Swamee, 2008, p. 82) used a list of a real pumping station cost data and obtained values of K_p and m_p to be equal to 5560 and 0.723 respectively. By substituting Eq.(4.20) into Eq.(4.19) and applying the parameters the pump cost C_p will as in Eq.(4.21):

$$C_p = 5560 \left[\frac{1.5\, \rho\, g\; Q\, h_o}{1000\, \eta}\right]^{0.723} \tag{4.21}$$

b. Cost of Operation of Pumping Plant (Ae)

The pumping system cost includes the annual operation cost of pump energy in \$/year as shown in Eq. (4.22) (Swamee, 2008, p. 87)**:**

$$Ae = \left[\frac{8.76\; \rho\; h_0\, Q\; R_E}{\eta}\right] \tag{4.22}$$

Where:

Ae: the annual cost of pumping station operation, \$/year
Q Pump flow, m^3/sec
η: pump efficiency, let η= 68 % (assumed)
R_E: rate of electricity cost, \$/ KW-hour

Capitalizing Annual costs of O&M of the Pumps

From table project time life = 25 yr , i =10% (Interest Rate) , P/A factor (Present Annual Payment) = (9.077) , as follows, (Blank, 2012, p. 595) :

$$Ae' = \left[\frac{8.76\; \rho\; h_0\, Q\; R_E}{\eta}\right] \; x\,(9.077) \tag{4.23}$$

Where:

Ae': The capitalized cost of the pumping station operation, \$

Substituting Eq. (4.21) and (4.23) into Eq.(4.18):

$$Fm = 5560\left[\frac{1.5\,\rho\, g\;\; Q\, h_o}{1000\,\eta}\right]^{0.723} + \left[\frac{8.76\;\rho\;\; h_o\, Q\;\, R_E}{\eta}\right] x\,(9.077) \qquad (4.24)$$

4.8.4 Pipes Layout and Length Calculations Using GIS

GIS map and software used to find the lengths (L) and best routes of the pipes that convey the treated wastewater from the DWWTUs to the green areas. Network Analysis - OD Cost Matrix method is used to find the least cost paths along a network from a number of origins to certain destination points (ESRI, 2013)**.**The road layer of Sulaimania City is used as a path layer of the pipe routes that connecting the DWWTUs and the GRs. In this study, the optimized nominated area's locations centroids represent the origin (31 points), and the green areas centroid represents the destinations (827 points). The cutoff distance in the GIS network analysis was selected to be equal to 1000 m (the maximum path length from the origin to destination point). The result shows the paths between each DWWTUs location and the surrounding green areas (within the 1000 m path). Although the lines are straight, they are representing a real path distance through the road layer between the origin and the destination point. The structure of the GIS Network Analysis – OD Cost Matrix is shown in Fig.(4.17). Each DWWTU is connected to a number of GRs, and on the other hand, some GRs are connected to more than one DWWTU.

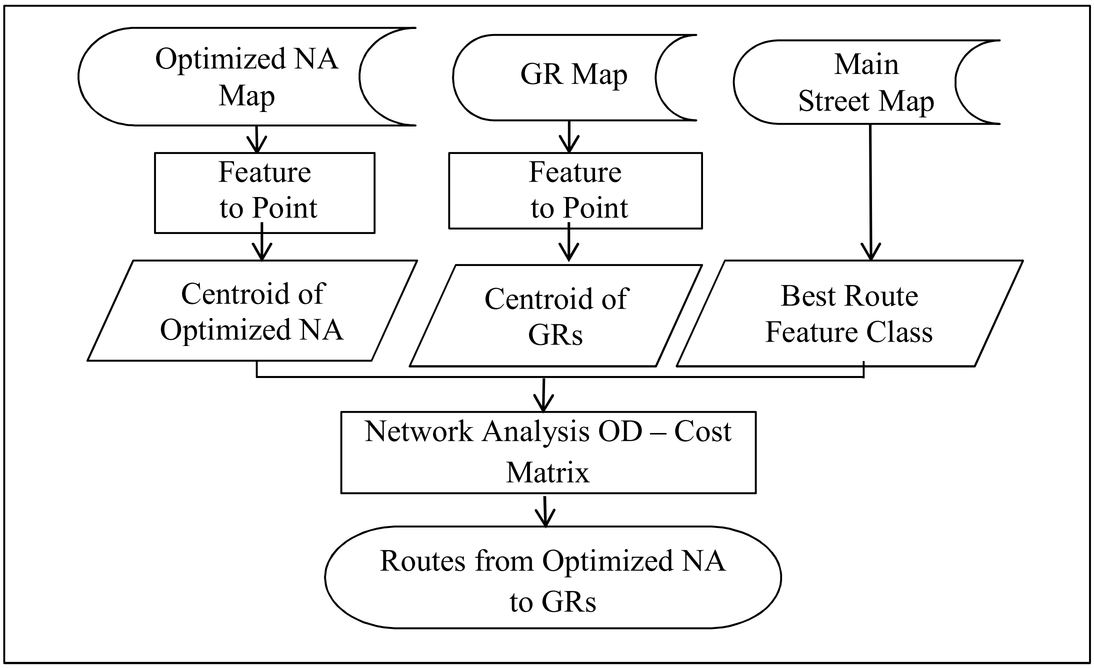


Fig.(4.17): The Flow Chart of GIS Network Analysis OD – Cost Matrix , (Researcher)

4.8.5 Elevation Difference between the DWWTUs and the GRs

Elevation differences (ELD) between the locations of the DWWTUs and the GRs that linked with is found to specify whether the conveying will be by gravity or by pumping. The elevations of the study area are found using Digital Terrain Model (DTM) map in GIS of the Study area. The flowchart of the GIS structure of the process is shown in Fig.(4.18). The ELD between the locations of the DWWTUs and the green areas are calculated. The depth of the sewer box at the DWWTU and the depth of the underground treatment unit (4m) are considered when calculating the elevation difference.

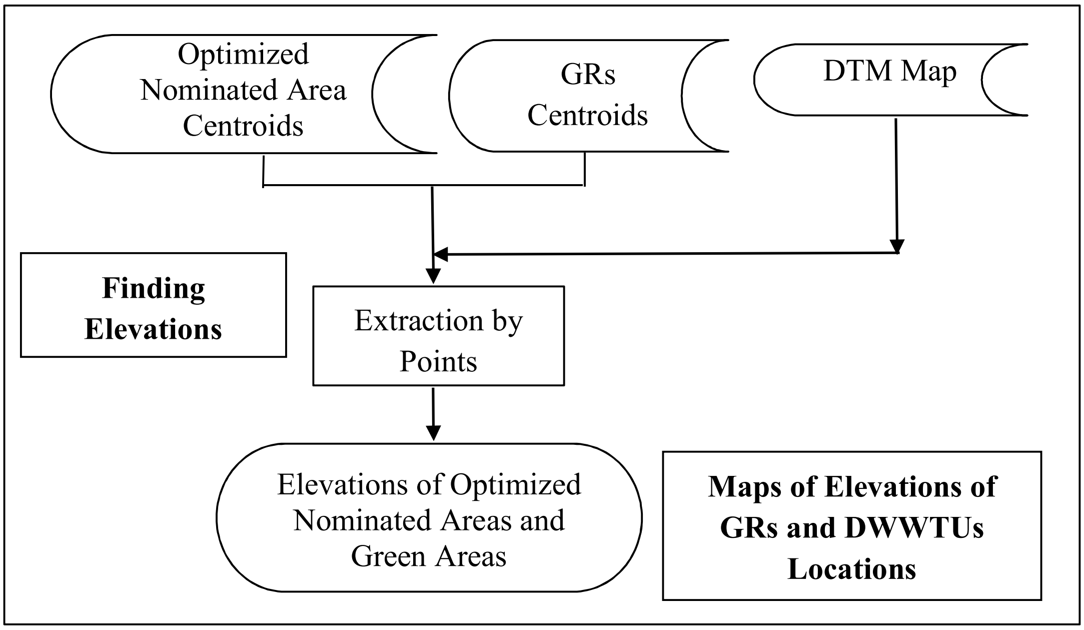


Fig. (4.18): The Flow Chart of Finding the Elevations of DWWTUs Locations and Green Areas Process using GIS, (Researcher)

4.8.6 The Transportation Model and the GA

Transportation model will provide the best way of distributing the reclaimed water to get the minimum cost of conveying and the maximum benefit. The reclaimed wastewater will be conveyed from the DWWTUs (origin points) to the green areas (destination points). The whole site (Sulaimania City) was considered together in a one transportation matrix, as there are some green areas that could be supplied from deferent DWWTUs. The cost element in the model is only for the piping network and for the cost of pumping system (if pressurized pipe is used). The cost of the treatment plant is not included in the transportation model and it will be measured separately based on the amount of flow that will be specified according to the amount of required reclaimed water for each green area.GA in a matrix form is used to solve the optimum amount of supplied flow to the green areas from each treatment unit. The methodology of the algorithm is by distributing the flow from each treatment unit to the connected green areas groups (within the 1000 m path). The amount of flow that will reach the green areas from the DWWTUs will be changed randomly and the cost will be calculated repeatedly until reaching the optimum solution. The details are shown in the following paragraphs:

a. The Transportation Model:

As explained in previous chapter three, the transportation model is represented by the amount of flow of reclaimed water *Qij* that supplied from each DWWTUs *i* (origin *i*) *to* each green area *j* (destination *j*) through pipe networks. The transportation array is representing the cost of supplying reclaimed water *fij* from each origin to each destination as shown in Fig. (4.19). The size of the array is equal to [31 x 827] as there are 31 DWWTUs and 827 green areas in the study area. The amount available (*ai*) represents the available reclaimed water flow treated at each DWWTU (origin *i*) and the amount required (*bj*) represents the irrigation demand of the green areas (destination *j*) . From the results of the OD – Matrix Analysis of GIS not all of the green areas will be supplied with water and that is because of either they are not close to any treatment unit or they are out of the cutoff path (1000 m). Those green areas that have no connection with the treatment units will be exist in the matrix but an amount of zero flow will be allocated for them.

To / From		Destination *j* (Green Areas GRs)					Amount Available a_i m^3/d
		GR1	***GR2***	***GR3***	…	***GR827***	
Origin *i* (DWWTUs)	*DWWTU1*	Q_{11} f_{11}	Q_{12} f_{12}	Q_{13} f_{13}	…	$Q_{1\,827}$ $f_{1\,827}$	$a1$
	DWWTU2	Q_{21} f_{21}	Q_{22} f_{22}	Q_{23} f_{23}	…	$Q_{2\,827}$ $f_{2\,827}$	$a2$
	DWWTU3	Q_{31} f_{31}	Q_{32} f_{32}	Q_{33} f_{33}	…	$Q_{3\,827}$ $f_{3\,827}$	$a3$
	⋮	⋮	⋮	⋮	⋮	⋮	⋮
	DWWTU31	$Q_{31\,1}$ $f_{31\,1}$	$Q_{31\,2}$ $f_{31\,2}$	$Q_{31\,3}$ $f_{31\,3}$	…	$Q_{31\,827}$ $f_{31\,827}$	$a31$
Amount Required b_j , m^3/d		$b1$	$b2$	$b3$	…	$b827$	

Fig. (4.19) : The Transportation Array of Conveying Flow from the DWWTUs to the GRs , (Researcher)

The value of cost fij represents the cost of piping from the DWWTUs to the green areas and the cost of pumping (if pumping is required) , it is found by applying Eqs.(4.12) and (4.24) as in below:

$$Total\, f_{ij} = \sum_{i=1}^{31}\sum_{j=1}^{827} f_{ij} = \sum_{i=1}^{31}\sum_{j=1}^{827} FP_{ij} + \sum_{i=1}^{31}\sum_{j=1}^{827} Fm_{ij} \qquad (4.25)$$

The total cost F of the objective function is equal to the cost of the treatment plants FT and the cost of piping and pumping ($Total\, f_{ij}$) . The FT cost is obtained by applying Eq.(4.10) and as in below:

$$Total\, FT = (12.61 \times 10^6)\ \sum_{i=1}^{31} Q_i^{\,0.727} + (9.73 \times 10^6) \sum_{i=1}^{31} Q_i^{\,0.801} \qquad (4.26)$$

$$F = Total\, f_{ij} + Total\, FT$$

Model Constraints

There are number of constraints in the model related to the amount of flow and others are related to the residual pressure at the green areas. Three constraints related to the flow should be satisfied which are:

(1) *Constraint*-1 : the amount of flow that reach each green area from the treatment units should be equal to the required demand at each GR.

$\sum_{i=1}^{31} Q_{ij} = b_j , \quad j= 1,2,.....,827$

(2) *Constraint*-2 : the total amount of flow required at each GR should be equal or less than the available flow .

$\sum_{j=1}^{827} Q_{ij} \leq a_i , \quad i= 1,2,.....,31$

(3) Constraint-3 : the amount of flow from each DWWTUs should not be a negative value .

$Q_{ij} \geq 0 , \; i=1,2,....,31, \; j=1,2,...827$

The constraint that related to the residual pressure is for both pressurized and gravity pipe, the residual pressure at the green area (tank T2) should be ≥ 2 m as explained in the objective function.

b. The Genetic Algorithm (GA)

GA is utilized to solve the model to get best values of Q_{ij} that gives the minimum cost solution *F*. Matrix form GA is used and the steps of the *GA* that been followed in the process are as shown below:

- **i. Initialization**: in this step a random number of solutions (*Np* =100, 200, 300, 400, 500, 600, 700, 800, 900, and 1000) are created based on different Q_{ij} and each solution represents a chromosome. Each chromosome is represented in a matrix of size [*31 x 827; NP*]. Solutions that not fulfilled the constraints will be eliminated. For instance, for *NP* =1000 if only 700 solution satisfy the constraints, the new *NP* will be equal to 700.
- **ii. Selection:** In this research all parents that satisfied constraints 1 and 2 are selected to be mated, that means 100% of the populations will survive and no chromosome will be killed.
- **iii. Crossovering:** new solutions will be produced by creating offspring from parent populations. Since the summation of each column represents the demand of each green area (*bj*) the crossovering process will done for columns to fulfill *constraint*- 1. Different location points (*PCO*) are taken, *PCO* = 5, 50, 100, 150, 200, 250, 300, 350, 400, 450, 500, 550, 600, 650,

700, 750 and 800 for each *NP*. The process is illustrated in Fig.(4.20) and Fig.(4.21).

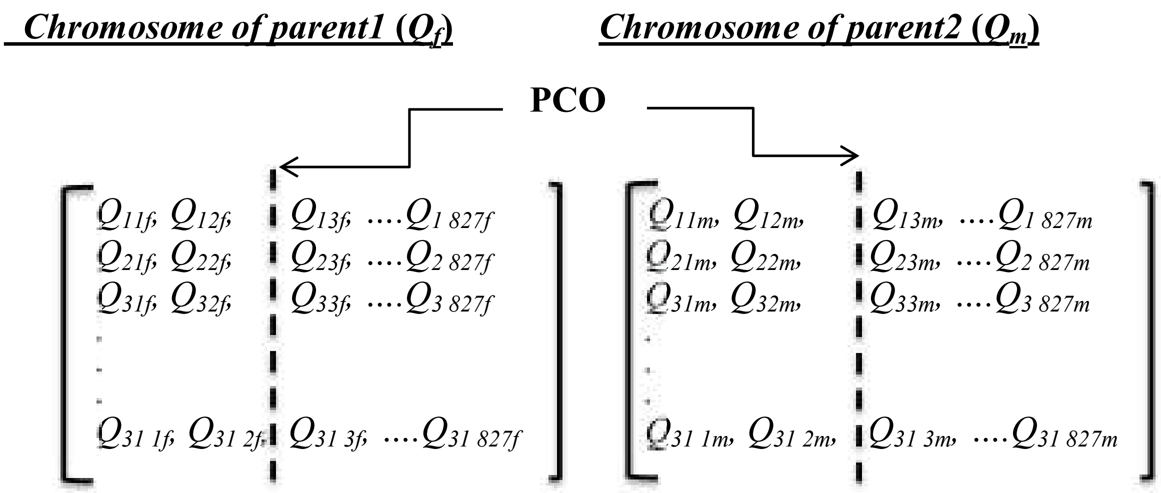


Fig.(4.20) : The Parents Before the Crossovering – *PCO* = 2

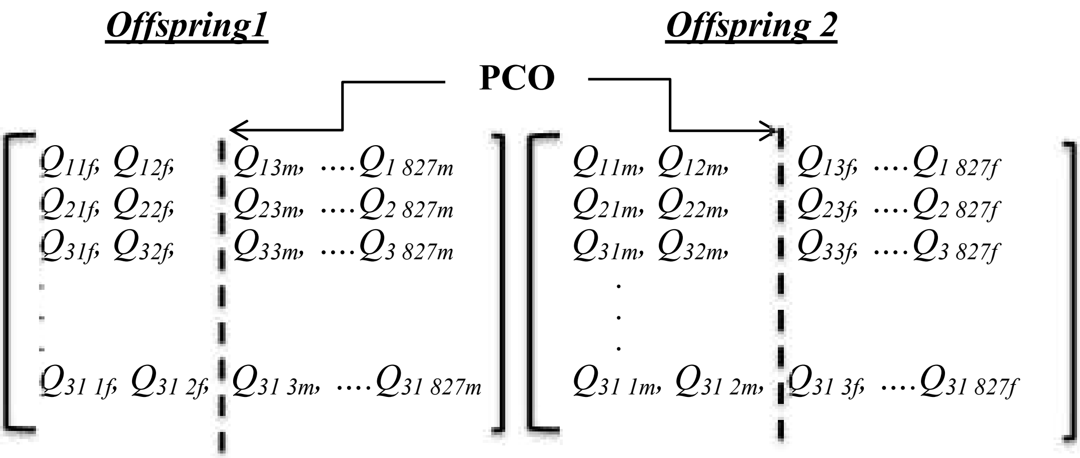


Fig.(4.21): The Produced Offspring after the Crossovering, (Researcher)

New population is created from the crossovering process and the new population will have a size equal to (*2 x Np*) [parents + offspring]. The new solutions are checked if it satisfied the constraints and the final population consists of the solutions that fulfill the constraints.

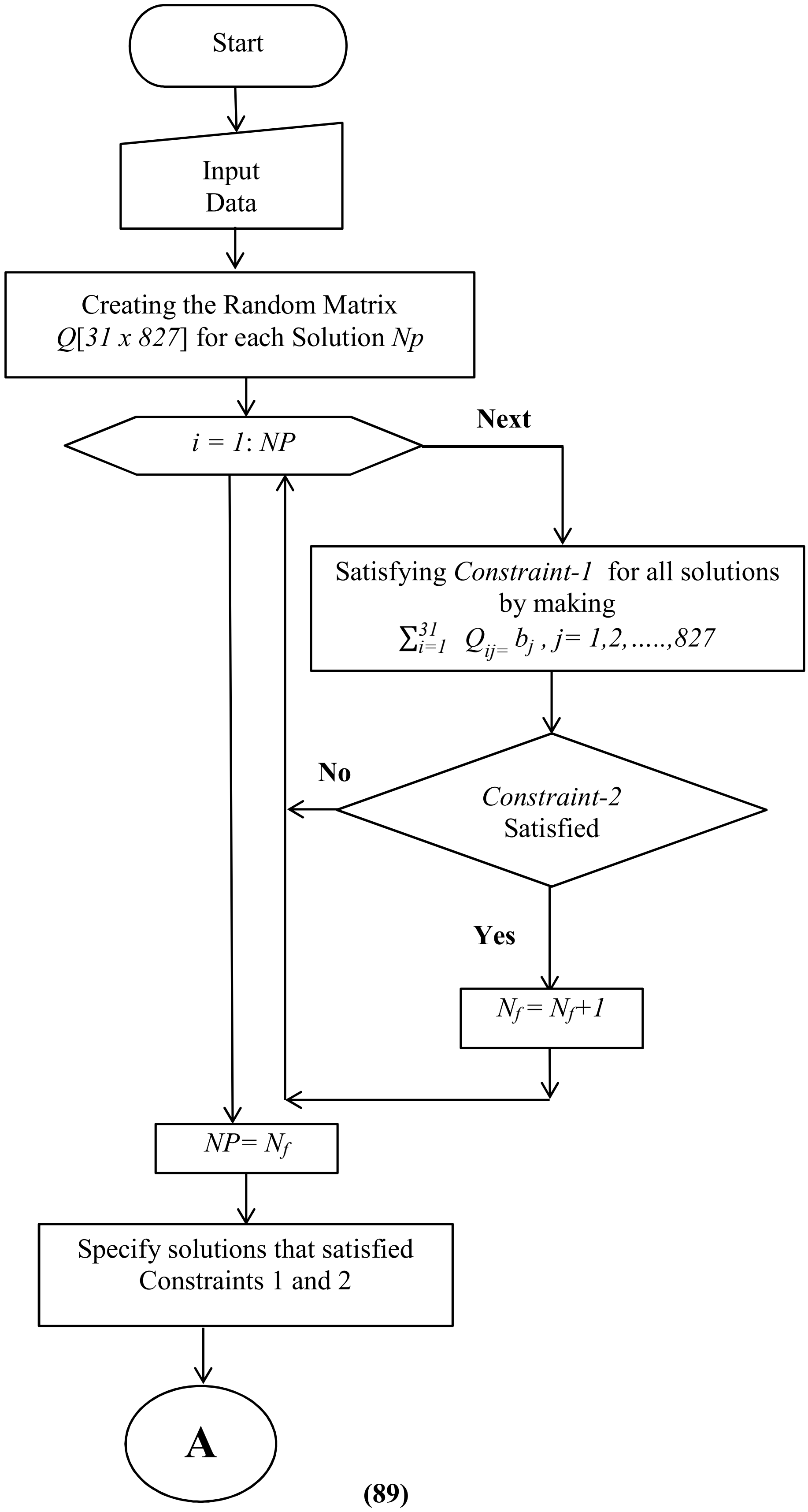
iv. **Evaluations:** The solutions that produced in step (**iii**) are applied into the objective function [Eqs.(4.29) and (4.30)] to find the cost values of each solution. The cost results (F_{min}) are arranged in an ascending order to find the optimum cost solution.

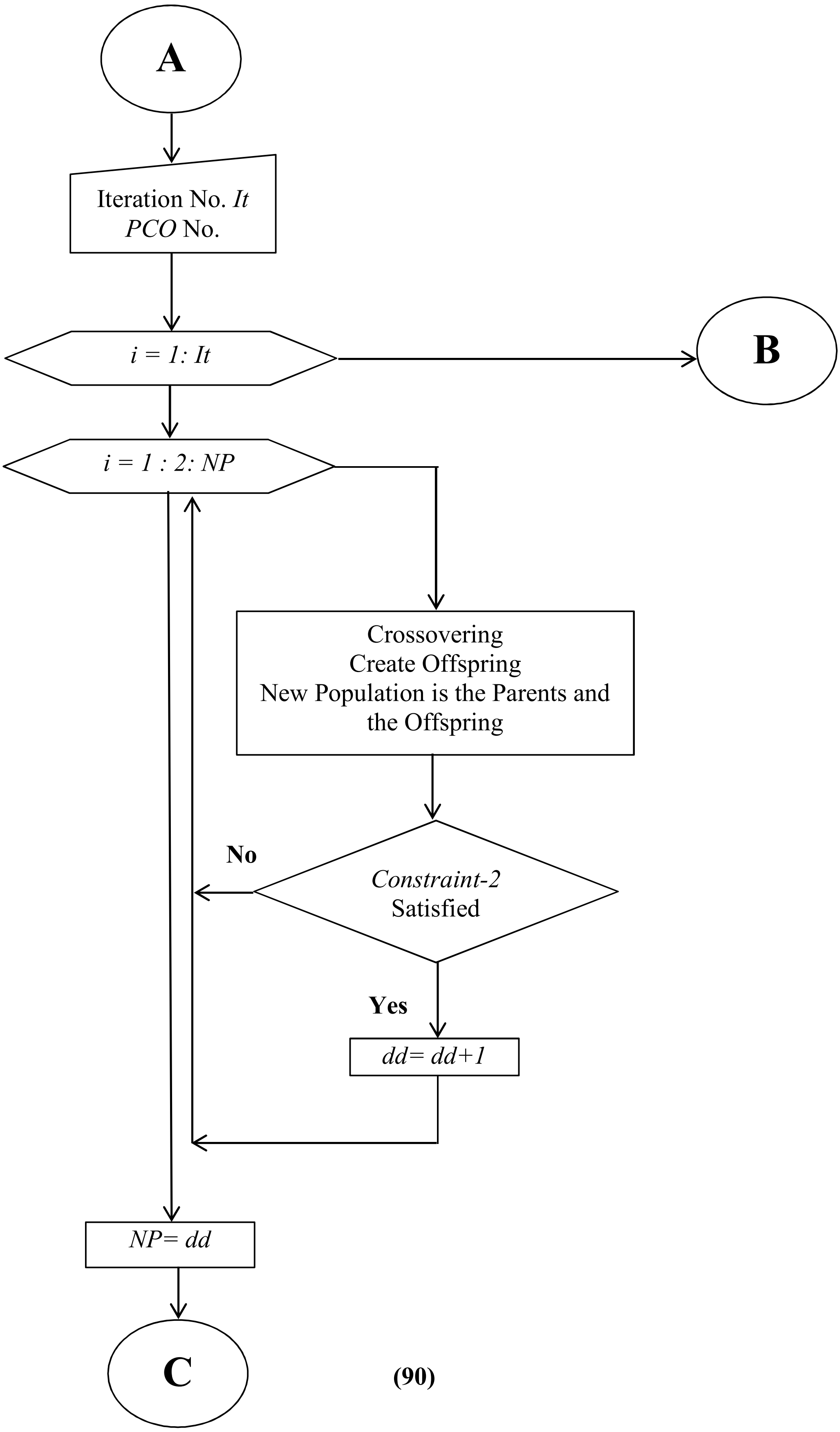
v. **Iterations (It):** The above steps (**i** to **iv**) are repeated four times (It = 4) and for each iteration the optimum cost solution is calculated. The final solution will be for the least cost results.

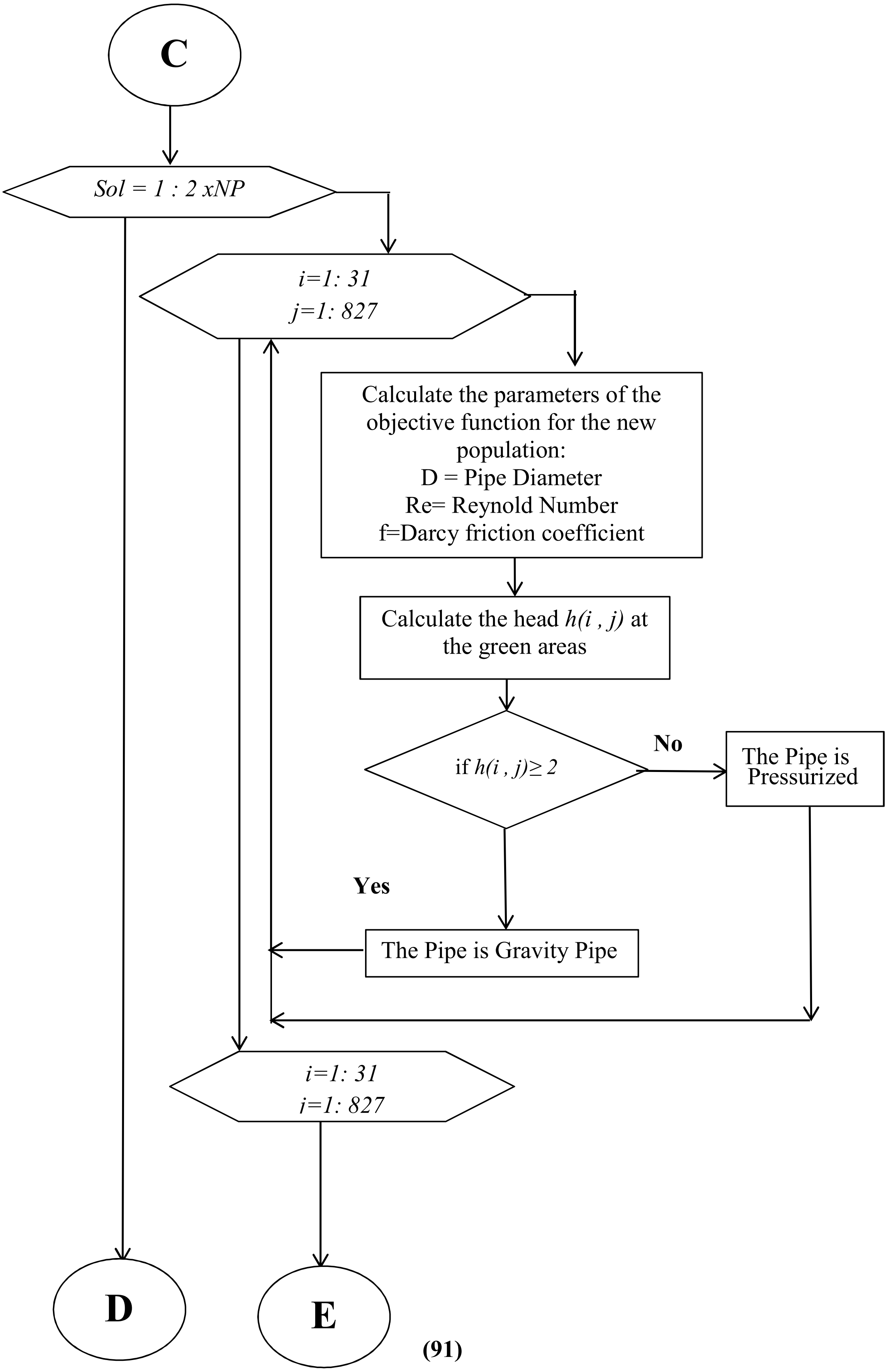
c. The Matlab Programing

The transportation model and the *GA* are implemented by using Matlab 2018a software program. The details of the program are illustrated in the flowchart as shown in Fig. (4.22). Below are some clarifications related to the program and the flowchart:

1. **Data Input:** data input in the flowchart is related to the pump properties that were mentioned in Eqs.(4.15) and (4.24). Moreover, data of the elevation differences, results of pipes lengths, the available sewage flow at each optimized nominated area, and the demands of the green areas.
2. **The Pipe Lengths :** In the program the lengths of pipes are represented in a matrix form $L(i , j)$ with dimensions equal to L [*31 x 827*] , and each value in the matrix represents the length of the pipe from the specified DWWTUi to the green area GRj. For cells that has no pipe links a value of 100,000 was allocated in the program which will give a high cost and it will be neglected automatically from the results.
3. **The Elevation Differences:** Elevation differences are represented in a matrix form $ELD(i , j)$ with dimensions equal to L [*31 x 827*], and each value in the matrix represents the elevation difference between the locations of the DWWTUs i and green area j.
4. **The Available Flow:** It was represented by a one dimensional matrix form $Q_s(i)$, the size of the matrix is equal to 31. Each value in the matrix represents the available flow at each DWWTUs location i.
5. **The Green Area's Demand**: It is represented by a one dimensional matrix form $Q_d(j)$, the size of the matrix equal to 827. Each value in the matrix represents the demand of each green area j.


Start
Input Data
Creating the Random Matrix $Q[31 \times 827]$ for each Solution Np
$i = 1: NP$
Next
Satisfying *Constraint-1* for all solutions by making $\sum_{i=1}^{31} Q_{ij} = b_j$, $j = 1,2,\ldots,827$
Constraint-2 Satisfied
No
Yes
$N_f = N_f + 1$
$NP = N_f$
Specify solutions that satisfied Constraints 1 and 2
A


A
Iteration No. It
PCO No.
i = 1: It
B
i = 1 : 2: NP
Crossovering
Create Offspring
New Population is the Parents and the Offspring
No
Constraint-2
Satisfied
Yes
dd= dd+1
NP= dd
C


C
Sol = 1 : 2 xNP
i=1: 31
j=1: 827
Calculate the parameters of the objective function for the new population:
D = Pipe Diameter
Re= Reynold Number
f=Darcy friction coefficient
Calculate the head h(i , j) at the green areas
if h(i , j)≥ 2
No
The Pipe is Pressurized
Yes
The Pipe is Gravity Pipe
i=1: 31
i=1: 827
D
E

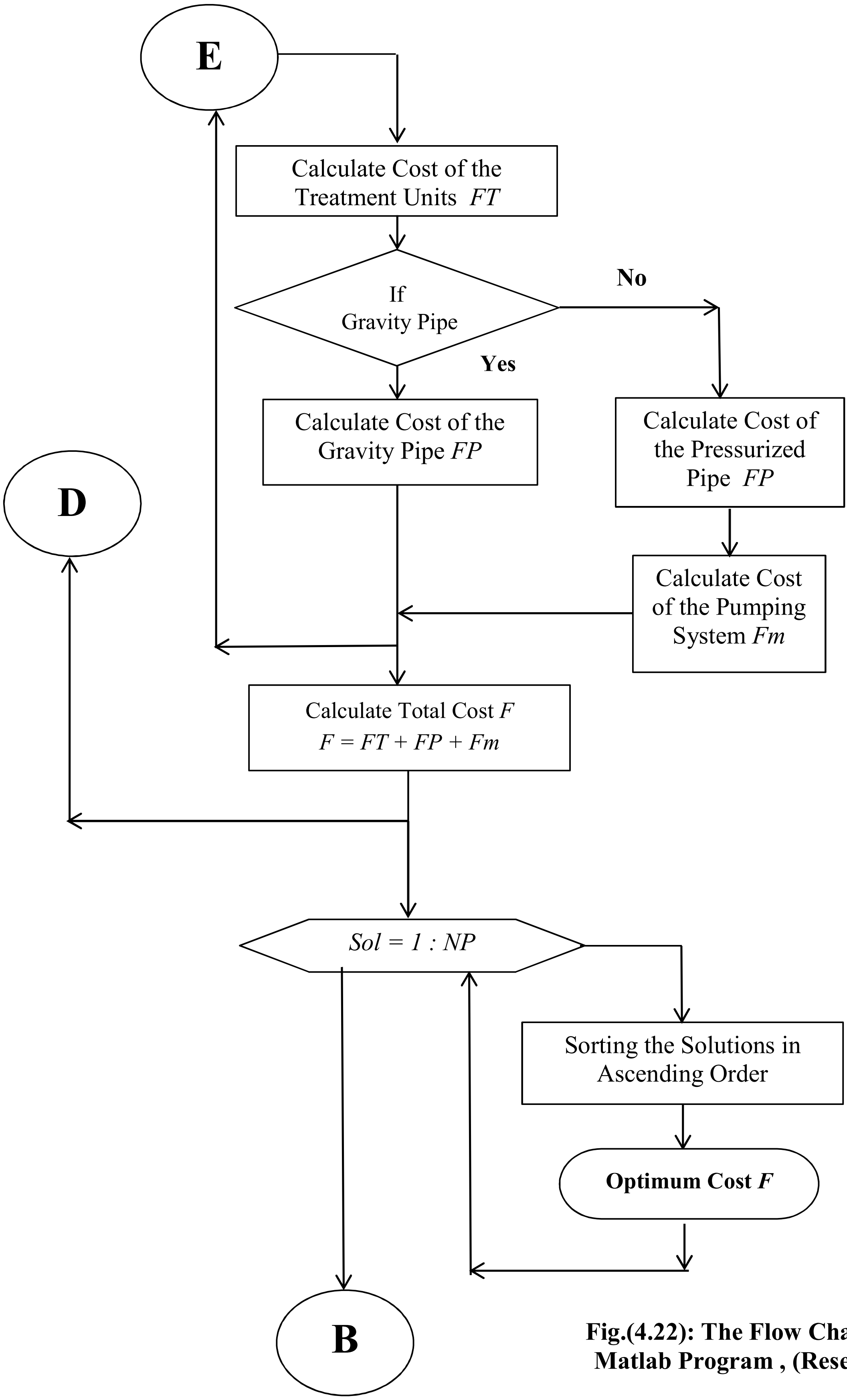


Fig.(4.22): The Flow Chart of the Matlab Program , (Researcher)

CHAPTER FIVE

RESULTS AND DISCUSSIONS

Chapter Five
Results and Discussion

5.1 The AHP

The results of the weights of the suitable criteria using AHP method shows that the Wi of the size of the nominated area's factor has the largest effect which is equal to 35 % and that was expected as obtaining lands inside a city like Sulaimania is very crucial and difficult. The other results are shown in Table (5.1):

Table (5.1): The weight (Wi) of the Five Criteria, (Researcher)

Suitability Criteria	Weigh (W) , in %
The Size of the Nominated Area	35
Distance to the GRs	21
Slope	10
Population Density	16
Depth of the Sewer Box	18

Consistency Ratio (CR) Checking

To find if the judgment was correct or it is far from reality , Consistency Ratio (CR) was found by applying Eqs.(3.4) and (3.5) as in below:

$$\lambda = (35\% \times 2.83) + (21\% \times 5) + (10\% \times 10) + (16\% \times 6.5) + (18\% \times 5.5)$$

$$\lambda = 5.073$$

$$CI = \frac{(\lambda - m)}{(m-1)} \quad = CI = \frac{(5.073 - 5)}{(5-1)} = 0.01829$$

For m = 5, RI =1.12, (table 3.2) ;

$$CR = \frac{CI}{RI} = \frac{0.01829}{1.12} = 1.63\ \%$$

Since CR is equal to 1.63 % (less than 10 %), it is an acceptable value and that means that the judgment of criterion's ranking was correct.

5.2 Suitability Model

The results of the suitability model classified the selected 134 nominated areas into 6 suitability ranks each of them has more than one suitability value as shown in Tables (A.5a) to (A.5j) in appendix A. The reason that the areas having more than suitability class is that each area effected by the six criteria together and in a different weighted values in addition to the restriction factors as well. Table (5.2) shows the suitability results of nominated areas NA5 and NA6. Fig. (5.1) shows the suitability classification results of nominated areas NA1, NA2, NA3, NA4, NA5, NA6, NB3, NB4, and NB5.

Table (5.2): The Suitability Results of Nominated Areas NA5 and NA6, (Researcher)

Classifications	**Areas in m^2**		**Area %**	
	NA5	**NA6**	**NA5**	**NA6**
R = Restricted	726	3,085	6.14	40.0
M.S = Moderately Suitable	0.00	0.00	0.00	0.00
S = Suitable	0.00	0.00	0.00	0.00
V.S =Very Suitable	111	1,311	0.94	17.0
H.S.= Highly Suitable	10,978	1,774	92.92	23.0
E.S= Extremely Suitable	0.00	1,542	-	20.0
Total Area of each Nominated area	**11,815**	**7,712**	**100%**	**100%**

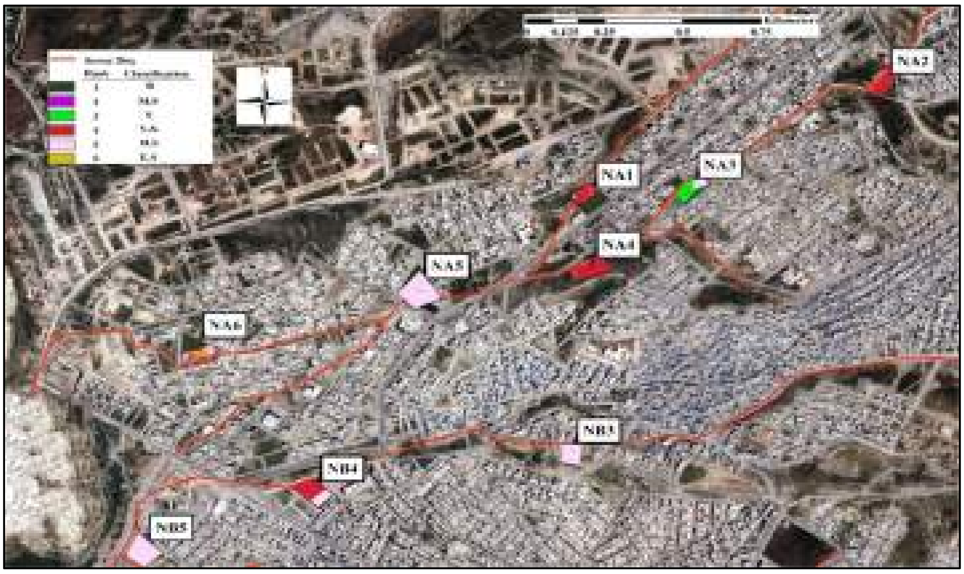


Fig.(5.1) Suitability Results of Nominated Areas
NA1, NA2, NA3, NA4, NA5, NA6, NB3, NB4, and NB5, (Researcher)

To select the optimum nominated areas, the weighted average value (WAV) of each nominated area is found by applying Eq. (5.1) (Anderson, 2013, p. 267):

$$WAV = \frac{(R \: x \: 0.0) + (M.S \: x \: 0.2) + (S \: x \: 0.4) + (V.S \: x \: 0.6) + (H.S \: x \: 0.8) + (E.S \: x \: 1.0)}{3} \tag{5.1}$$

The amount of WAV of each nominated area is normalized by applying Eq.(5.2) as shown below:

$$\text{Normalized WAV} = \text{NWAV} = \frac{(\text{WAV- min})}{(\text{max - min})} \tag{5.2}$$

Where:

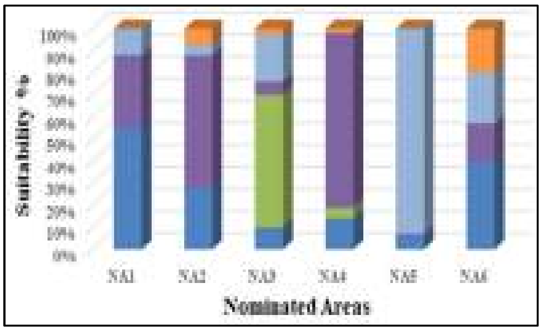
min, max; minimum and maximum value of WAV of nominated areas located on each sewer box

The results of the NWAV of each nominated areas are shown in table (A.6) in appendix A, and Table (5.3) shows the results of the NWAV of nominated areas of sewer box group A.

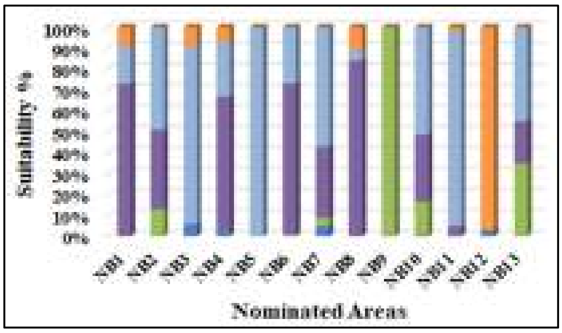
Table (5.3) : The Normalized WAV of Nominated Areas Group A, (Researcher)

Nominated Areas	NA1	NA2	NA3	NA4	NA5	NA6
WAV %	10	17	16	17	25	16
NWAV	0.00	0.47	0.39	0.48	1.00	0.41

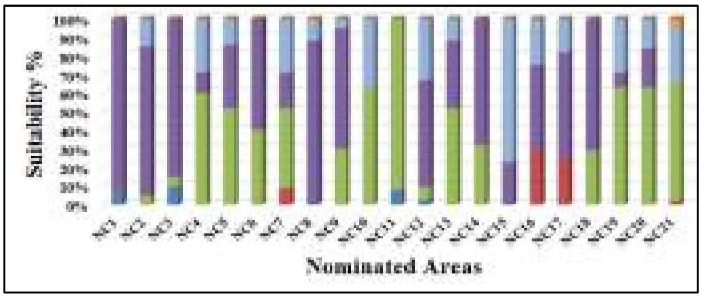
Figs.(5.2a) to (5.2j) show the suitability classifications of nominated areas' groups NA, NB, NC, ND, NE, NF, NG, NH, NI and NJ respectively.



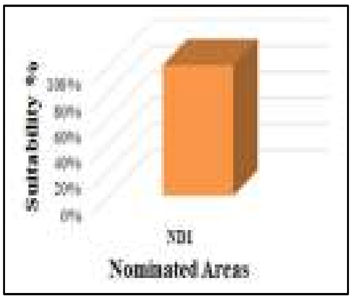
(a)



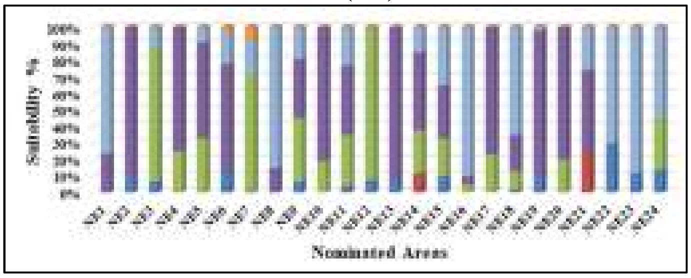
(b)



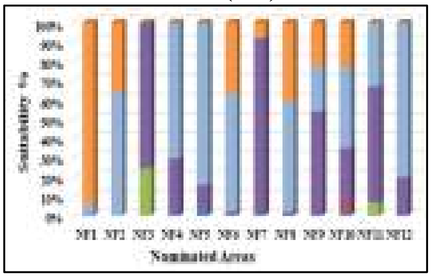
(c)



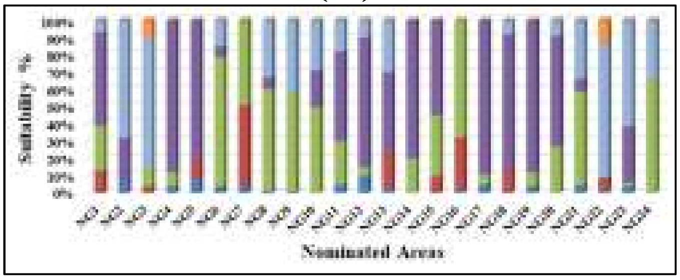
(d)



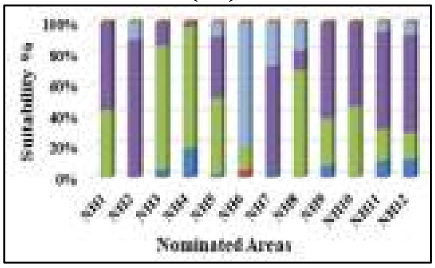
(e)



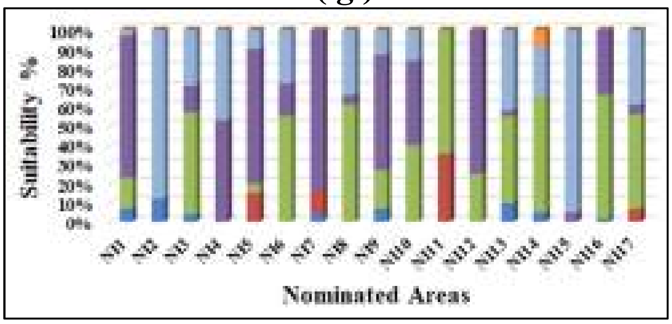
(f)



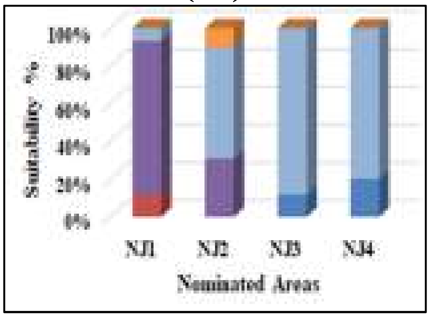
(g)



(h)



(i)



(j)

Extremly Suitable ▪ Very Suitable ▪ Moderately Suitable
Highly Suitble ▪ Suitable ▪ Restricted

Figs.(5.2): Suitability Classifications of the Nominated Areas on lines: (a) Line A, (b) Line B, (c) Line C, (d) Line D, (e) Line E, (f) Line F, (g) Line G, (h) Line H , (i) Line I and (j) Line J, (Researcher)

From the results of the final suitability of the GIS for each nominated area (134 areas), NWAV is calculated. The values of NWAV reflect the level of suitability of the location to be used for installing the DWWTUs. For instance, the NWAV of nominated area NC12 is calculated as in below;

Total area of NC12 is equal to 3,144 m^2 and the suitability classifications are; R = 80.65 m^2 , S = 229.63 m^2 , V.S = 1,792.77 m^2 , H.S = 1,041m^2 and has no other classification levels (M.S = 0 and E.S. = 0).

R % = (80.65/3,144) x 100 = 2.56 %, M.S % = (0.0/3,145) x 100 = 0.0 %,
S% = (229.6/3,144) x 100 = 7.3 %, V.S%. = (1,792.77 /3,144) x 100= 57%,
H.S.% = (1,041/3,144) x 100 = 33.1 %, E.S.% = (0.0/3,145) x 100 = 0.0 %.

Substitute into Eq.(5.1);

$$\text{WAV}= \frac{(2.6 \times 0.0) + (0.0 \times 0.2) + (7.3 \times 0.4) + (57 \times 0.6) + (33 \times 0.8) + (0.0 \times 1.0)}{3}$$

WAV =21%, the minimum value of WAV of sewer box line C = 12% and the maximum value is 25%, substitute into equation (5.2);

$$\text{NWAV} = \frac{(21 - 12)}{(25 - 12)} = 0.71$$

The values of NWAV are ranged from 0.0 to 1.0 with an average of 0.5. The optimum locations from the 134 nominated areas are the areas that have the highest NWAV. Many reference points tried starting from 0.40, 0.45, 0.50, 0.55, 0.60, to 1.0 and it is found that the number of nominated areas that having NWAV ≥ 0.45 is 92 and that will be a big number and it is also not practical, while 31 nominated areas have NWAV ≥ 0.5 and that seems to be a reasonable number. Table(5.4) shows the results of the optimized 31nominated areas and table (A.7) in appendix A shows the results of NWAV of areas. The final 31 optimum nominated areas are distributed in organized and strategical positions in the study area and are located over the 10 main sewer box lines. The number of the selected areas per each sewer box is ranged from one to five. Line A has only one suitable area as the preliminary selected areas from the beginning was only 6 areas, because line A is short and covers small parts of the city's districts. Figs.(5.3a), (5.3b) and (5.3c) shows the 31 optimum locations of the proposed DWWTUs.

Table (5.4): Values of NWAV of the 31 Optimized Nominated Areas, (Researcher)

Optimized Nominated Area	Line	NWAV	Optimized Nominated Area	Line	NWAV
OA1	A	1.00	OF2	F	0.75
OB1	B	0.66	OF3		0.76
OB2		0.71	OF4		0.73
OB3		0.70	OG1	G	0.74
OB4		1.00	OG2		0.84
OC1	C	0.67	OG3		0.87
OC2		0.68	OG4		0.73
OC3		0.71	OH1	H	0.75
OC4		1.00	OH2		0.97
OD1	D	1.00	OH3		0.77
OE1	E	0.93	OI1	I	0.83
OE2		0.98	OI2		0.81
OE3		0.98	OI3		1.00
OE4		0.80	OJ1	J	1.00
OE5		0.82	OJ2		0.82
OF1	F	1.00			

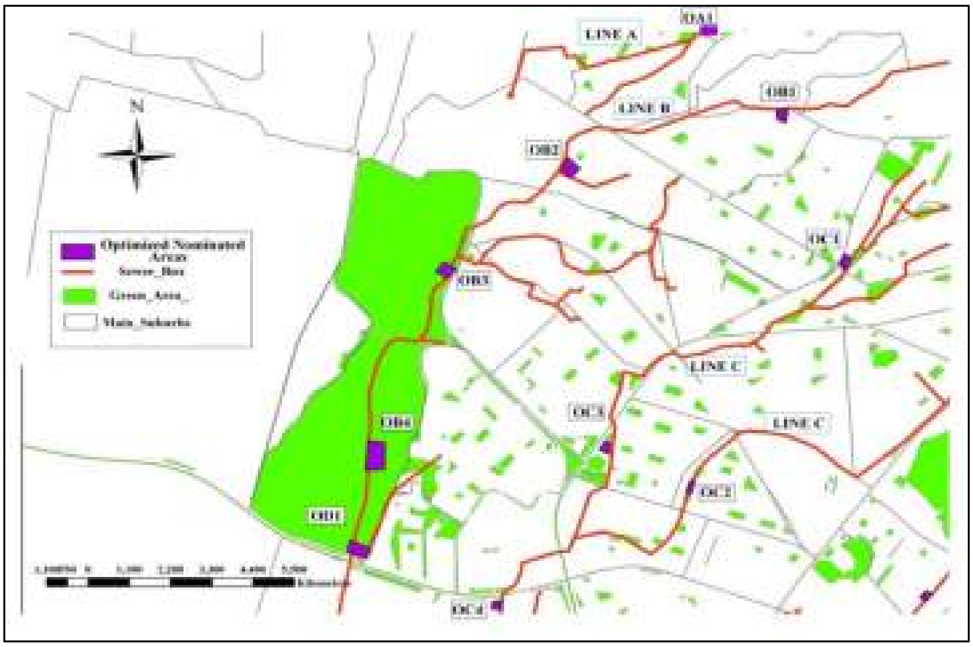


Fig. (5.3a):The Final Optimized Suitable Nominated Areas on Lines A, B, C and D, (Researcher)

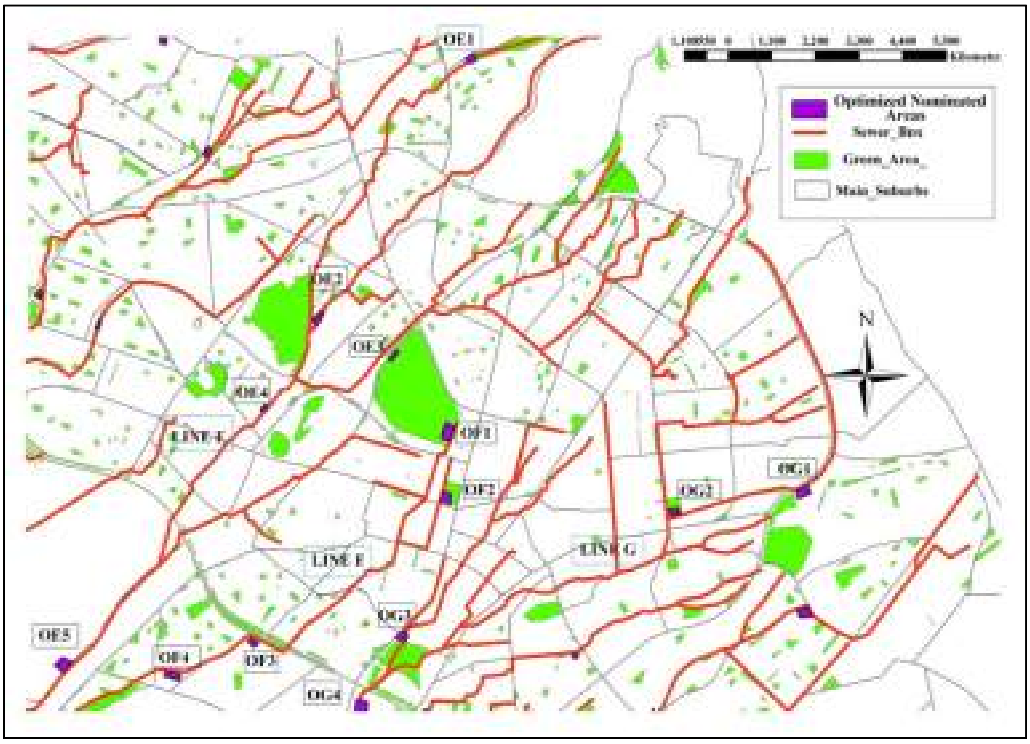


Fig.(5.3b): The Final Optimized Suitable Nominated Areas on Lines E, F, and G , (Researcher)

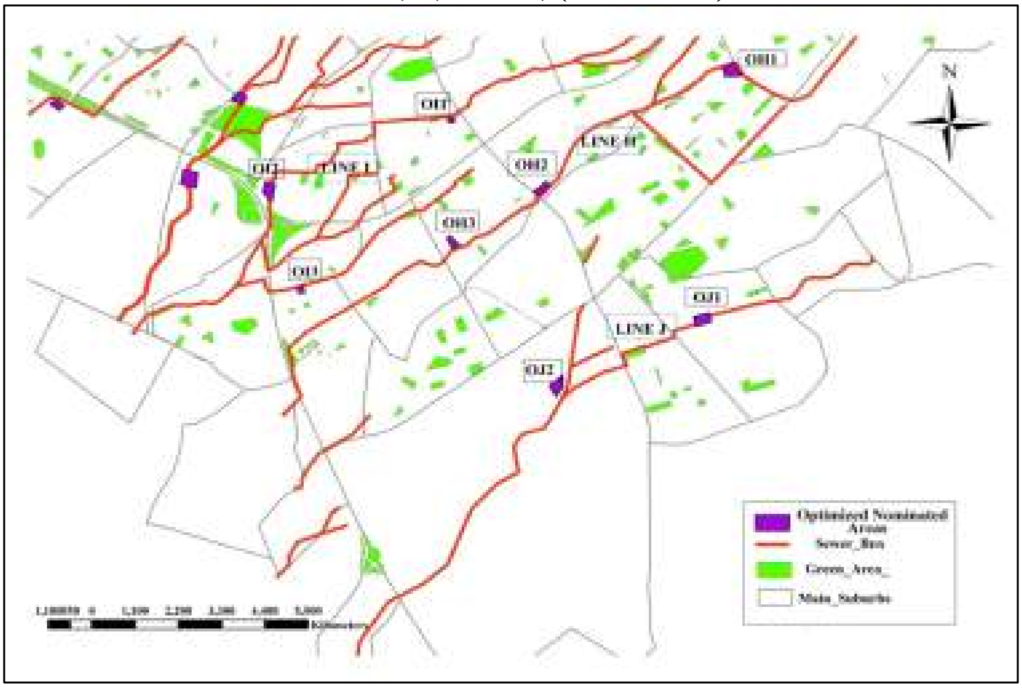


Fig.(5.3c): The Final Optimized Suitable Nominated Areas on Lines H, I and J, (Researcher)

5.3 The Network Analysis – OD Cost Matrix

The results of the GIS network analysis produced 603 pipes from the DWWTUs to the green areas. Not all of the green areas are connected with the DWWTUs as some of GRs are out of the cutoff path (1000 m). Other cutoff distance used in the program, such as 1,250 m and 1,500 m. The results did not show obvious changes as the additional connected GRs have small green area sizes with longer pipe lengths. Fig. (5.4) shows the paths (blue lines) from OI2 and OG4 to the green areas within the cutoff route. The lines from the optimized nominated areas to each green area represent the supplying pipes from the DWWTUs. The details are shown in table (A.12) in appendix A.

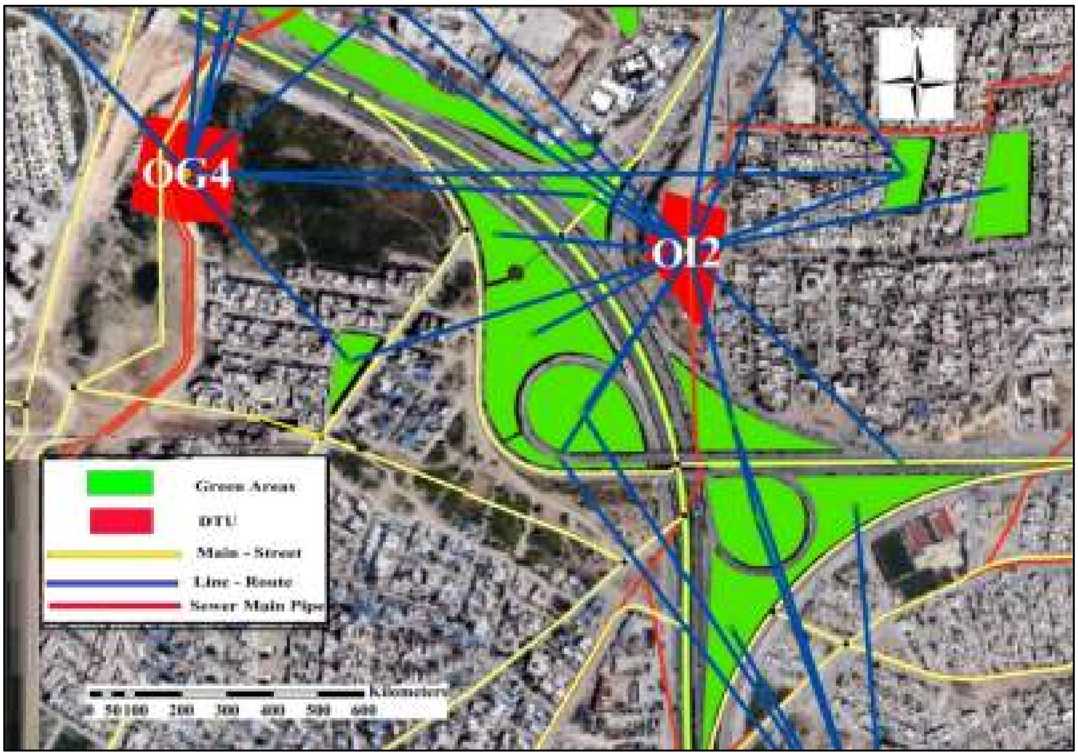


Fig.(5.4): The Results of the Network Analysis – OD Cost Matrix of Optimized Nominated Areas OI2 and OG4, (Researcher)

Most of DWWTUs linked to a significant number of green areas such as; OE1 connected to 33 green areas, and OE17 connected to 34 green areas. Practically it is not applicable to set out this big number of pipes from one treatment plant. To solve the issue, green areas that connected to each DWWTU organized into groups. Each group shares a storage tank T2 to receive treated water from that DWWTU. The conveying pipes connect the DWWTUs and the storage tank T2 of each group of green areas. As a result, the number of pipes reduced from 603 lines to 159 main pipes. For instance, treatment unit OC3 connected to 25 green areas through 25 pipes. Those pipes are grouped and replaced by 6 main pipes (6 groups of green areas). Figs (5.5) and 5.6 shows the results of conveying pipe layouts of the GIS analysis from treatment unit OC3 before and after grouping respectively. The results of grouping of all pipes are shown in table (A.13) in appendix A. Fig. (B.3) in appendix B shows the grouping map of all green areas of the study area.

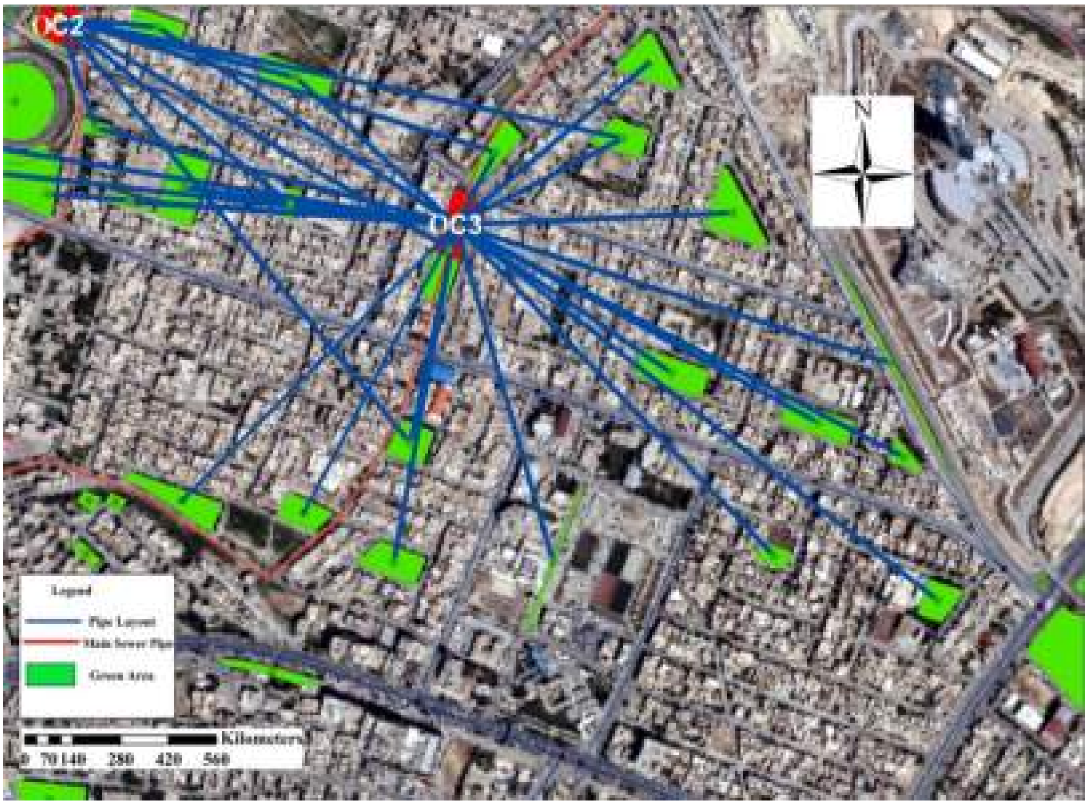


Fig.(5.5): Results of GIS Network Analysis OD – Cost Matrix of Optimized Nominated Area OC3, (Researcher)

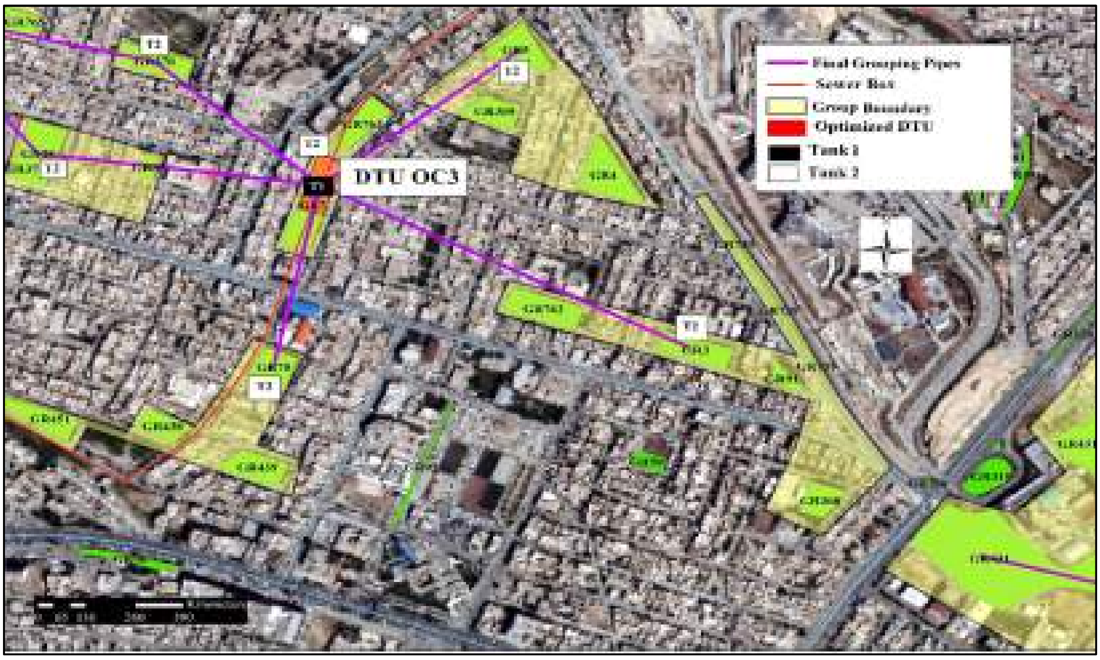


Fig.(5.6): Results of Grouping Conveying Pipes of OC3 Treatment Unit, (Researcher)

The results of the elevations of the 827 green areas' centroid points and the 31 optimized nominated areas centroid points are shown in tables (A.14) and (A.15) in appendix A respectively. Fig.(5.7) shows the elevations of part of the study area.

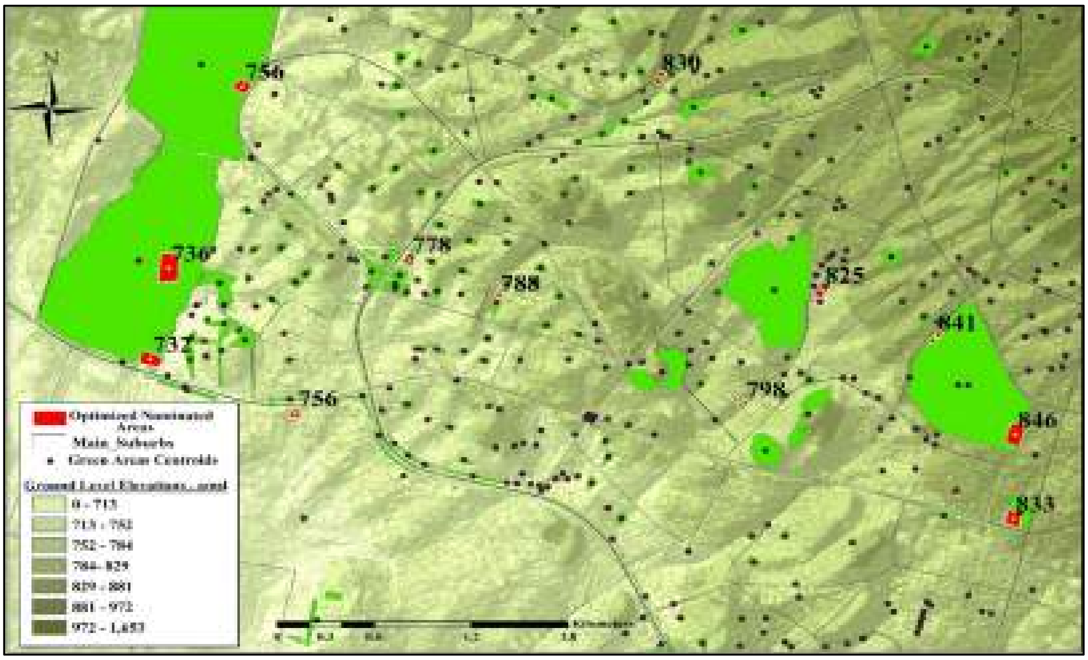


Fig.(5.7): The Elevations of the GR and the Optimized NA, (Researcher)

5.4 The Optimization Model

In general, the model was capable of finding the optimum solution for the DWWTUS sizes and the algorithm was complex in terms of the size of data of the study in compare to previous applications of the genetic algorithm in wastewater management as in this study a whole city was applied.

The model was run with different populations and it was noticed that the costs F_{min} were high for small NPs , moreover, the results were not stable at the beginning. GA method is a random process and the only step for getting the corrects results is testing the stability. Sensitivity analysis was done to achieve the stable solution and find final optimum F_{min} and that was done by fixing the number of NPs and changing the PCO values and running the program three times for each PCO location. For each run four iterations were taking (No. of runs =3 and It = 4) and all the results from each iteration were selected to be used in the mating pool in order to enhance the results. Selecting 100 % of the parents will take more computer running time but it will give better results as it will give a chance for all to participate in the process (Chong, 2013, p. 197).

In the Sensitivity analysis the difference (DR) of the obtained F_{min} values of the runs of the last iteration of each PCO in each NP were taking as in below:

DR1 = F_{min} (of first run) - F_{min} (of second run)
DR2 = F_{min} (of first run) - F_{min} (of third run)
DR3 = F_{min} (of third run) - F_{min} (of second run)1`

The comparisons of the results are based on the amount of DR in which the preferred F_{min} value is for the PCO that gives the smallest DR. For example, the results of F_{min} and DR values of NP = 100 , PCO = 5 and It=4 are shown in Table (5.5) .

Table (5.5): Values of F_{min} in \$ for *NP* = 100, *PCO* = 5, (Researcher)

	Run No.	*It_4*	*DR*
PCO – 5 ***NP = 100***	*Run-1*	21,751,866 \$	Run1 – Run2 = 83,000 \$
	Run-2	21,668,866 \$	Run2 – Run3 = 4,000 \$
	Run-3	21,672,866 \$	Run2 – Run3 = 79,000 \$

From the F_{min} values of stable solution of NPs equal to 100, 200, 300, 400, 500, 600, 700, 800, 900, and 1000 with different values of PCOs and number of iterations equal to four, the followings results and discussions were obtained:

1. It was obvious that the values of DR at NP ≥ 500 are small and that reflects the stability of the results at that point. For populations less than 100 the values of F_{min} are high in compare to the results of NP ≥ 100. Therefore, F_{min} values of NP < 100 are neglected.

2. A number of trials were done for NP =25, 50, and 75 with different PCOs as shown in Fig. (5.8). It is clear that there is wide range of difference between the F_{min} values for instance; for Np = 25, the difference between the minimum value and maximum value of F_{min} = 893,000 $. Moreover, there are big jumps in the results between the PCOs. The reason is that for small NPs the stability is not achieved.

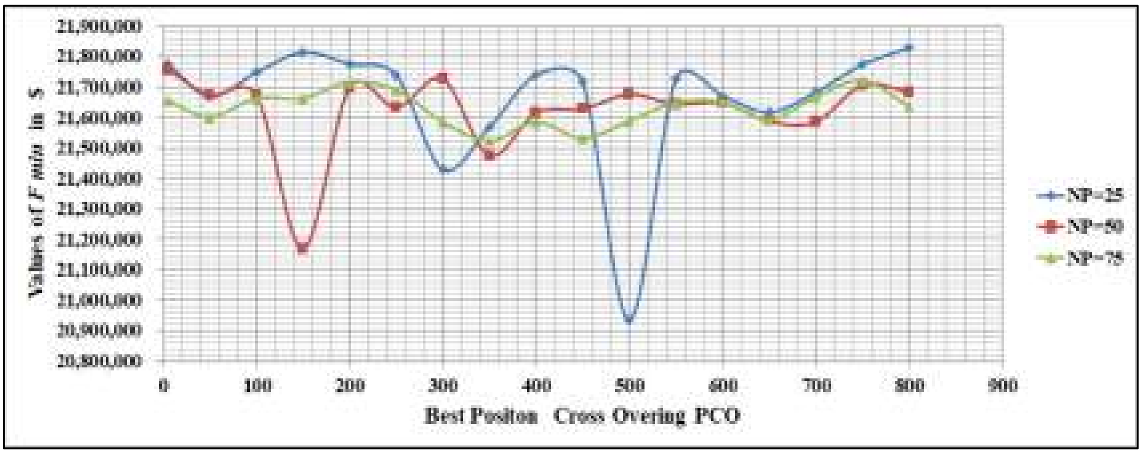


Fig.(5.8): The Values of F_{min} in $ for NP = 25, 50 and 75 , (Researcher)

3. The results of the three runs of the ten NPs and different PCOs showed that the values of F_{min} are ranged from 21,325,000 $ to 21,752,000$ with an average equal to 21,546,000 $. Figs.(5.9a) to (5.9r) show the results of the three runs of all NPs and PCOs starting from 5 to 800 steps 50.

4. According to the sensitivity analysis, the satiability was conducted at NP = 500 and therefore, it was selected for the optimum solution as for population more than 500, high computer running time is required and the results are almost the same.

5. The results mainly affected by the PCO locations for instance, at PCO = 200 for all NP values the Fmin values are high. The reason is that every location of PCO represents a GR position in the map and in the matrix and when the mating of parents occurred at that point the arrangement at that area gave the worst result due to the connection type of the green areas to the DWWTUs.

6. After the first run, only the solutions that satisfy constraints 1 and 2 will pass and selected for the cross over process. The first constraint was satisfied when developing the random matrix and regarding the second constraint, it was fulfilled through the vertical cross over method. In this way, each GR will receive the required demand. Therefore, it is noticeable that the values of the F_{min} at the PCOs have the same trend.

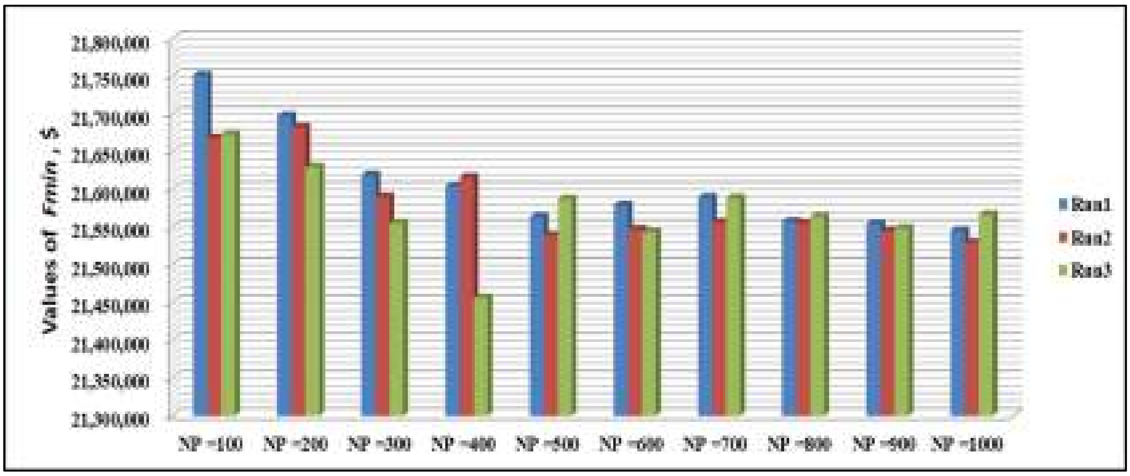


Fig.(5.9a): The Values of F_{min} in $ for all *NPs* and for *PCO* = 5, (Researcher)

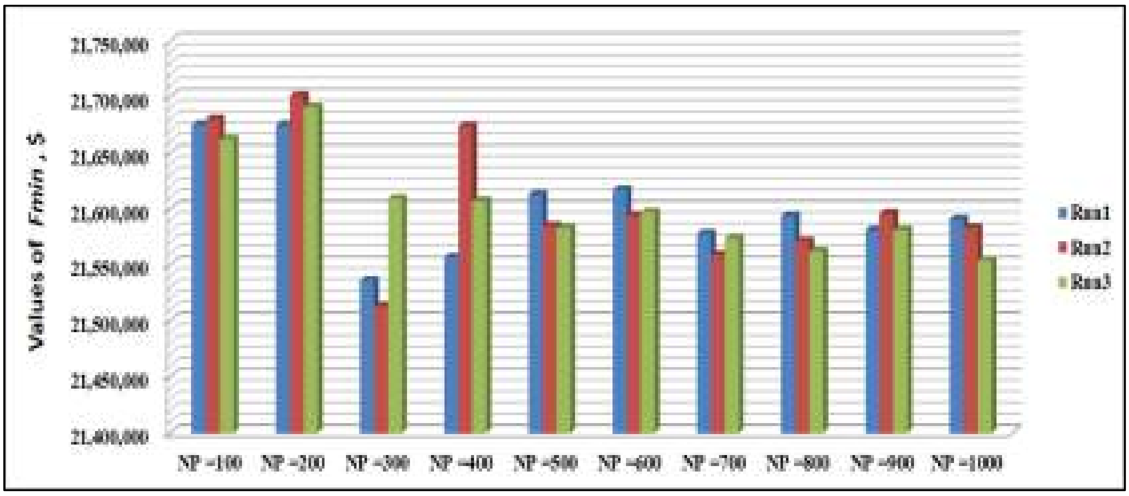


Fig. (5.9b): The Values of Fmin in $ for all NPs and for PCO = 10, (Researcher)

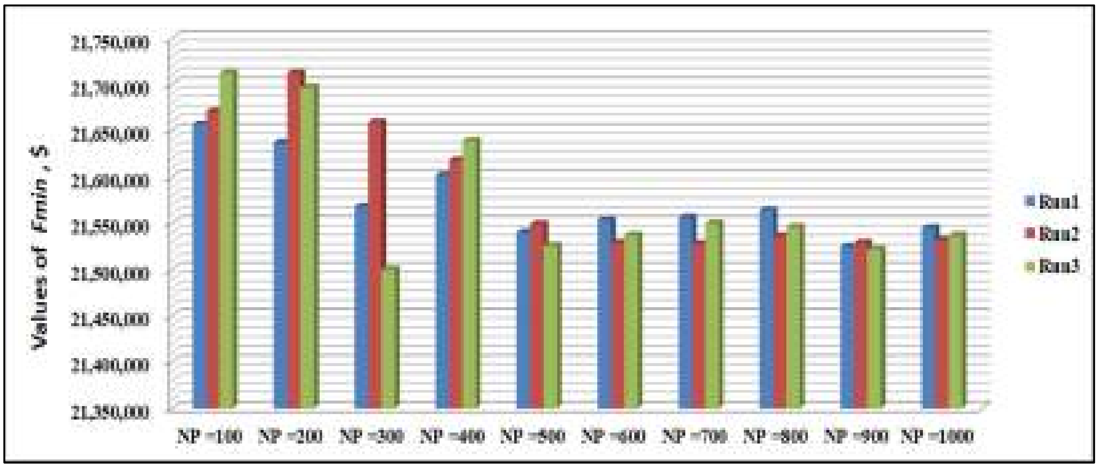


Fig.(5.9c): The Values of Fmin in $ for all NPs and for PCO = 50, (Researcher)

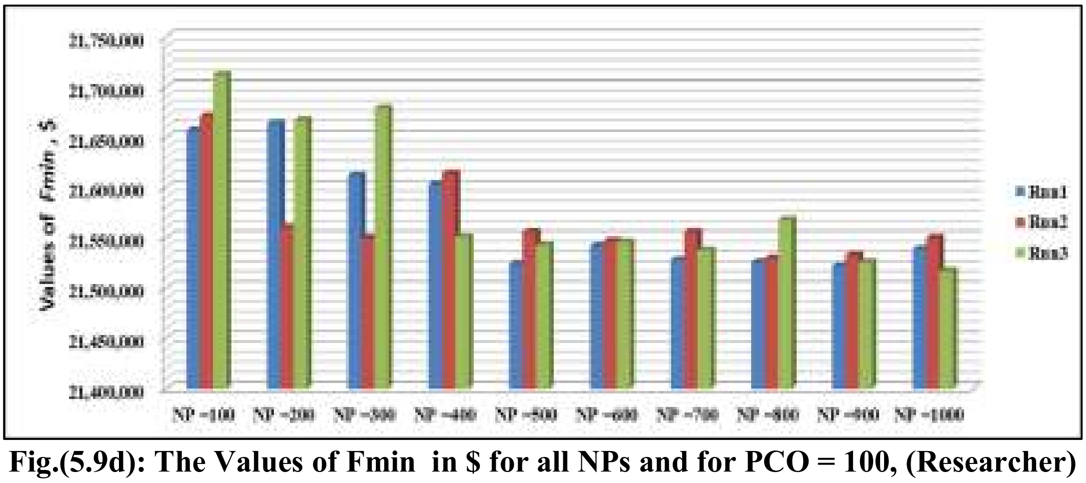


Fig.(5.9d): The Values of Fmin in $ for all NPs and for PCO = 100, (Researcher)

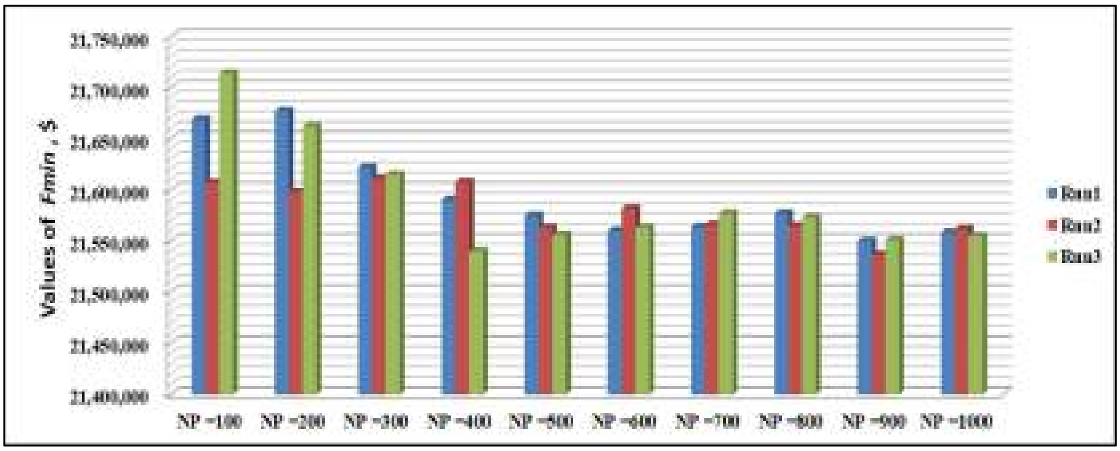


Fig.(5.9e): The Values of Fmin in \$ for all NPs and for PCO = 150, (Researcher)

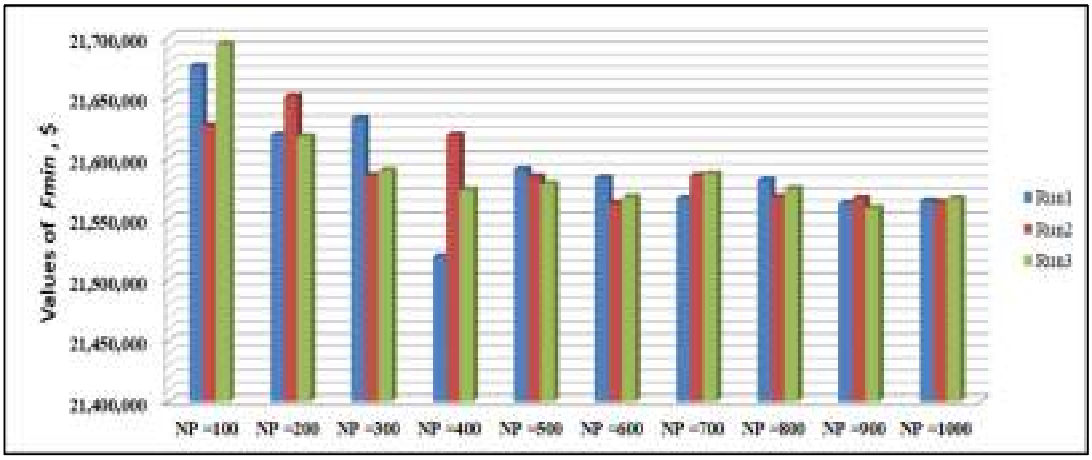


Fig.(5.9f): The Values of Fmin in \$ for all NPs and for PCO = 200, (Researcher)

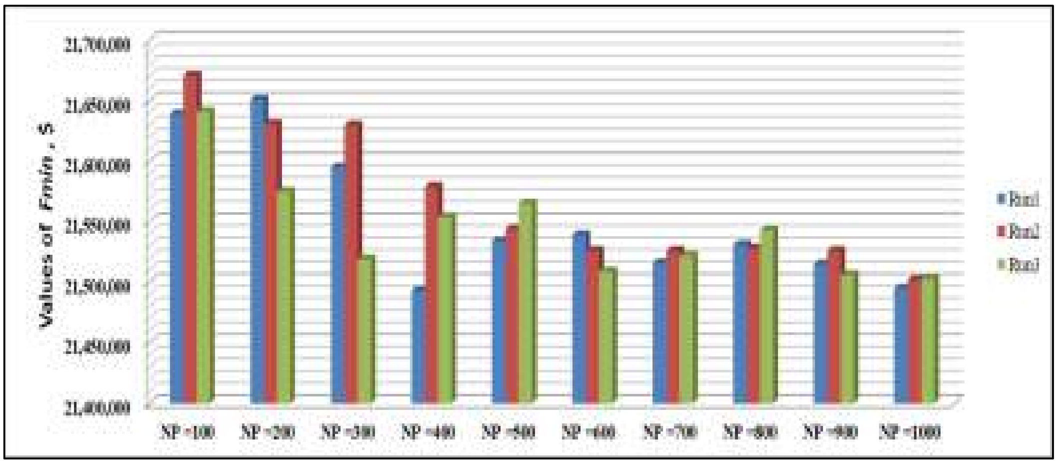


Fig.(5.9g): The Values of Fmin in \$ for all NPs and for PCO = 250, (Researcher)

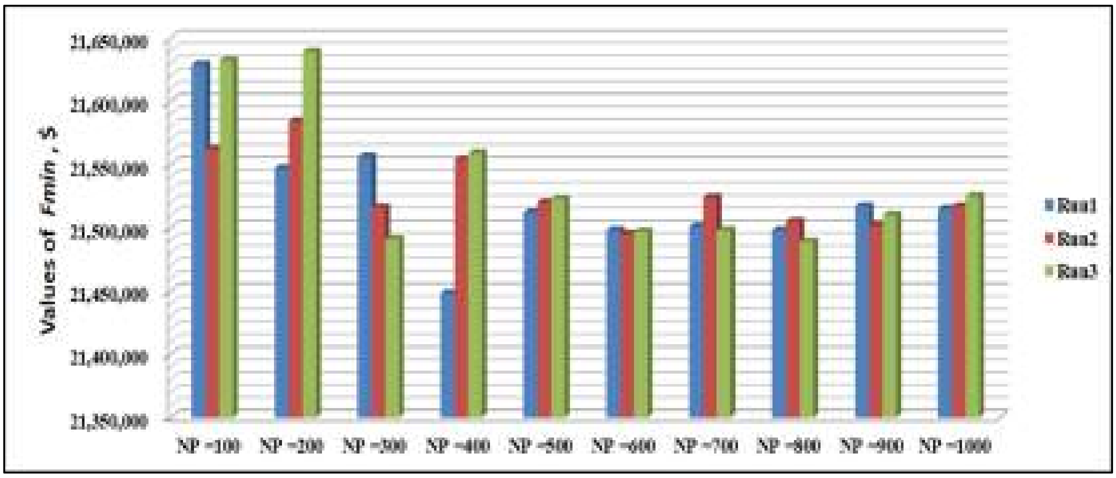


Fig.(5.9h): The Values of Fmin in $ for all NPs and for PCO = 300, (Researcher)

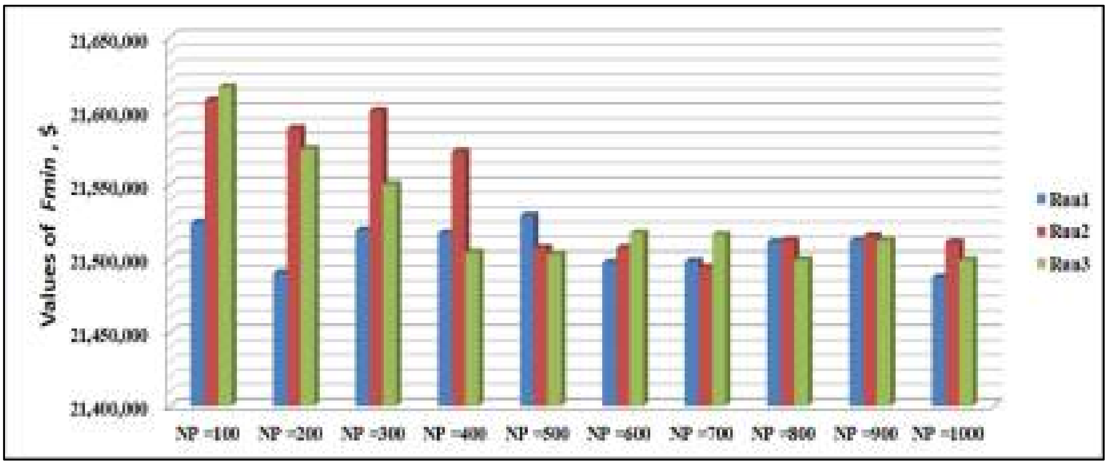


Fig.(5.9i): The Values of Fmin in $ for all NPs and for PCO = 350, (Researcher)

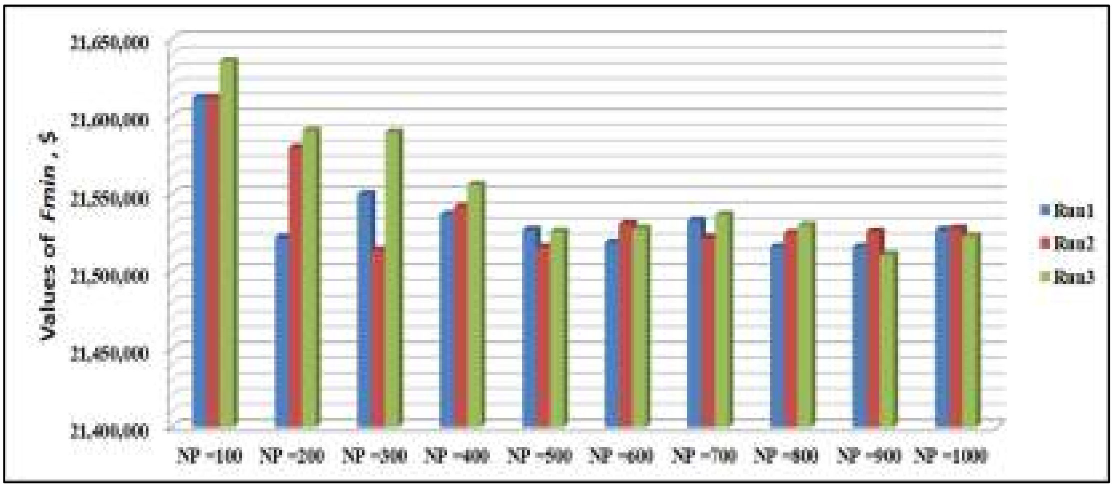


Fig.(5.9j): The Values of Fmin in $ for all NPs and for PCO = 400, (Researcher)

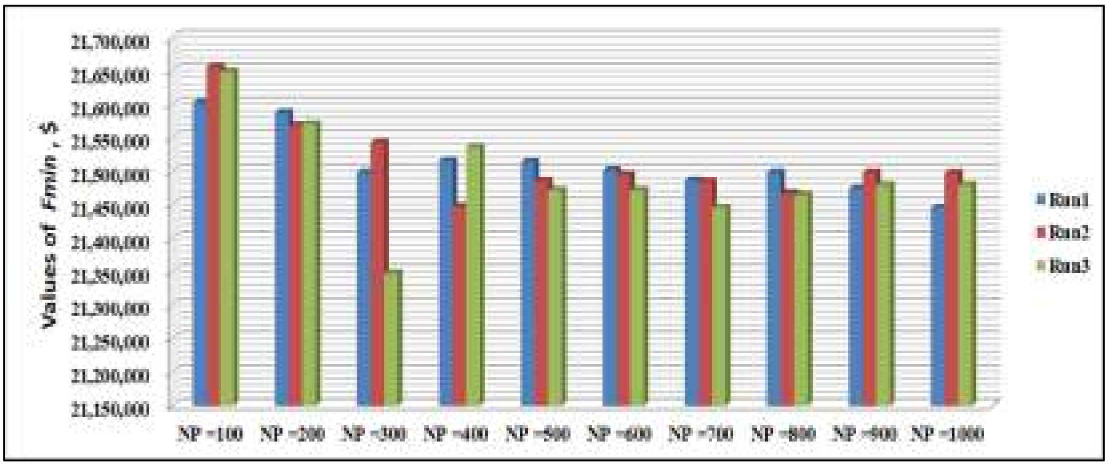


Fig.(5.9k): The Values of Fmin in $ for all NPs and for PCO = 450, (Researcher)

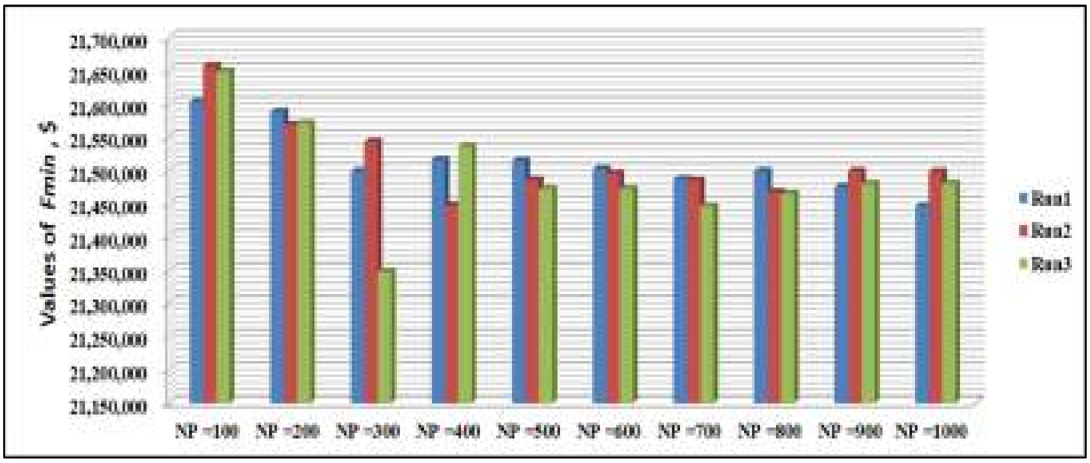


Fig.(5.9l): The Values of Fmin in $ for all NPs and for PCO = 500, (Researcher)

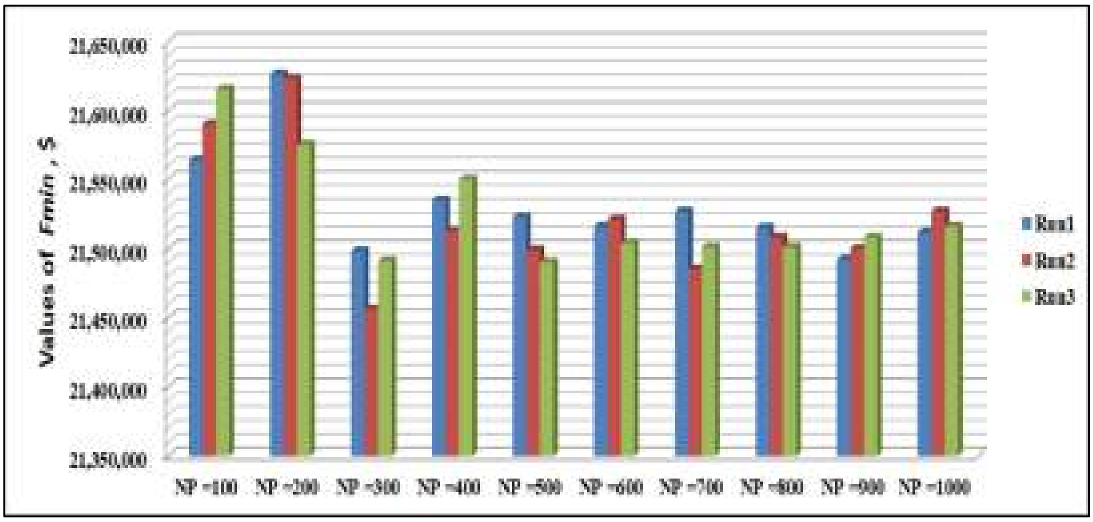


Fig.(5.9m): The Values of Fmin in $ for all NPs and for PCO = 550, (Researcher)

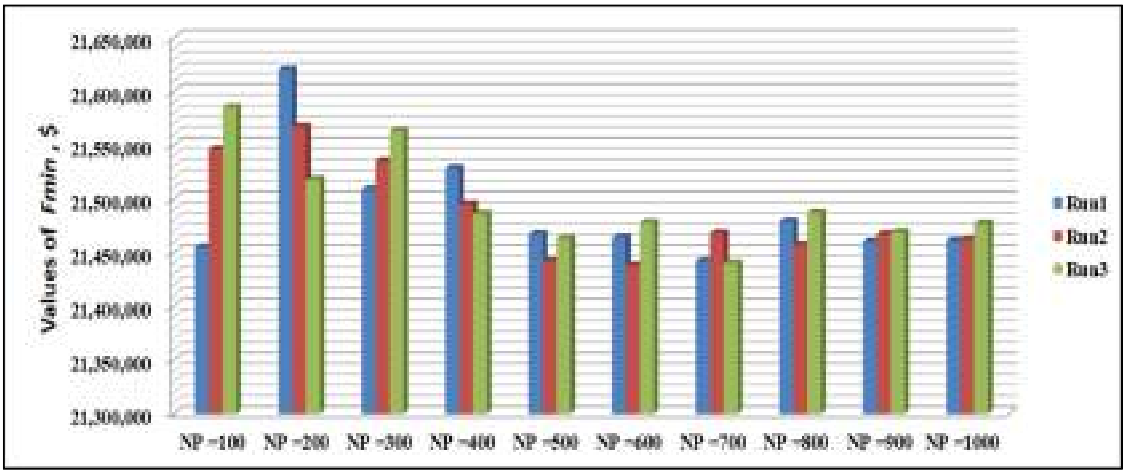


Fig.(5.9n): The Values of Fmin in $ for all NPs and for PCO = 600, (Researcher)

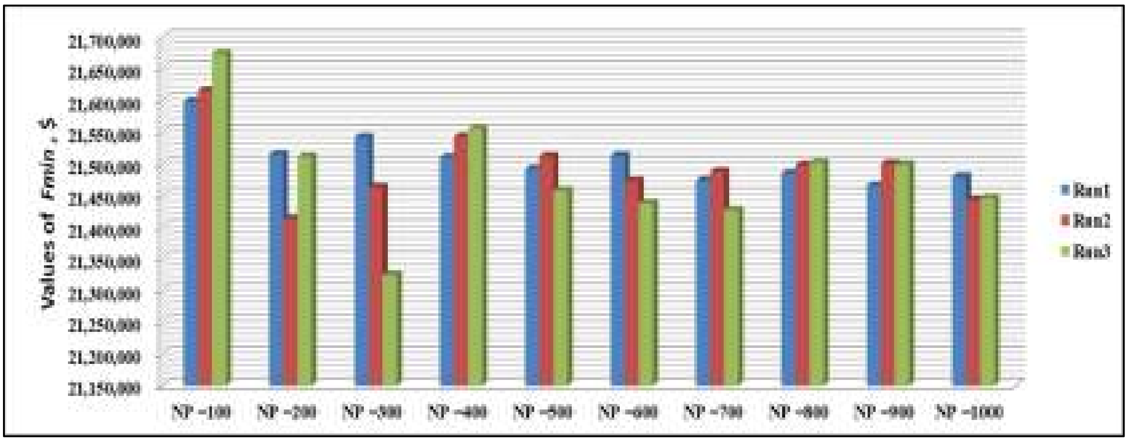


Fig.(5.9o): The Values of Fmin in $ for all NPs and for PCO = 650, (Researcher)

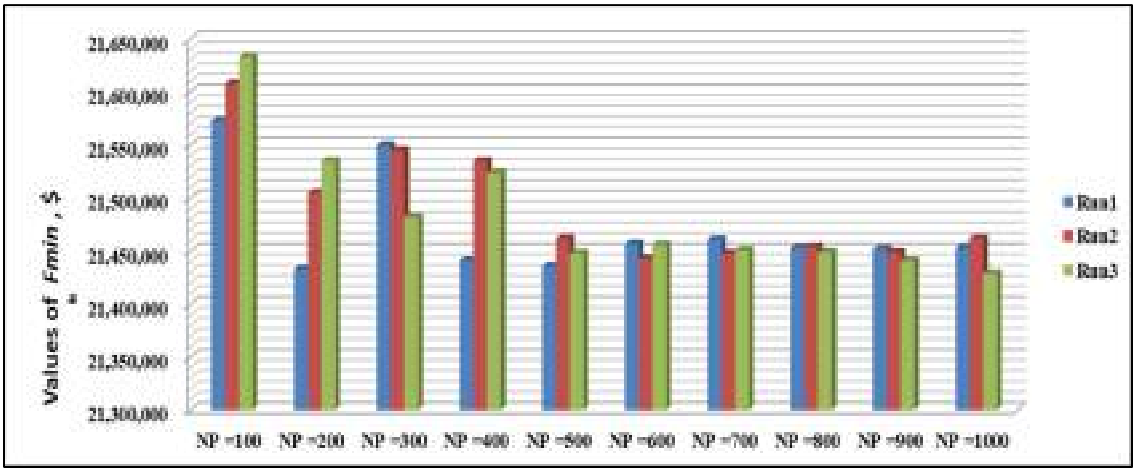


Fig.(5.9p): The Values of Fmin in $ for all NPs and for PCO = 700, (Researcher)

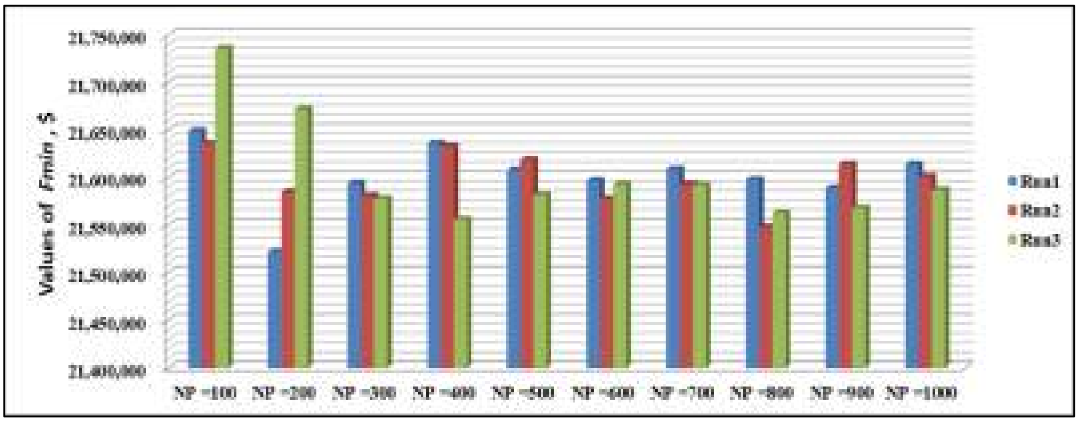


Fig.(5.9q): The Values of Fmin in $ for all NPs and for PCO = 750, (Researcher)

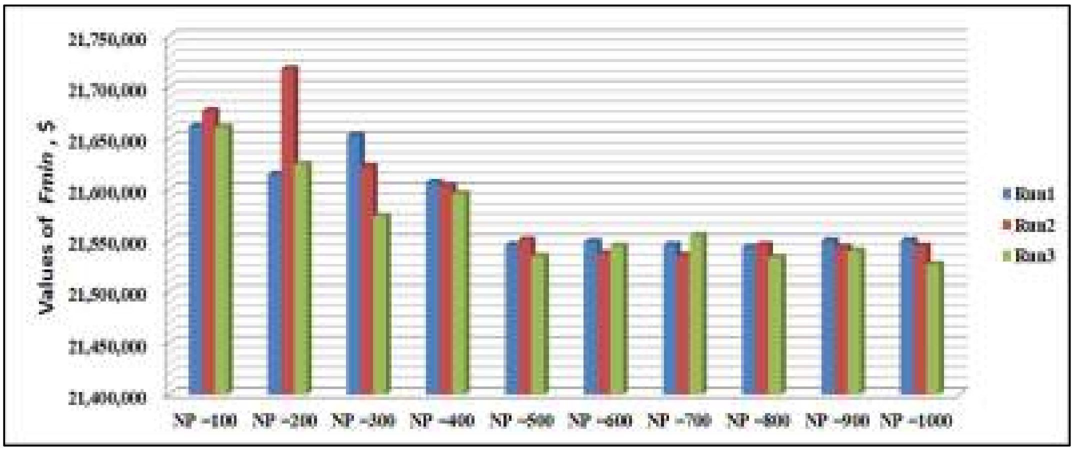


Fig.(5.9r): The Values of Fmin in $ for all NPs and for PCO = 800, (Researcher)

7. The numbers of iterations (It) were taken to be 4 and for each iteration three runs are conducted. When increasing It to 5 and 6 the results are similar to the results of iteration number four. The explanation of that is the results of each iteration will be a new population and pass through the check of constraints, mating and cross overing. In this process the results will be improved after passing each iteration and at It = 4 they will reach to their best results and cannot be improved any more at It = 5 and 6. Therefore, to avoid computer running time only four iterations are considered. Table (5.6) shows the results of F_{min} of NP =400, 700 and 1000 with PCOs = 400, 150 and 250 respectively. It is obvious that the results are the same after iteration 3 and in some runs after iteration 4.

Table (5.6) : The Results of (F_{min} x 10^3) of Six Iterations of Selected *NPs* and *PCOs*, (Researcher)

Iterations		It_1	It_2	It_3	It_4	It_5	It_6
NP = 400							
PCO - 400	*Run1*	21,637	1,537	21,537	21,537	1,537	1,537
	Run2	1,585	1,570	21,542	21,542	1,542	21,542
	Run3	1,673	1,612	21,589	1,556	21,556	21,556
NP =700							
PCO - 150	*Run1*	1,669	21,634	21,599	21,564	21,564	21,564
	Run2	1,566	1,566	21,566	1,566	21,566	1,566
	Run3	1,566	21,566	21,566	21,577	21,577	21,577
NP = 1000							
PCO - 250	*Run1*	1,595	21,575	21,495	21,495	21,495	21,495
	Run2	1,620	21,575	21,502	21,502	21,502	21,502
	Run3	1,626	1,560	21,503	21,503	21,503	21,503

8. In order to find the optimum solution further runs are done for NP = 500 by applying more PCOs with steps = 5 and It = 4. In this way more detailed search will be conducted. Figs.(5.10a) to (5.10h) shows F_{min} values of NP = 500 for PCOs = 5 to 825 by steps = 5. It is obvious that the values of F_{min} at the PCOs are very close and the difference between the maximum and minimum values is 196,000 \$ which is small amount.

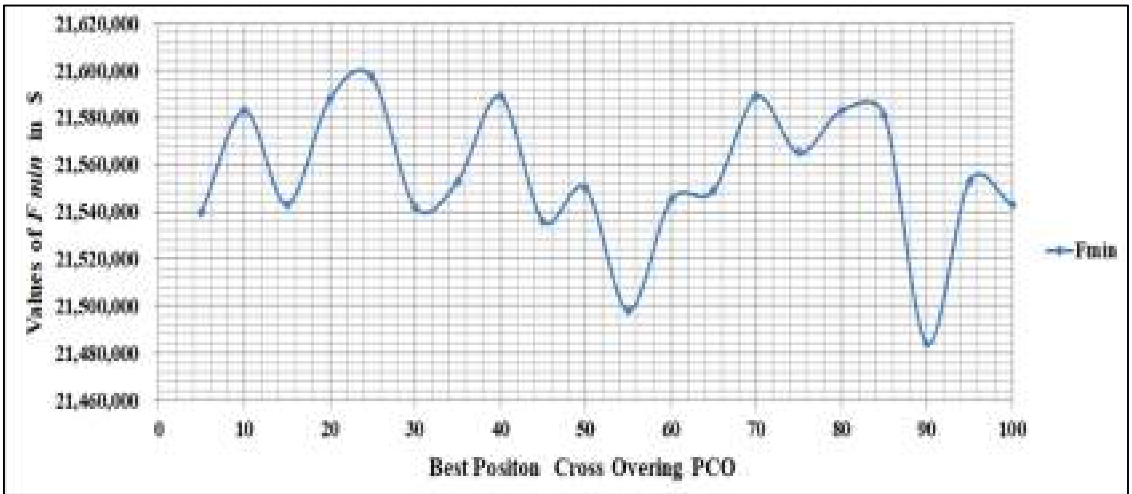


Fig.(5.10a): Values of Fmin of NP = 500 and PCOs of 5 to 100, (Researcher)

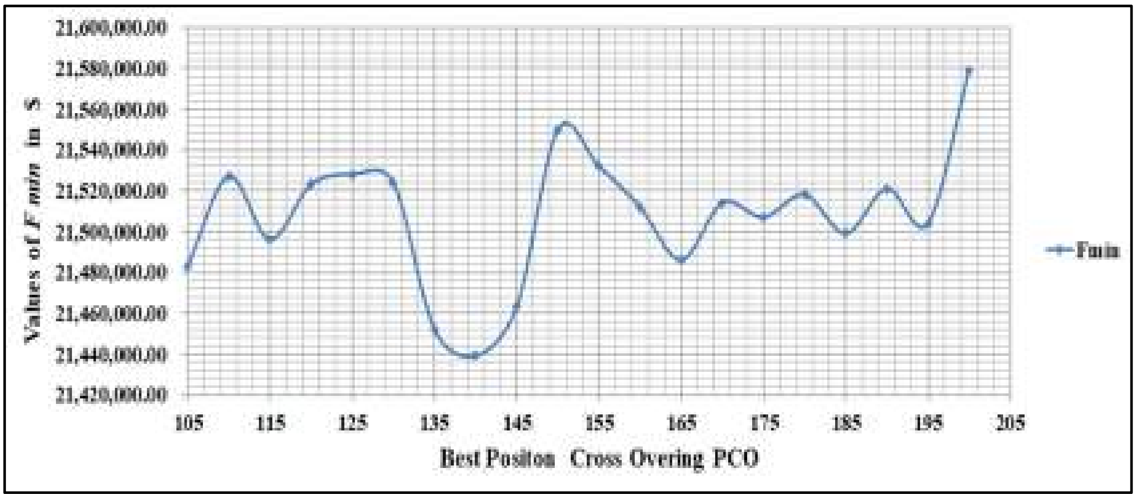


Fig.(5.10b): Values of Fmin of NP = 500 and PCOs of 105 to 200, (Researcher)

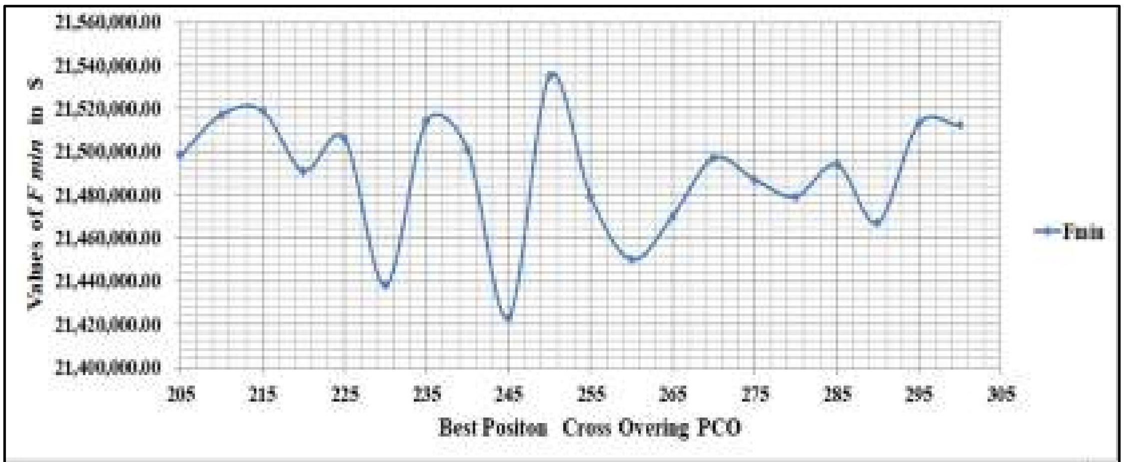


Fig.(5.10c): Values of Fmin of NP = 500 and PCOs of 205 to 300, (Researcher)

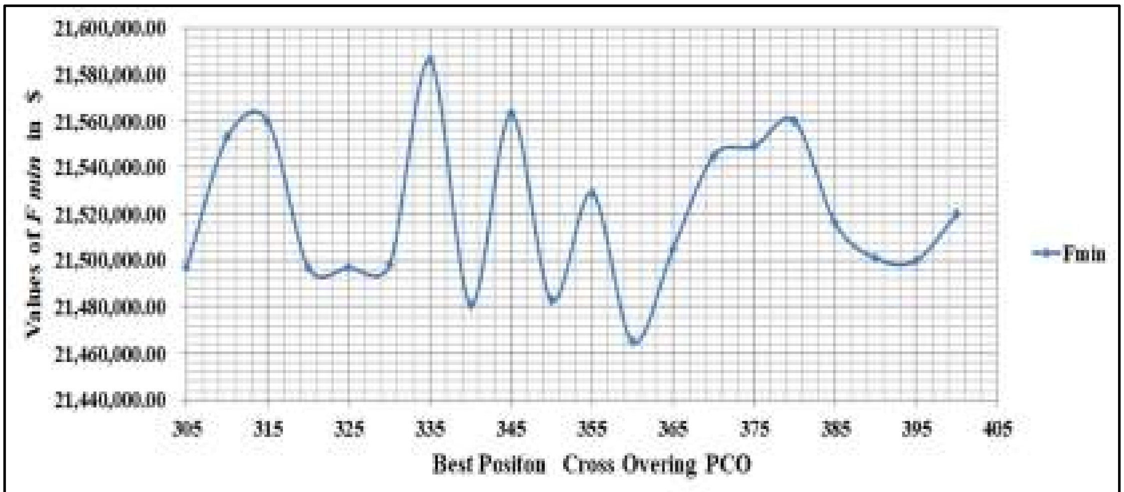


Fig.(5.10d): Values of Fmin of NP = 500 and PCOs of 305 to 400, (Researcher)

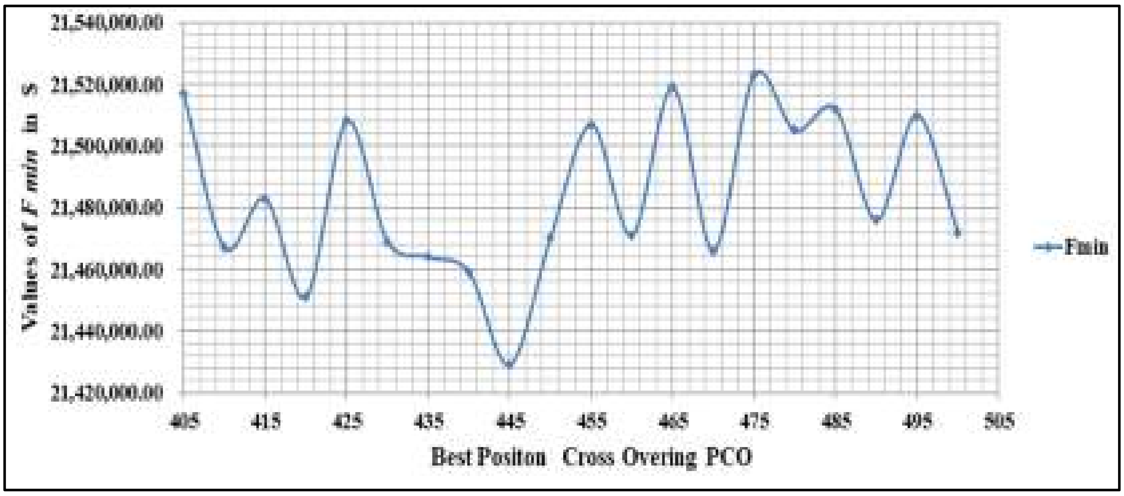


Fig.(5.10e): Values of Fmin of NP = 500 and PCOs of 405 to 500, (Researcher)

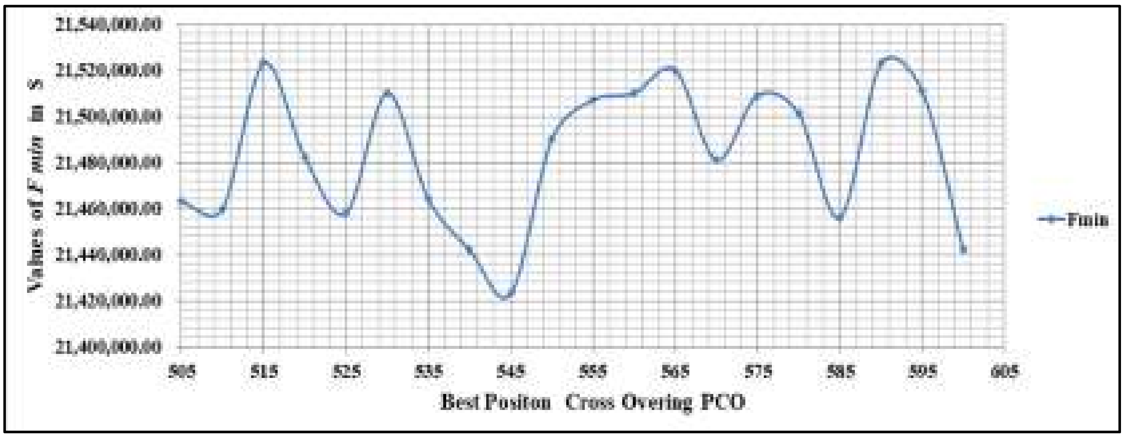


Fig.(5.10f): Values of Fmin of NP = 500 and PCOs of 505 to 600, (Researcher)

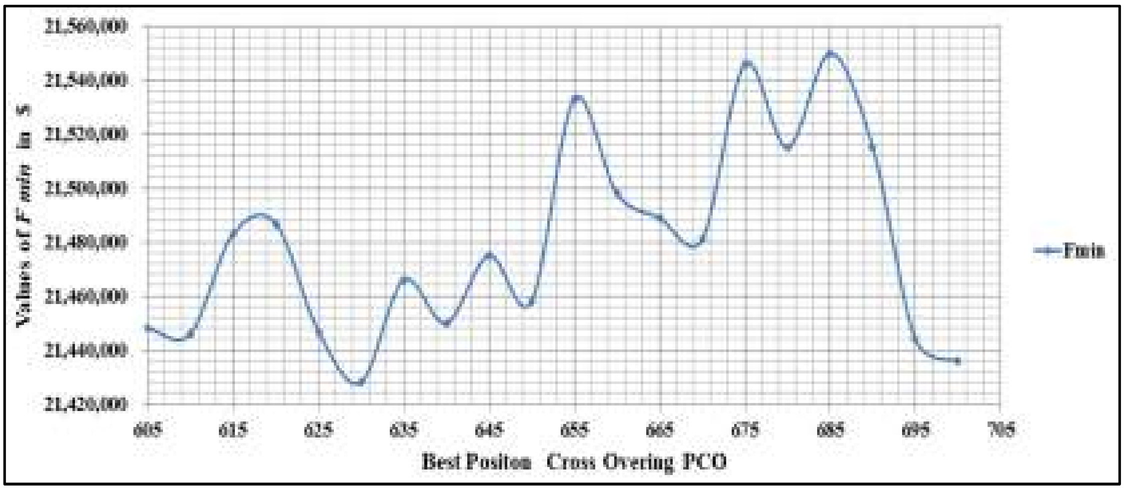


Fig.(5.10g): Values of Fmin of NP = 500 and PCOs of 605 to 700, (Researcher)

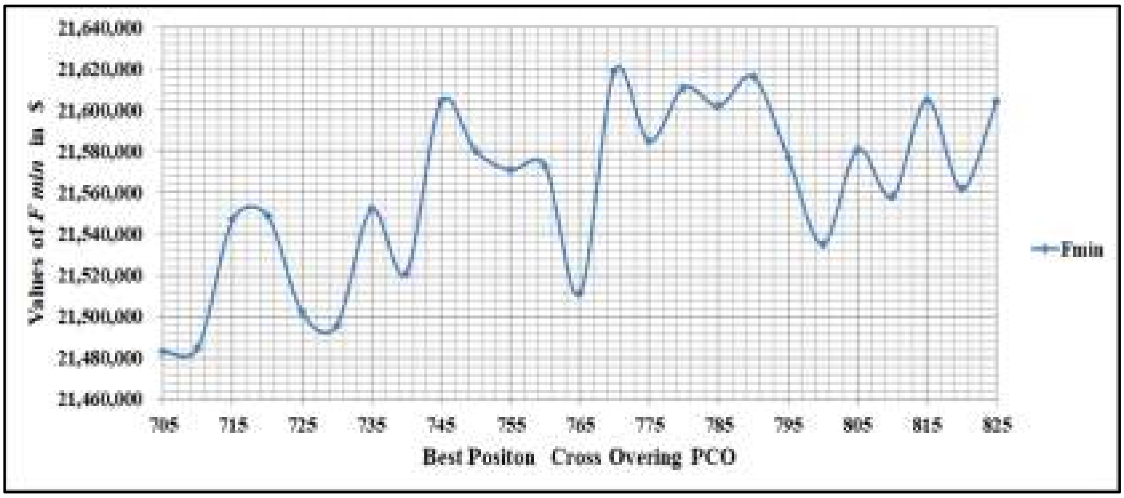


Fig.(5.10h): Values of Fmin of NP = 500 and PCOs of 705 to 825, (Researcher)

The results of the runs of Np = 500 shows minimum values of F_{min} and six least values are selected from the results as shown in Table (5.7). Further runs around each of the six values are conducted. The additional runs are by taking four steps before and after each selected value as shown in Fig.5.11a to 5.11f.

Table (5.7): Values of the six (F_{min} x 10^3) of *NP = 500* and *It = 4*, (Researcher)

$F_{min\ 1}$	$F_{min\ 2}$	$F_{min\ 3}$	$F_{min\ 4}$	$F_{min\ 5}$	$F_{min\ 6}$
21,439 $	21,423 $	21,429 $	21,423 $	21,428 $	21,436 $
PCO 140	*PCO 245*	*PCO 445*	*PCO 545*	*PCO 630*	*PCO 700*

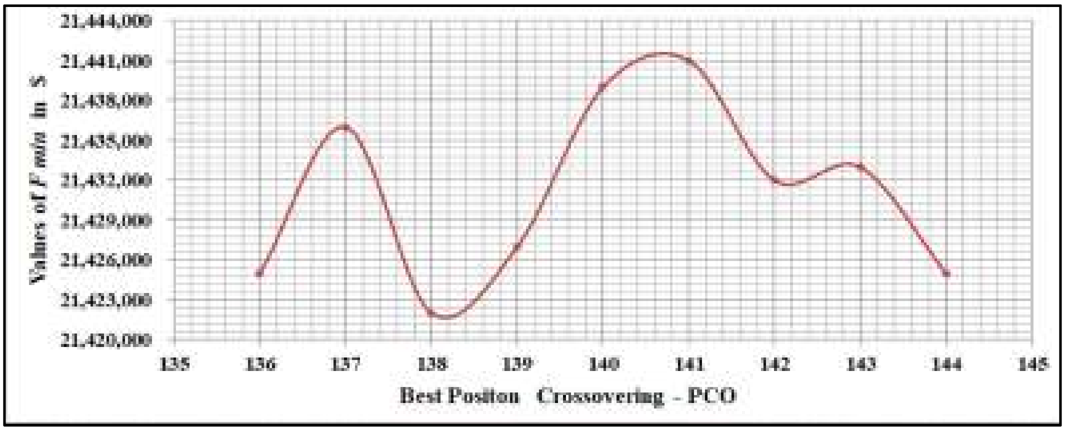


Fig.(5.11a): Additional Eight Runs around Fmin 1 , NP = 500, Step 1, (Researcher)

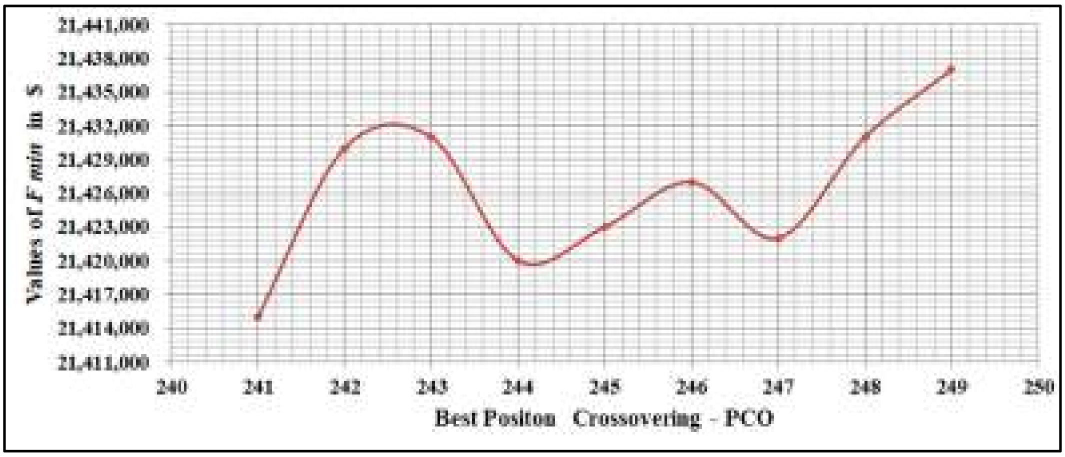


Fig.(5.11b): Additional Eight Runs around Fmin 2 , NP = 500, Step 1, (Researcher)

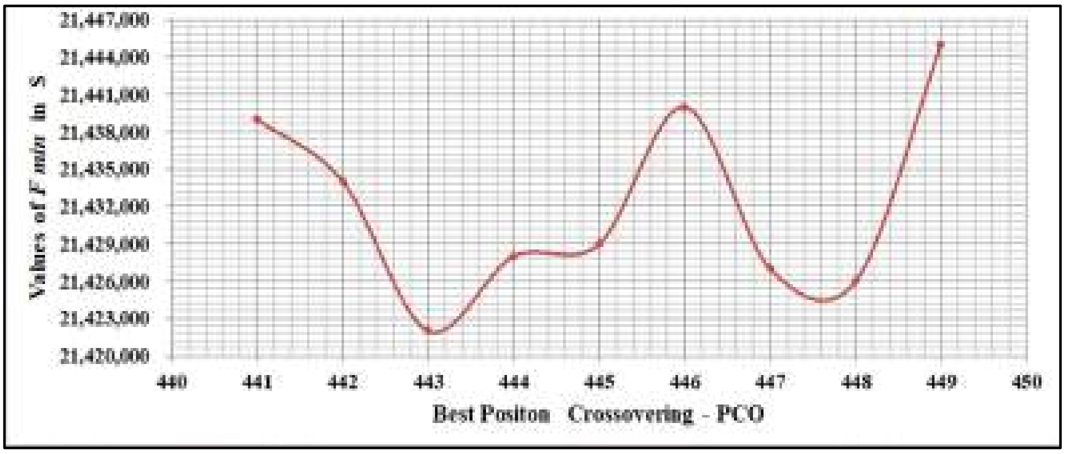


Fig.(5.11c): Additional Eight Runs around Fmin 3 , NP = 500, Step 1, (Researcher)

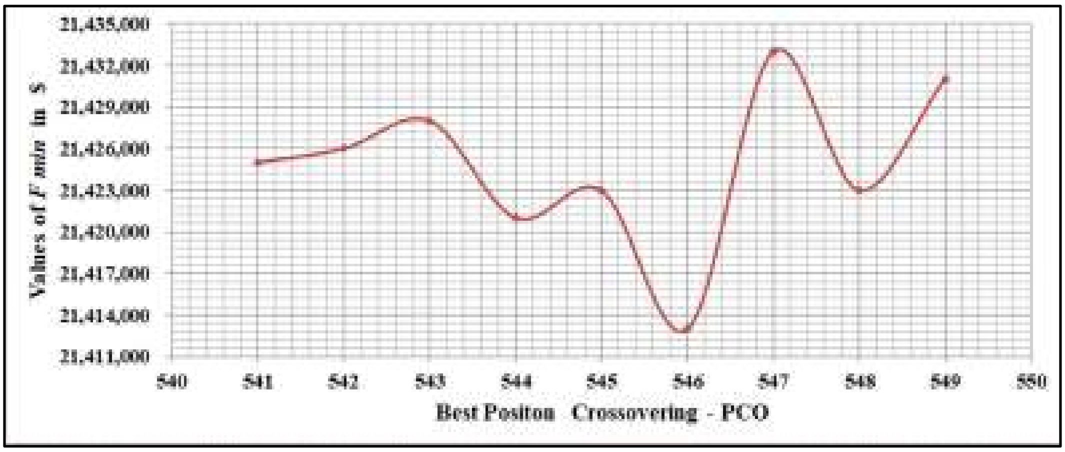


Fig.(5.11d): Additional Eight Runs around Fmin 4 , NP = 500, Step 1, (Researcher)

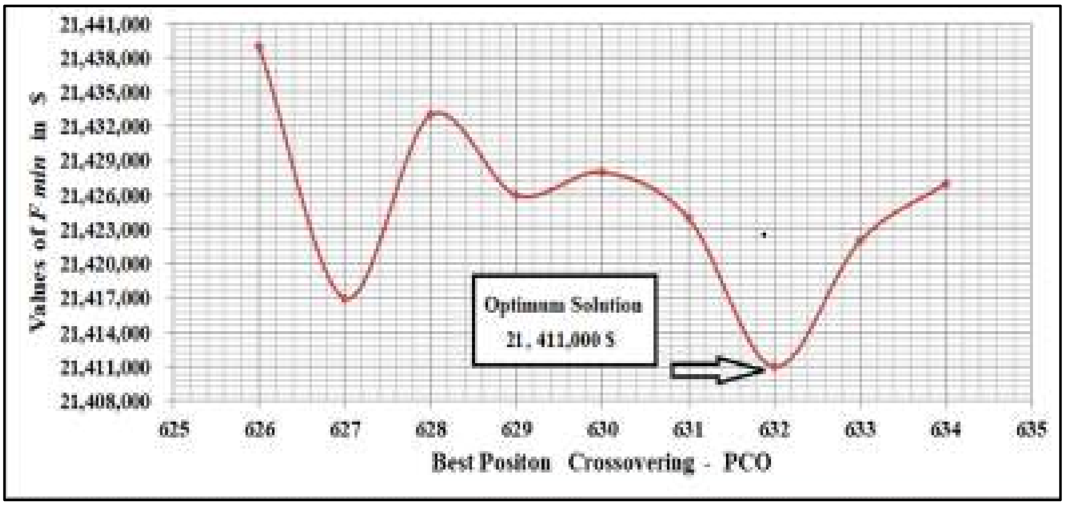


Fig.(5.11e): Additional Eight Runs around Fmin 5 , NP = 500, Step 1, (Researcher)

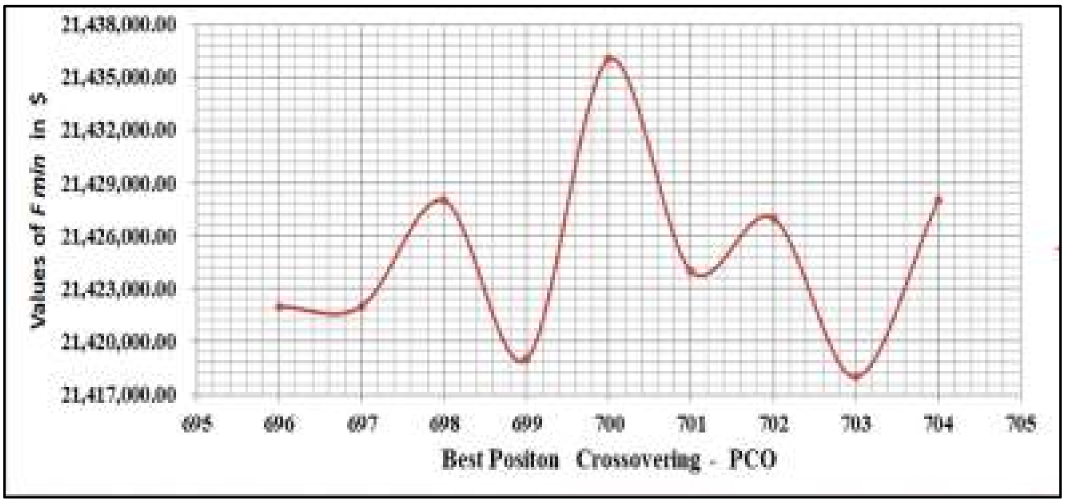


Fig.(5.11f): Additional Eight Runs around Fmin 6 , NP = 500, Step 1, (Researcher)

5.5 Optimum Solution

The optimum value of F_{min} is founded to be equal to 21,411,000 $ and it is obtained at *PCO* = 632 as shown in Fig. (5.31e). The best solution gives the results of: (1) the optimum capacity of each DWWTUs, (2) the size and head of the pumps at each DWWTUs, (3) the pipe diameters and the lengths from each DWWTU to the green areas , (4) number of the pressurized and gravity pipes and (5) dimensions of each DWWTUs. The details are shown below;

5.5.1 Optimum Pipe Sizes

The pipe sizes that obtained from the optimum solution are the inner diameter and the pipe type that selected is PE 100 PN 16 as used by the (DOWS, 2017).The pipe thicknesses of PE -100 PN16 are added (Uponor Limited, 2008). Table (5.8) shows the interpretation results of the piping networks that supplies the green areas and the detail results are shown in tables (A.16) in appendix A.

Table (5.8) : The Interpretation Results of the Piping Networks, (Researcher)

No.	Item	Details
1	Flow of pipe Q , m^3/day	12.0 - 1,634.0
2	Diameters (OD) , mm	20 – 180
3	Velocity, m/s	0.60 – 1.20
4	Number of Pipes	159
5	**Pipe Diameters**	**Pipe Lengths, m**
	20 mm	5,949.00
	25 mm	19,257.00
	32 mm	15,755.00
	40 mm	13,463.00
	50 mm	18,139.00
	63mm	11,693.00
	75 mm	6,020.00
	90 mm	1,461.00
	110 mm	3,161.00
	160 mm	963.00
	180 mm	931.00
	Total Pipe Length , m	**96,792.00**

5.5.2 Optimum DWWTUs Capacities

One of the aims of the optimization model was to find the sizes of each DWWTUs that gives the minimum cost. The optimum sizes of the 31 DWWTUs were found and they have different sizes started from (150 – 2,100) m^3/day. Most of the treatment units' sizes ranged from (500 – 700) $m^{3/}$day. The total capacities of the DWWTUs are about 26, 150 m^3/day. Table (5.9) shows the details of the capacity of the treatment units.

Table (5.9): The Results of the Optimum Sizes of the DWWTUs, (Researcher)

No	DWWTUs	Primary Design Capacity m^3/day	Standard Size m^3/day	Location
1	OA1	640	700	Line A
2	OB1	525	600	Line B
3	OB2	303	500	Line B
4	OB3	1,735	1,750	Line B
5	OB4	1,724	1,750	Line B
6	OC1	1,032	1,250	Line C
7	OC2	595	600	Line C
8	OC3	468	500	Line C
9	OC4	792	800	Line C
10	OD1	435	500	Line D
11	OE1	1,550	1,600	Line E
12	OE2	1,657	1,750	Line E
13	OE3	2,087	2,100	Line E
14	OE4	1,026	1,250	Line E
15	OE5	506	600	Line E
16	OF1	95	150	Line F
17	OF2	376	500	Line F
18	OF3	338	500	Line F
19	OF4	562	600	Line F
20	OG1	1,292	1,500	Line G
21	OG2	362	500	Line G
22	OG3	711	750	Line G
23	OG4	489	500	Line G
24	OH1	694	700	Line H
25	OH2	522	600	Line H
26	OH3	340	500	Line H
27	OI1	587	600	Line I
28	OI2	692	700	Line I
29	OI3	295	500	Line I
30	OJ1	780	800	Line J
31	OJ2	467	500	Line J

5.5.3 Optimum Pump Capacities

The results showed that 15 pipes out of the total 159 pipes are gravity pipe and the remaining reclamation pipes supplied by pumping. Table (5.10) showed the locations of the gravity pipes. The remaining pipes are pressurized and each pipe works under a specific pump head as shown in Table (A.17) in appendix A. Each treatment unit supplies a number of green areas through a number of pipes. Each pipe has a required pressure head (*ho*). The selected pump head of each DWWTU is the maximum pressure head value of the pipes that supply the green area groups. For example, seven pipes are connected to DWWTU OA1 and each has its pressure head as shown in Table (5.11). The selected pump head for OA1 treatment plant is equal to 48 m. Table (5.12) shows the pressure heads of the pumps of the 31 DWWTUs.

Table (5.10) : The Gravity Pipes from Optimized DWWTUs to the GRs, (Researcher)

No.	Pipe		No.	Pipe	
	From DWWTU	To GR		From DWWTU	To GR
1	OB3	GR 732	9	OF1	GR713
2	OB4	GR 733	10	OF2	GR 670
3	OC2	GR 88	11	OG1	GR 729
4	OC3	GR 7	12	OG2	GR 761
5	OD1	GR 733	13	OG3	GR 826
6	OE1	GR 722	14	OH3	GR 532
7	OE2	GR 283	15	OI1	GR 66
8	OE3	GR 713			

.

Table (5.11): The Pressure Head of Pipes of DWWTU named OA1, (Researcher)

Treatment Unit	GA	Pressure Head *ho*, m
OA1	GR 411	9
OA1	GR 717	18
OA1	GR 693	33
OA1	GR 758	48
OA1	GR 252	39
OA1	GR 228	22
OA1	GR 537	30

Table (5.12) : The Pump Heads of the Reclaimed Water Tank (T1) of each DWWTU, (Researcher)

No	DWWTUs	Max. Pump Head , m	No	DWWTUs	Max. Pump Head , m
1	OA1	48	17	OF2	54
2	OB1	54	18	OF3	128
3	OB2	73	19	OF4	106
4	OB3	30	20	OG1	114
5	OB4	95	21	OG2	125
6	OC1	112	22	OG3	72
7	OC2	67	23	OG4	88
8	OC3	53	24	OH1	71
9	OC4	46	25	OH2	105
10	OD1	61	26	OH3	127
11	OE1	35	27	OI1	88
12	OE2	82	28	OI2	118
13	OE3	120	29	OI3	93
14	OE4	90	30	OJ1	123
15	OE5	37	31	OJ2	111
16	OF1	36			

5.6 The Extended Aeration Package Units Details

The details of the main components of each treatment unit include sizing the (1) inlet chamber, (2) Screen ,(3) aeration tank, (4) secondary clarification, (5) disinfection tank, (6) storage tank for the reclaimed water, (7) pumping station, and (8) aerobic digester. The design parameters are clarified in chapter three. The results of the details of all decentralized extended aeration package plants are shown in Tables (5.13) to (5.16).

Sample of Design Calculation of DWWTU- OG1

Available Area = 9,809 m^2, District Name = Kaziwa 234,
The daily average flow of the treatment plant Q_{AV} =1,500 m^3/day (from table (5.9) , No. of Capita=7,500

1. Inlet Chamber:

Q_{AV} =1,500 m^3/day , Peak Daily Factor = 2.5
Q_{PD} = 1,500 x 2.5 = 3,750 m^3/day
Assume detention time = 1 min
Use two tanks

Volume of tank = (Q_{PD} /time)/(No. of tanks)
= (3,750 m^3/day /(1 minx 3600 x 24)/2) = 1.30 m^3
Assume depth of water = 1.25 m
Area required for inlet chamber = (1.30/1.25) = 0.9 m^2
Assume L/W = 1.0
Depth of tank = 0.70 m
Length of tank = 0.70 m

2. Screen Chamber /Fine Screen
Q_{PD} = 1,500 x 2.5 = 3,750 m^3/day = 0.043 m^3/s
Assume clear spacing between bars = 6.00 mm
Velocity head of screen = 0.6 m/s
Assume side water depth = 0.5 m
Area = Q/ V = (0.043/0.6) = 0.70 m^2
Assume angle of inclination 60 °
Assume detention period in the screen channel = 5 sec
Length of screen chamber = V x time = 0.60 m/s x 5 sec= 3 m
Inclined Height =0.40 m

3. Flow Equalization Basin:
Q_{AV} =1,500 m^3/day , Peak Daily Factor = 2.5
Q_{PD} = 1,500 x 2.5 = 3,750 m^3/day
Assume No. of Tanks = 2, Detention time = 2 hr
Volume of Each tank =((3,750/(24)) x 2)/2 = 156 m^3
Let depth of tank = 4 m, L/W = 1.0
Surface area =156 /4 =39.1 m^2 , say 40 m^2
L=6.235 say 6.5 m = W

4. The Aeration Tank Design (Va)
Volume of Aeration Tank (Va) = Q_{AV} x Detention time
Va = 1,500 m^3/day x 1.0 day (24 hr, table (3.3))= 1,500 m^3
Use two tanks of 780 m^3 , assume the depth H = 4.0 m , L=15m, W=13m
BOD_5 Kg/capita.day = 81 g/capita.day, O_{teff} = 6%, O_2 % in air = 23.2 %,
ρ_a =1.2 Kg/m^3 at standard temperature and pressure
Peak daily BOD_5 =2.5 x 81 g/capita. day x 7500 capita =1,534 Kg/day

$$\text{Air required } (m^3/\text{day}) = \frac{1{,}534}{6\ \%\ \text{x}\ 1.21\ \text{x}\ 23.2\%\ \text{x}1440\ \text{min/day}}$$

Air required for both aeration tanks = 63 m^3/min

5. The Secondary Clarifier

The overflow rate based on peak hourly flow = $33 m^3/m^2$ .day, (from table 3.3),

Peak hourly Factor = 4.0

Q_{ph} = 1,500 x 4.0 = 6,000 m^3/day

$$\text{Tank surface area} = \frac{6,000}{32.6} = 184 \text{ m}^2$$

Use two tanks each have surface area equal to 92 m^2

6. The Chlorination Tank (Vc)

Chlorination tank volume (Vc) = Q_{ph} x detention time,

Use detention time = 30 min, (Table 3.3)

$$\text{Chlorination tank volume (Vc)} = \frac{4 \times 1,500 \text{ m}^3/\text{day} \times 0.5 \text{ hr}}{24 \text{hr/day}} = 125 \text{ m}^3$$

7. Treated Water Tank T1

Q_{AV} = 1,500 m^3/day ,

Assume detention time = 1. 0 hr

$$\text{Volume of tank T1} = \frac{1,500 \text{ m}^3/\text{day}}{24 \text{ hr/day}} \times 1.0 \text{ hr} = 62.5 \text{ m}^3$$

The flow diagram of the designed OA1 - EA package plant is shown in Fig. (5.12).

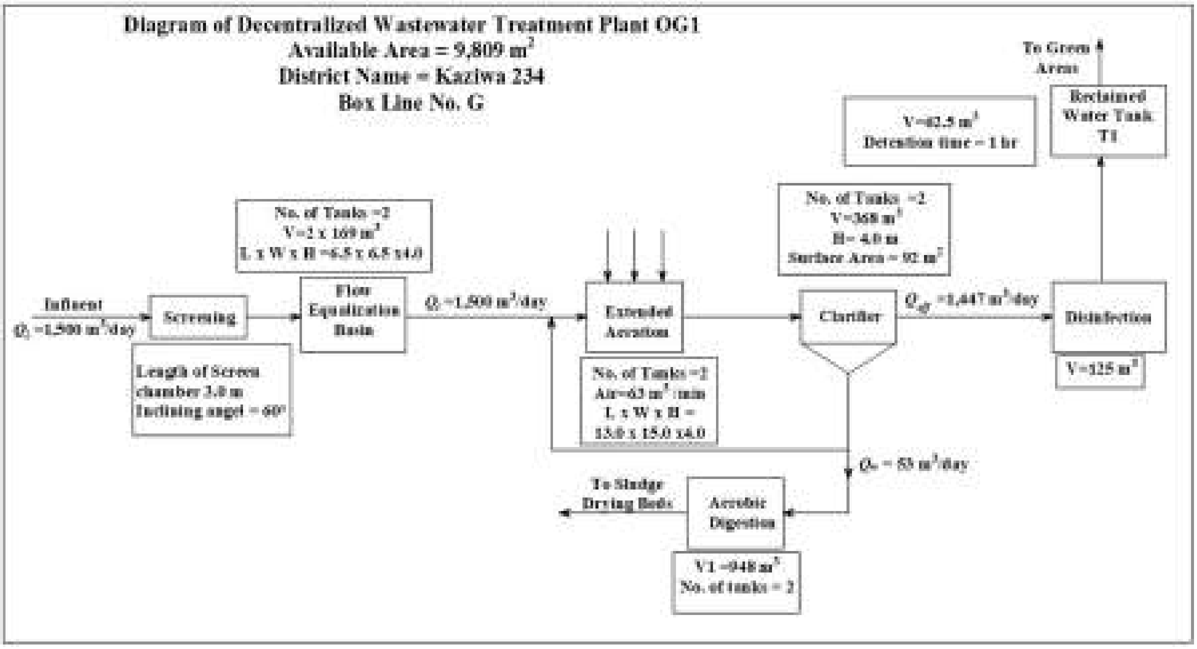


Fig.(5.12):The Flow Diagram of the Detail of DWWTU OG1, (Researcher)

Table (5.13): The Details for the Design of the Aeration Tanks of the DWWTUs, (Researcher)

DWWTU	Capacity , m^3/day	Va [a], m^3	No. of Tanks	H [b], m	Surface Area, m^2	Required Air , m^3/min
OA1	700	700	1.0	4.0	175.0	30
OB1	600	600	1.0	4.0	150.0	25
OB2	500	500	1.0	4.0	125.0	21
OB3	1,750	1,750	2.0	4.0	218.8	74
OB4	1,750	1,750	2.0	4.0	218.8	74
OC1	1,250	1,250	2.0	4.0	156.3	53
OC2	600	600	1.0	4.0	150.0	25
OC3	500	500	1.0	4.0	125.0	21
OC4	800	800	1.0	4.0	200.0	34
OD1	500	500	1.0	4.0	125.0	21
OE1	1,600	1,600	2.0	4.0	200.0	67
OE2	1,750	1,750	2.0	4.0	218.8	74
OE3	2100	2,100	2.0	4.0	262.5	89
OE4	1,250	1,250	2.0	4.0	156.3	53
OE5	600	600	1.0	4.0	150.0	25
OF1	150	150	1.0	4.0	37.5	6
OF2	500	500	1.0	4.0	125.0	21
OF3	500	500	1.0	4.0	125.0	21
OF4	600	600	1.0	4.0	150.0	25
OG1	1,500	1,500	2.0	4.0	187.5	63
OG2	500	500	1.0	4.0	125.0	21
OG3	750	750	1.0	4.0	187.5	32
OG4	500	500	1.0	4.0	125.0	21
OH1	700	700	1.0	4.0	175.0	30
OH2	600	600	1.0	4.0	150.0	25
OH3	500	500	1.0	4.0	125.0	21
OI1	600	600	1.0	4.0	150.0	25
OI2	700	700	1.0	4.0	175.0	30
OI3	500	500	1.0	4.0	125.0	21
OJ1	800	800	1.0	4.0	200.0	34
OJ2	500	500	1.0	4.0	125.0	21

a; Va = Volume of the Aeration Tank, b; H = Height.

Table (5.14) : The Details for the Design of the Secondary Clarifier of the DWWTUs. , (Researcher)

DWWTU	Capacity m^3/day	Q_{ph} [a] m^3/day	No. of Tanks	H [b] , m	Surface Area, m^2	Volume, m^3
OA1	700	2,800	1.0	4.0	86	344
OB1	600	2,400	1.0	4.0	74	295
OB2	500	2,000	1.0	4.0	61	245
OB3	1,750	7,000	2.0	4.0	107	430
OB4	1,750	7,000	2.0	4.0	107	430
OC1	1,250	5,000	2.0	4.0	77	307
OC2	600	2,400	1.0	4.0	74	295
OC3	500	2,000	1.0	4.0	61	245
OC4	800	3,200	1.0	4.0	98	393
OD1	500	2,000	1.0	4.0	61	245
OE1	1,600	6,400	2.0	4.0	98	393
OE2	1,750	7,000	2.0	4.0	107	430
OE3	2,100	8,400	2.0	4.0	129	515
OE4	1,250	5,000	2.0	4.0	77	307
OE5	600	2,400	1.0	4.0	74	295
OF1	150	600	1.0	4.0	18	74
OF2	500	2,000	1.0	4.0	61	245
OF3	500	2,000	1.0	4.0	61	245
OF4	600	2,400	1.0	4.0	74	295
OG1	1,500	6,000	2.0	4.0	92	368
OG2	500	2,000	1.0	4.0	61	245
OG3	750	3,000	1.0	4.0	92	368
OG4	500	2,000	1.0	4.0	61	245
OH1	700	2,800	1.0	4.0	86	344
OH2	600	2,400	1.0	4.0	74	295
OH3	500	2,000	1.0	4.0	61	245
OI1	600	2,400	1.0	4.0	74	295
OI2	700	2,800	1.0	4.0	86	344
OI3	500	2,000	1.0	4.0	61	245
OJ1	800	3,200	1.0	4.0	98	393
OJ2	500	2,000	1.0	4.0	61	245

a; Q_{ph} = Peak hourly flow, b; H = Height.

Table (5.15) : The Details for the Design of the Chlorination Tank of DWWTUs, (Researcher)

DWWTU	Capacity m^3/day	Q_{ph} [a] m^3/day	Vc [b] m^3	DWWTU	Capacity m^3/day	Q_{ph} [a] m^3/day	Vc [b] m^3
OA1	700	2,800	58.3	OF2	500	2,000	41.7
OB1	600	2,400	50.0	OF3	500	2,000	41.7
OB2	500	2,000	41.7	OF4	600	2,400	50.0
OB3	1,750	7,000	145.8	OG1	1,500	6,000	125.0
OB4	1,750	7,000	145.8	OG2	500	2,000	41.7
OC1	1,250	5,000	104.2	OG3	750	3,000	62.5
OC2	600	2,400	50.0	OG4	500	2,000	41.7
OC3	500	2,000	41.7	OH1	700	2,800	58.3
OC4	800	3,200	66.7	OH2	600	2,400	50.0
OD1	500	2,000	41.7	OH3	500	2,000	41.7
OE1	1,600	6,400	133.3	OI1	600	2,400	50.0
OE2	1,750	7,000	145.8	OI2	700	2,800	58.3
OE3	2,100	8,400	175.0	OI3	500	2,000	41.7
OE4	1,250	5,000	104.2	OJ1	800	3,200	66.7
OE5	600	2,400	50.0	OJ2	500	2,000	41.7
OF1	150	600	12.5				

a; Q_{Ph} = Peak Hourly Flow, b; Vc = Volume of Chlorination Tank.

Table (5.16) : The Details of the Treated Wastewater Tank T1 of the DWWTUs, (Researcher)

DWWTU	Capacity m^3/day	Detention Time, hr	V_{T1} [a], m^3	H [b], m
OA1	700	1.0	29.2	1.5
OB1	600	1.0	25.0	1.5
OB2	500	1.0	20.8	1.5
OB3	1,750	1.0	72.9	1.5
OB4	1,750	1.0	72.9	1.5
OC1	1,250	1.0	52.1	1.5
OC2	600	1.0	25.0	1.5
OC3	500	1.0	20.8	1.5
OC4	800	1.0	33.3	1.5
OD1	500	1.0	20.8	1.5
OE1	1,600	1.0	66.7	1.5
OE2	1,750	1.0	72.9	1.5
OE3	2,100	1.0	87.5	1.5
OE4	1,250	1.0	52.1	1.5

a; V_{T1} = Volume of Treated Wastewater Tank T1, b; H = Height.

Table (5.16):

DWWTU	Capacity m^3/day	Detention Time, hr	$V_{T1}{}^{a}$, m^3	H^{b}, m
OE5	600	1.0	25.0	1.5
OF1	150	1.0	6.3	1.5
OF2	500	1.0	20.8	1.5
OF3	500	1.0	20.8	1.5
OF4	600	1.0	25.0	1.5
OG1	1,500	1.0	62.5	1.5
OG2	500	1.0	20.8	1.5
OG3	750	1.0	31.3	1.5
OG4	500	1.0	20.8	1.5
OH1	700	1.0	29.2	1.5
OH2	600	1.0	25.0	1.5
OH3	500	1.0	20.8	1.5
OI1	600	1.0	25.0	1.5
OI2	700	1.0	29.2	1.5
OI3	500	1.0	20.8	1.5
OJ1	800	1.0	33.3	1.5
OJ2	500	1.0	20.8	1.5

a; V_{T1} = Volume of Treated Wastewater Tank T1, b; H = Height.

5.7 The Sludge Disposal

This part is related to all the processing related to the sludge produced from the extended aeration plant such as; calculating the produced sludge rate, design of the aerobic digester and the sand drying bed's design and location in the study area. The details are shown in the followings paragraphs:

5.7.1 The Wastewater Flow Calculations Q_w

The Waste flow Q_w is calculated using Eqs.(3.14) and (3.15) as shown below:

$$\theta_c = \frac{VX}{Q_w\ X + (Q_{in} - Q_w)X_e}$$

From table(3.3), assume the following data:
$\theta_c =$ 25 days , X = 4000 mg/L,
X_e = 20 mg/L , (EPA, 2000, p. 4)

$t = \dfrac{V}{Q_{in}}$, $t = 24\ hrs \longrightarrow V = t\,Q_{in} = (24/24)\,Q_{in} \longrightarrow V = Q_{in}$

Substituting Eq.(3.17) into Eq.(3.18) get :

$$Q_w = 0.0352\,Q_{in} \longrightarrow Q_w = 3.52\ \%\ Q_{in} \qquad (5.3)$$

$$Q_{eff} = Q_{in} - Q_w \longrightarrow Q_{eff} = 96.48\ \%\ Q_{in} \qquad (5.4)$$

Applying Eqs.(5.3) and (5.4) values of Q_w and Q_{eff} are found and the details for all treatment units are shown in Table (5.17);

Table (5.17) : The Values of the Waste Flow Q_w from each DWWTU, (Researcher)

DWWTU	Size m^3/d	Q_w m^3/d	Q_{eff} m^3/d	DWWTU	Size m^3/d	Q_w m^3/d	Q_{eff} m^3/d
OA1	700	25	675	OF2	500	18	482
OB1	600	21	579	OF3	500	18	482
OB2	500	18	482	OF4	600	21	579
OB3	1,750	62	1,688	OG1	1,500	53	1,447
OB4	1,750	62	1,688	OG2	500	18	482
OC1	1,250	44	1,206	OG3	750	26	724
OC2	600	21	579	OG4	500	18	482
OC3	500	18	482	OH1	700	25	675
OC4	800	28	772	OH2	600	21	579
OD1	500	18	482	OH3	500	18	482
OE1	1,600	56	1,544	OI1	600	21	579
OE2	1,750	62	1,688	OI2	700	25	675
OE3	2,100	74	2,026	OI3	500	18	482
OE4	1,250	44	1,206	OJ1	800	28	772
OE5	600	21	579	OJ2	500	18	482
OF1	700	5	145				

5.7.2 The Aerobic Digester Design:

The design of the aerobic digester is for the tank volume and the required oxygen and air as in below;

1. The Tank Volume: It is calculated by applying Eq. (3.16);

$$V_d = \frac{Q_W \; X_i}{X(K_d \, P_v + 1/\theta_c)}$$

The values of Q_W from Table(5.17) are applied and the following data are assumed;
K_d =0.06 day^{-1} at temperature 15 ˚C and K_d =0.14 day^{-1} at temperature 25 ˚C , P_v = 0.8 , X = 70 % Xi , (Eddy, 2014, p. 840)

The temperature variation during winter and summer will effects on the volatile solid reduction % and Fig.(5.13) shows the relation between the [Sludge age (θ_c) x Temperature ˚C] and the volatile solid reduction % (Eddy, 2014, p. 840).

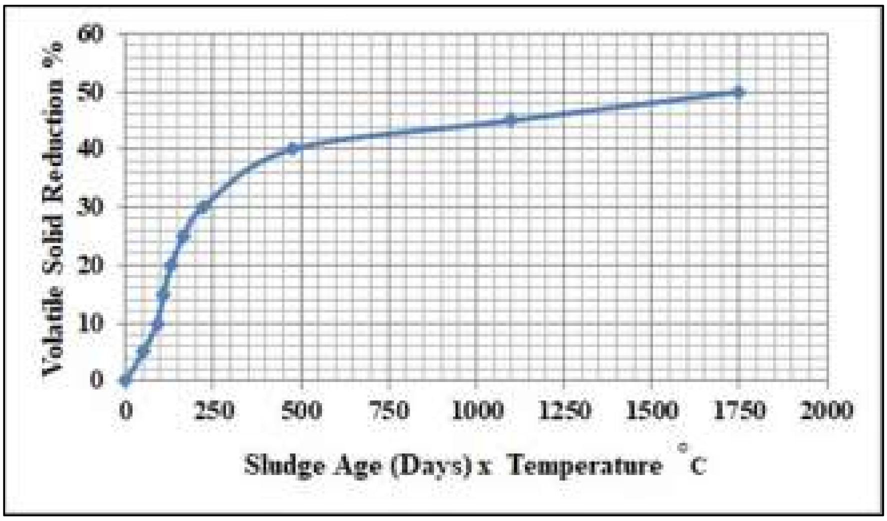


Fig. (5.13): Volatile Solid Reduction in Aerobic Sludge Digester as a Function of Digester Liquid Temperature and Sludge Age (Eddy, 2014, p. 838).

The values of the required sludge ages during summer and winter are found as in below:

- The value of volatile reduction % is taken to be equal to 40% as shown in Table (3.4) and from Fig.(5.13) the value of [*Temperature x* θ_c] will equal to 475 ˚C . day.
- The required sludge age at 15 ˚C will equal to: θ_c = 475/15 = 31.7 days and using the same sludge age for temperature 25 ˚C the % of volatile removal will = 44% .

- The tank should be covered to maintain the temperature within (15 – 25) ˚C.
- The X_i value represents the influent suspended solid concentration in mg/L of the digester and it is calculated from the solid load [Table (3.4)], and the sludge waste flow to the digester (Qw).

2. The Required Air Volume V_A: is calculated by applying Eq.(3.17)

Kg O_2/ day = VSS x 1.045 [Kg O_2/Kg cell tissue destroyed],
VSS = 0.8 x TSS
TSS Kg/day= Q_{AV} x dry solid, [dry solid =0.8 Ib/10^3 gal=0.096 Kg/m^3 , Table (3.3)]
Volume of air required (V_A) at standard conditions
V_A = [Kg O_2/ day] /[ρ_a Kg/m^3 x 23.2 % of O_2 in air x Q_{eff} %],
(ρ_a air density=1.225 Kg/m^3 at T=15 ˚C and 1.183 Kg/m^3 at T=25 ˚C)

<u>Sample of Design Calculation of the Aerobic Digester of OA1's DWWTU</u>

For treatment unit OA1, Q_w = 25 m^3/day, the number of capita served by DWWTU named OA1 = 3,500.

1. The Digester Volume Vd

Volume of the aerobic digester of DWWTU OA1 is found by applying Eq.(3.16), [winter condition]:

$$Vd = \frac{(25 \text{ x } Xi)}{(0.7 \text{ x } Xi)\ (0.06 \text{ x } 0.8 + \frac{1}{31.7})} = 492 \text{ m}^3$$

Table (5.18) shows the results of Vd of the 31 DWWTUs.

2. The Air Required V_A

The amount of oxygen required is measured by applying Eq. (3.17) as in below;
TSS in Kg /day = 1,500 m^3/day x 0.096 Kg/m^3 = 67 Kg/day
Table (5.18) shows the results of the TSS of the 31 DWWTUs.
VSS = 0.8 x TSS = 0.8 x 67 = 53.6 Kg/day

The required O_2 is ;

a. **For Winter**

Reduced VSS = 53.6 x 0.40 = 21.44 Kg VSS /day

Kg O_2/ day = 21.44 x 1.045 = 22.40 Kg O_2/day

b. **For Summer**
Reduced VSS = 53.6 x 0.44 = 23.58 Kg VSS /day
Kg O_2/ day = 23.58 x 1.045 = 24.65 Kg O_2/day

The volume of air (V_A) required at 20 ˚C and assuming oxygen transfer efficiency = 10 %:

a. **For Winter**
V_A = [22.40 Kg/day] / [1.225 Kg/m^3 x 23.2 % of O_2 in air x 10%]
V_A = 791 m^3/day

b. **For Summer**
V_A = [24.65 Kg/day] / [1.183 Kg/m^3 x 23.2 % of O_2 in air x 10%]
V_A = 901 m^3/day

Table (5.19) shows the results of V_A of the 31 DWWTUs.

Table (5.18) : The Volumes Vd of the Aerobic Digesters of the 31 DWWTUs, (Researcher)

DWWTU	TSS Kg/day	Vd , m^3	DWWTU	TSS Kg/day	Vd , m^3
OA1	67	492	OF2	48	316
OB1	58	379	OF3	48	316
OB2	48	316	OF4	58	379
OB3	168	1,106	OG1	144	948
OB4	168	1,106	OG2	48	316
OC1	120	790	OG3	72	474
OC2	58	379	OG4	48	316
OC3	48	316	OH1	67	443
OC4	77	506	OH2	58	379
OD1	48	316	OH3	48	316
OE1	154	1,011	OI1	58	379
OE2	168	1,106	OI2	67	443
OE3	202	1,328	OI3	48	316
OE4	120	790	OJ1	77	506
OE5	58	379	OJ2	48	316
OF1	14	95			

Table (5.19) : The Volume of the Required Rate of Air in Winter and Summer for the Sludge Digester of the 31 DWWTUs, (Researcher)

DWWTU [a]	V_A, m^3 air/day		DWWTU [a]	V_A, m^3 air /day	
	Winter	Summer		Winter	Summer
OA1	791	901	OF2	565	644
OB1	678	773	OF3	565	644
OB2	565	644	OF4	678	773
OB3	1,978	2,253	OG1	1,696	1,931
OB4	1,978	2,253	OG2	565	644
OC1	1,413	1,610	OG3	848	966
OC2	678	773	OG4	565	644
OC3	565	644	OH1	791	901
OC4	904	1,030	OH2	678	773
OD1	565	644	OH3	565	644
OE1	1,809	2,060	OI1	678	773
OE2	1,978	2,253	OI2	791	901
OE3	2,374	2,704	OI3	565	644
OE4	1,413	1,610	OJ1	904	1,030
OE5	678	773	OJ2	565	644
OF1	170	193			

5.7.3 The Drying Bed Design

The drying bed is designed based on the number of capita of the DWWTUs which is equal to 130,750 capita. By applying Eq. (3.19) for covered drying beds the total area required is:

Total bed Area A = 0.15 m^2 x No. of capita = 0.15 x 130,750 = 19,613 m^2
The dimensions of the drying bed cells are calculated as in below:
Cell Area Ac = L (length) x W (width)
Let L = 45 m, W = 12 m, Ac = 540 m^2
Number of cells NC = A / Ac = 19,613/540 = 37 cell

5.7.4 The Drying Beds Proposed Location

The best location is selected from the GIS map as shown in Fig.(5.14) with the following details;

1. The specified available area is equal to 150,000 m^2.
2. The Latitudes are between (35° 30' 26.91" – 35° 30' 15.49") N and Longitudes are between (45° 24' 18.74" – 45° 24' 36.23") E

3. The ground elevations are between (735 - 730) amsl
4. The ground water levels are between (700 – 710) amsl (Qaradaghy, 2015)
5. Faraway from Qilyasan Stream in a distance of 1,760 m.
6. Faraway from the residential areas by a distance not less than 1,600 m.

Fig.(5.14): The Location of the Sludge Drying Bed, (Researcher)

CHAPTER SIX

CONCLUSION, RECOMMENDATIONS AND PUBLICATIONS

Chapter Six
Conclusions, Recommendations and Publications

6.1Conclusions

The purpose of this study was to find the optimum number, sizes and locations of the DWWTUs in Sulaimania city. Moreover, the reclaimed water from the DWWTUs to be reused for irrigation purposes of the green areas inside the city. From the results and analysis the following points were concluded;

1. The method that used to find the suitable location of the DTWWTs was very robust and it helped to determine a solution of difficult decisions in comparing with ordinary methods. The suitability model (MCDM) was developed by using GIS, Analytical Hierarchy process AHP and statistical analysis to select the optimum locations. As a result of the suitability model, 31 optimum locations out of the 134 areas were found to serve the city.

2. The Transportation Model and GA in a Matrix form were capable of connecting an enormous amount of data that covers the whole city of Sulaimania. This combination was used for the first time in this type of applications and it could successfully obtain an optimal solution. The algorithm has adequate flexibility to assume various types of scenarios and compare the optimum solutions. The applied genetic algorithm was robust, avoiding local optima to attain the global optimum. The algorithm has the flexibility of adapting the cost estimates to any geographical region.

3. The developed model allows easy way to determine the required GA parameters as the minimum required number of NP, the cross over position and number of iterations. The minimum NP value that produce stable results was found at NP=500. The cross over matrix process was created and checked, keep the developed offspring feasible and they also satisfy the constraints as the

parent's solution. As a result optimized sizes of 31 EA treatment plants were found.

4. The reclaimed water pipes best routs and lengths from the DWWTUs to the GRs were found using Network Analysis - OD Cost Matrix method in GIS for finding .This tool was used for the first time in piping networks and it was a fast and accurate method.

5. The digested sludge is conveyed to one big sand drying bed having an area equal to 19,613 m^2 and it consists of 37 cell. The length of each cell is 45 m and the width is 12 m. The location was found to be in the south west part of Sulaimania city.

6. The obtained DWWTUs can mitigate the problem of water scarcity in Sulaimania city as the treated wastewater will cover 55.17 % of the total water requirement of the green areas.

7. The number of gravity pipes was found to be 15 and the pressurized pipes are 144. The diameters are ranged from 20 mm – 180 mm. The number of pumps are 31 pump (one pump at each DWWTU plus one standby) and the pump heads ranged from (30 – 128) m.

8. From population forecasting calculations, the population density for individual districts were found and they were ranged from more than 300 capita/ha. to districts having population densities less than 50 capita/ha.

9. Calculations of the main sewer boxes' depths and invert levels were done and all the data related to the sewer boxes were added to the GIS sewer attributes. Moreover, corrections of the sewer paths in the GIS maps that received from Sulaimania Municipality has been done through site visits and matching with the as-built drawings.

6.2 Recommendations:

Below are some recommendations for future studies related to the current research:

1. The developed suitability model can easily generalized to be applied to any similar studies, by adding more criteria or more restrictions.

2. The same study could be applied for the other three suburbs of the city, Bakrajo , Rapareen and Tasloja. Especially the sewerage systems of the suburbs are separate and individual and have no effect on each other.

3. It is recommended to make a detail study about the characteristics of the wastewater of Sulaimania city by taking samples from different point and make a complete chemical, physical and biological tests.

4. It is important to make a study about reusing the treated wastewater from the DWWTUs for groundwater recharging, especially there are big number of wells in the study area.

5. It is a useful study to locate water tanks from the reclaimed water for firefighting and distribute it in the study area.

6. A study about specifying the details of each green area in Sulaimania city in terms of vegetation types and demands.

6.3 Publications:

1. A multi-criteria GIS model for suitability analysis of locations of decentralized wastewater treatment units: case study in Sulaimania, Iraq, Ako Rashed Hama , Rafea Hashim Al-Suhili , Zeren Jamal Ghafour, *2019 , Heliyon Journal , The Authors. Published by Elsevier Ltd., Article Nowe01355.*

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APPENDIX A

Table (A.1): The Details of the Sewer Box Branches (GDOSM-GIS, 2017)

Line	Branch	Dimension	No. of Box	Length, m
A	A1	2.5 m x 2.5 m	1.0	1,529
	A2	2.5 m x 2.5 m	1.0	1,884
	A3	1.5 m x 2.0 m	1.0	552
	A4	2.5 m x 2.5 m	1.0	315
	A5	2.5 m x 2.5 m	1.0	504
	A6	2.5 m x 2.5m	1.0	1,519
	A7	2.5 m x 2.5 m	1.0	883
B	B1	2.0 m x 2.0 m	1.0	1,341
	B2	1.0 m x 1.0 m	1.0	273
	B3	2.0 m x 2.0 m	1.0	1,655
	B3 -1	2.5 m x 2.5 m	1.0	1,459
	B4	1.0 m x 1.0 m	1.0	504
	B5	2.5 m x 2.5 m	1.0	941
	B6	1.0 m x 1.0 m	1.0	697
	B7	1.0 m x 1.0 m	1.0	375
	B8	1.0 m x 1.0 m	1.0	1,390
	B9	1.0 m x 1.0 m	1.0	248
	B10	1.0 m x 1.0 m	1.0	221
	B11	1.0 m x 1.0 m	1.0	661
	B12	1.5 m x 1.5 m	1.0	155
	B13	2 (2.5 m x 2.0 m)	2.0	1,306
	B14	2 (2.0 m x 2.0 m)	2.0	313
C	C1	1.0 m x 1.0 m	1.0	168
	C2	1.2 m x 1.2 m	1.0	506
	C3	1.2 m x 1.2 m	1.0	435
	C4	1.2 m x 1.5 m	1.0	402
	C5	1.0 m x 1.0 m	1.0	243
	C6	1.5 m x 1.5 m	1.0	421
	C7	1.2 m x 1.2 m	1.0	662
	C8	1.5 m x 2.0 m	1.0	557
	C9	1.5 m x 1.5 m	1.0	906
	C10	1.0 m x 1.0 m	1.0	226
	C11	2.0 m x 2.0 m	1.0	2,869
	C12	3.0 m x 3.0 m	1.0	422
	C13	1.0 m x 1.0 m	1.0	,96
	C14	3.0 m x 3.0 m	1.0	2,217
	C15	1.0 m x 1.0 m	1.0	560
	C16	1.0 m x 1.0 m	1.0	294
	C17	2.0 m x 2.0 m	1.0	3,205
	C18	3.0 m x 3.0 m	1.0	1,229
	C19	2.0 m x 2.0 m	1.0	589
	C20	3.0 m x 2.2 m	1.0	241
	C21	1.0 m x 1.0 m	1.0	303
	C22	1.0 m x 1.0 m	1.0	833
	C23	2.2 m x 3.0 m	1.0	573
	C24	2.5 m x 3.0 m	1.0	2,583
	C25	2(3.0 m x 3.0 m)	2.0	968

Table (A.1):

Line	Branch	Dimension	No. of Box	Length, m
D	D1	2.0 m x 2.0 m	1.0	947
E	E1	2.0 m x 2.0 m	1.0	4,457
	E2	2.0 m x 2.0 m	1.0	2,619
	E2 -1	2.0 m x 2.5 m	1.0	1,393
	E3	2.0 m x 2.5 m	1.0	608
	E4	2.0 m x 2.5 m	1.0	538
	E5	2.0 m x 2.5 m	1.0	452
	E6	1.0 m x 1.5 m	1.0	1,530
	E7	1.5 m x 1.5 m	1.0	1,265
	E8	1.0 m x 1.0 m	1.0	906
	E9	1.5 m x 1.5 m	1.0	822
	E10	3.0 m x 2.5 m	1.0	856
	E11	1.0 m x 1.0 m	1.0	1,131
	E12	3.0 m x 2.5 m	1.0	341
	E13	3.0 m x 2.5 m	1.0	2,110
	E14	1.0 m x 1.0 m	1.0	394
	E15	3.0 m x 2.5 m	1.0	828
	E16	3.0 m x 2.5 m	1.0	549
	E17	2.0 m x 2.0 m	1.0	223
	E18	3.0 m x 2.5 m	1.0	643
	E19	3.0 m x 2.5 m	1.0	529
	E20	3.0 m x 2.5 m	1.0	1,582
	E21	2.0 m x 2.5 m	1.0	2,190
	E22	1.5 m x 1.5 m	1.0	1,233
	E22 -1	1.0 m x 1.1 m	1.0	662
	E23	1.0 m x 1.0 m	1.0	489
	E24	2.0 m x 2.0 m	1.0	997
	E25	3.5 m x 2.0 m	1.0	1,789
	E25 -1	3.0 m x 2.75 m	1.0	1,023
F	F1	1.5 m x 1.5 m	1.0	769
	F2	1.0 m x 1.0 m	1.0	497
	F3	1.5 m x 1.5 m	1.0	494
	F4	1.5 m x 1.5 m	1.0	1,139
	F5	3.0 m x 3.0 m	1.0	3,701
	F6	1.0 m x 1.0 m	1.0	1,213
	F7	3.0 m x 3.0 m	1.0	3,210
G	G1	1.0 m x 1.0 m	1.0	2,143
	G1 -1	2.0 m x 2.0 m	1.0	1,722
	G2	1.0 m x 1.0 m	1.0	766
	G3	1.0 m x 1.0 m	1.0	121
	G4	1.0 m x 1.0 m	1.0	362
	G5	1.0 m x 1.0 m	1.0	513
	G6	1.0 m x 1.0 m	1.0	543
	G7	1.0 m x 1.0 m	1.0	365
	G8	2.5 m x 3.0 m	1.0	1,595
	G9	2.5 m x 3.0 m	1.0	328

Table (A.1) :

Line	Branch	Dimension	No. of Box	Length, m
G	G10	2.0 m x 1.5 m	1.0	725
	G11	2.5 m x 2.5 m	1.0	502
	G12	2.0 m x 2.0 m	1.0	370
	G13	2.5 m x 2.5 m	1.0	453
	G14	2.5 m x 3.0 m	1.0	1,550
	G15	2.5 m x 2.5 m	1.0	1,634
	G16	2.5 m x 2.5 m	1.0	936
	G17	2.5 m x 2.5 m	1.0	247
	G18	2.5 m x 2.5 m	1.0	619
	G19	2.5 m x 3.0 m	1.0	2,353
	G20	1.0 m x 1.0 m	1.0	341
	G21	1.5 m x 1.0 m	1.0	557
	G22	1.0 m x 1.0 m	1.0	434
	G23	1.5 m x 1.0 m	1.0	391
	G24	2.5 m x 2.5 m	1.0	1,274
	G25	2.5 m x 2.5 m	1.0	1,362
	G26	2.0 m x 2.0 m	1.0	785
	G27	2.5 m x 2.5 m	1.0	2,178
H	H1	2.5 m x 2.5 m	1.0	1,057
	H2	2.0 m x 2.0 m	1.0	668
	H3	2(2.0 m x 2.5 m)	2.0	4,191
	H4	1.0 m x 1.0 m	1.0	755
	H5	2(2.0 m x 2.5 m)	2.0	334
	H6	1.5 m x 1.5 m	1.0	2,466
	H7	2(2.0 m x 2.5 m)	2.0	7,812
I	I1	2.0 m x 2.0 m	1.0	850
	I2	1.5 m x 1.5 m	1.0	739
	I3	2.0 m x 2.0 m	1.0	3,256
	I4	2.0 m x 1.5 m	1.0	793
	I5	2.0 m x 1.5 m	1.0	900
	I6	2.0 m x 2.0 m	1.0	688
	I7	2.0 m x 2.0 m	1.0	430
	I8	1.5 m x 1.5 m	1.0	2,235
	I9	2.0 m x 2.0 m	1.0	895
J	J1	1.0 m x 2.0 m	1.0	2,857
	J2	1.0 m x 1.0 m	1.0	512
	J3	1.0 m x 1.0 m	1.0	357
	J4	1.0 m x 1.0 m	1.0	269
	J5	1.5 m x 2.0 m	1.0	5,520

Table (A.2): The Details of the 134 Nominated Areas, (Researcher)

NA[a]	Sewer Box	Area m^2	NA[a]	Sewer Box	Area m^2
NA1	A	7,540	ND1	D	10,686
NA2	A	5,413	NE1	E	4,950
NA3	A	5,736	NE2	E	6,028
NA4	A	8,236	NE3	E	4,446
NA5	A	11,815	NE4	E	3,327
NA6	A	7,712	NE5	E	2,742
NA7	A	70,445	NE6	E	3,427
NB1	B	5,202	NE7	E	3,196
NB2	B	7,843	NE8	E	5,625
NB3	B	5,121	NE9	E	3,730
NB4	B	7,430	NE10	E	6,663
NB5	B	8,337	NE11	E	1,613
NB6	B	5,121	NE12	E	2,198
NB7	B	5,544	NE13	E	3,599
NB8	B	6,744	NE14	E	3,921
NB9	B	4,919	NE15	E	1,472
NB10	B	4,365	NE16	E	6,613
NB11	B	8,034	NE17	E	1,381
NB12	B	22,460	NE18	E	2,681
NB13	B	5,272	NE19	E	2,994
NC1	C	2,974	NE20	E	1,925
NC2	C	4,042	NE21	E	4,587
NC3	C	4,345	NE22	E	8,851
NC4	C	6,623	NE23	E	11,663
NC5	C	3,629	NE24	E	21,684
NC6	C	2,833	NF1	F	13,327
NC7	C	1,774	NF2	F	8,508
NC8	C	3,851	NF3	F	1,351
NC9	C	1,794	NF4	F	4,819
NC10	C	1,351	NF5	F	3,649
NC11	C	3,327	NF6	F	4,708
NC12	C	3,145	NF7	F	4,153
NC13	C	4,163	NF8	F	8,468
NC14	C	3,821	NF9	F	4,335
NC15	C	3,276	NF10	F	9,315
NC16	C	3,790	NF11	F	20,575
NC17	C	6,180	NF12	F	3,296
NC18	C	3,508	NG1	G	3,952
NC19	C	3,559	NG2	G	9,809
NC20	C	5,928	NG3	G	5,565
NC21	C	5,232	NG4	G	3,821

a : NA= Nominated Area

Table(A.2)

NA[a]	Sewer Box	Area m^2	NA[a]	Sewer Box	Area m^2
NG5	G	1,653	NI10	I	1,280
NG6	G	1,936	NI11	I	1,764
NG7	G	1,210	NI12	I	1,139
NG8	G	1,502	NI13	I	3,337
NG9	G	2,671	NI14	I	2,147
NG10	G	5,071	NI15	I	2,369
NG11	G	2,782	NI16	I	1,784
NG12	G	3,296	NI17	I	6,240
NG13	G	3,246	NJ1	J	14,214
NG14	G	1,573	NJ2	J	9,163
NG15	G	2,510	NJ3	J	10,182
NG16	G	2,188	NJ4	J	45,807
NG17	G	5,655			
NG18	G	5,796			
NG19	G	3,478			
NG20	G	5,544			
NG21	G	2,712			
NG22	G	6,754			
NG23	G	13,518			
NG24	G	11,180			
NH1	H	5,776			
NH2	H	9,627			
NH3	H	2,077			
NH4	H	5,262			
NH5	H	4,194			
NH6	H	5,524			
NH7	H	4,425			
NH8	H	4,839			
NH9	H	6,361			
NH10	H	3,236			
NH11	H	6,926			
NH12	H	15,272			
NI1	I	2,329			
NI2	I	1,714			
NI3	I	2,389			
NI4	I	9,295			
NI5	I	5,020			
NI6	I	1,250			
NI7	I	1,905			
NI8	I	4,647			
NI9	I	1,270			

a : NA= Nominated Area

Table (A.3): Population Density of Sulaimania Zones (population of 2018), (Researcher)

No.	District Name/ Number	Area, m^2	Population (Capita)	Pop Density (Capita/ha)
1	Shorsh 101	534,502	8,115	152
2	Rapareen 102(parki Azadi)	534,502	6,510	122
3	Ali Naji 103	437,367	5,612	128
4	Ashti 1 104	454,547	9,985	220
5	Andazyaran 105	361,515	4,199	116
6	Ashti 2 106	613,528	11,954	195
7	Baranan 107	403,098	5,864	145
8	Baxan 108	746,290	5,170	69
9	Handren 109	315,088	4,635	147
10	Qazi Mohamed 110	434,803	10,414	240
11	Baxtiyari 111	467,516	3,984	85
12	Mamostayan 112	288,524	8,322	288
13	Hakari 113	305,502	9,209	301
14	Kareza wskk 1(Daban) 114	307,684	11,936	388
15	Shirwana 115	376,513	1,000	27
16	Dabashan 116	613,528	11,160	182
17	Baxtiyari Taza 117	409,050	16,095	393
18	Kareza Wshk (2) 118	301,305	5,956	198
19	Sarchnar(1) 119	541,747	9,036	167
20	Swren 120	483,331	12,185	252
21	Sarchnar(2) 121	379,292	11,441	302
22	Besarani 122	538,511	16,241	302
23	Harawazi (Grdi Sarchnar) 123	1,237,340	14,401	116
24	Badinan 124	382,643	7,477	195
25	Shakraka 125	304,082	9,745	320
26	Zargata 126	855,239	19,356	226
27	Sayrangay Sarchnar 127	1,959,477	234	1
28	Mashxalan 128	431,491	11,321	262
29	Qlyasan 129	283,755	8,510	300
30	Kani Speka 130	537,720	20,275	377
31	Xwar Kurdsat 134	1,683,354	1,902	11
32	Kurdsat (1) 136	853,675	3,627	42
33	Kurdsat (2) 138	423,543	4,275	101
34	Sardaw 140	851,528	5,362	63
35	Sarwari 142	581,454	5,497	95
36	Zerin(Zargatay kon) 144	445,677	5,910	133
37	Hemin 146	546,271	4,921	90
38	Nergz(Kani Kurda) 148	339,070	2,204	65
39	Kwestan 150	375,452	4,300	115
40	Naghada 152	980,089	3,345	34
41	Farmanbaran 154	487,903	6,150	126
42	Bekas 156	490,005	5,340	109
43	Kalakn(Mayani Daraka) 158	663,467	3,435	52
44	Bastan 160	616,610	4,769	77

Table (A.3)

No.	District Name/ Number	Area, m^2	Population (Capita)	Pop Density (Capita/ha)
45	Gundi Kalakn(1) 162	422,660	7,819	185
46	Peramagrwn 164	719,464	3,989	55
47	Zirak 168	335,527	6,160	184
48	Baxtawari 170	371,540	1,565	42
49	Chnarok 172	746,714	8,317	111
60	Bazrgani 201	180,515	1,776	98
51	Dargazen 202	109,697	1,968	179
52	Shexan 203	214,983	3,067	143
53	Sabonkaran 204	308,149	11,040	358
54	Kaneskan 205	349,511	9,775	280
55	Malkani 206	414,216	12,053	291
56	Grdi joga 207	204,599	2,066	101
57	Guyzha 208	340,023	9,818	289
58	Sulaimani taza 209	396,651	8,724	220
59	Darwgha 210	369,428	10,435	282
60	Twi malek 211	460,396	12,420	270
61	Ali kamal 212	385,418	12,005	311
62	Majid Bag (2) 213	371,854	14,322	385
63	Shahidan 214	382,846	13,103	342
64	Majid Bag (1) 215	313,864	6,546	209
65	Azadi (1) 216	418,231	13,261	317
66	Azmar 217	599,419	8,502	142
67	Hawara Barza 218	679,846	22,152	326
68	Hawari taza 219	294,538	7,021	238
69	Guyzhay taza 220	261,473	8,429	322
70	Nali (Gundi Almani) 221	596,438	1,650	28
71	Azadi (2) 222	641,063	17,899	279
72	Chiya Guyzha 223	1,752,730	7,328	42
73	Ibrakem Ahmed 224	446,907	9,041	202
74	Mahwi(Zhala) 226	750,266	659	9
75	Bahashti Shar(1) 228	1,278,896	124	1
76	Bahashti Shar(2) 230	474,798	-	0
77	Kaziwa 234	1,426,241	6,147	43
78	Saywan 301	383,693	11,058	288
79	Sarshaqam(1) 302	208,513	7,855	377
80	Xabat(1)304	307,816	10,799	351
81	Rozh halat(1) 305	553,189	13,904	251
82	Xabat(2)306	309,908	5,904	191
83	Mama yara 307	227,011	7,295	321
84	Zmnako 308	387,546	10,867	280

Table (A.3)

No.	District Name/ Number	Area, m^2	Population (Capita)	Pop Density (Capita/ha)
85	Rozh halat (2309	524,674	8,041	153
86	Sarshqama(2) (Cholakan) 310	725,291	7,314	101
87	Hiwa 311	934,258	4,870	52
88	Chiya 316	572,582	4,904	104
89	Kani Shakrao 317	533,427	3,181	327
90	Kani Ba 318	662,669	6,861	116
91	Nwaroz 320	640,593	6,870	86
92	Chra xan 322	582,624	7,198	60
93	Asayish323	333,978	2,610	104
94	Balambo(Zerinok) 324	316,216	8,389	107
95	Kora Kazhaw 325	1,040,209	3,639	124
96	Waloba 326	628,361	12,148	78
97	Marden (1) 327	1,134,155	5,944	265
98	Blesa 328	133,679	4,033	35
99	Gundi Qirga (1) 329	229,791	2,354	193
100	Sharafxan(shex abas) 330	577,411	7,343	52
101	Gundi Qirga (2) 331	415,322	2,527	302
102	Shokakani yakgrtw 332	693,732	1,738	102
103	Srwsht 333	1,028,788	7,017	127
104	Xastaxanay Shorsh 334	4,829,240	6,481	61
105	Gola bax 335	669,087	4,565	25
106	Guni Qaratoghan 336	370,147	4,256	68
107	Qasabxanai new338	996,157	7,289	13
108	Gundi Hawana(1) 340	958,183	7,011	68
109	Gundi Hawana(2) 342	756,752	171	115
110	Gundi Hawana(3) 344	640,899	6,029	73
111	Zhalai Sarw 346	445,983	1,965	73
112	Zhalai Xwarw 348	366,278	1,914	2
113	Chwar bax401	567,183	14,106	94
114	Wais 402	352,614	3,370	44
115	Garmiyan 403	146,899	1,432	52
116	Shex mohiden 404	570,457	21,418	249
117	Aw Barek 405	452,926	13,405	96
118	Musherawa 406	483,194	11,108	98
119	Sharawani 407	586,084	18,364	375
120	Rzgari (1) 408	426,957	11,656	296
121	Mawlana 409	1,190,513	3,899	230
122	Ablax (1)410	281,720	2,189	313
123	Chwarchra(1) 411	550,521	2,045	273
124	Chiya 316	572,582	4,904	33

Table (A.3)

No.	District Name/ Number	Area, m^2	Population (Capita)	Pop Density (Capita/ha)
125	Kani Shakrao 317	533,427	3,181	78
126	Kani Ba 318	662,669	6,861	37
127	Rizgari (2) 412	754,123	11,941	158
128	Chwarchra(2) 413	690,624	6,525	94
129	(Ablax 2)414	554,663	3,155	57
130	Chwarchra(3) 415	840,238	4,509	54
131	Peshasazi 416	5,371,205	2,129	4
132	Chwarchra(4) 417	458,401	4,050	88
133	Awbaraw asha spi 418	1,729,130	8,978	52
134	Chwarchra(5) 419	754,936	6,763	90
135	Gundi Kanaswra 420	398,531	7,498	188
136	Gundi Kani Goma 422	413,504	8,212	199
137	Zankoi Slemani Nwe 501	458,401	-	0
138	Sarw Kurdsat 502	1,118,363	3,507	31
139	Gundi Qularaisi Khwarw 503	801,677	10,055	125
140	Gundi Kalakn(2) 504	546,041	7,420	136
141	Gundi Qularaisi sarw 505	773,711	8,087	105
142	Parki Haware Shar 506	4,635,434	100	0
143	Haware Shar, 508	381,695	2,559	67
144	Qaiwan(1) 510	209,209	2,503	120
145	Tavga(1) 512	360,596	5,970	166
146	Qaiwan(2) 514	686,973	4,932	72
147	Tavga(2) 516	457,970	4,803	105
148	518	1,364,745	771	6
149	520	1,496,305	423	3
150	Gundi Xewata 524	447,682	3,744	84
151	Gundi Mala Daood 526	33,822	141	42
152	Gundi Kani Bardena 528	851,711	890	10
153	Gundi Fayal 530	1,548,672	9,712	63
154	Mwkryan 701	1,119,706	8,430	75
155	Aso 703	609,455	4,226	69

Table (A.4a): Depths of the Sewer Box - Line A at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
A1	1161	860	830	2.5	3	4.50	
	368	830	820	2.5		3.70	NA1
A2	873	895	860	2.5	5	4.80	
	780	860	835	2.5		6.30	NA2
	231	835	820	2.5		4.00	NA3
A3	552	840	830	1.5	4	6.50	
A4	315	830	820	1.5		7.60	NA4
A5	504	820	806	2.5		4.80	NA5
A6	643	806	805	2.5		7.10	NA6
	876	805	780	2.5		4.00	NA7

Table (A.4b): Depths of the Sewer Box - Line B at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
B1	571	900	884	2.0	4	3.15	NB1
	770	884	857	2.0		3.10	
B2	273	870	857	1.0	4	2.10	NB2
B3	1425	857	840	2.0		5.20	NB3
	249	840	817	2.0		3.50	
B3-1	640	817	797	2.5		5.00	NB4
	818	797	775	2.5		4.25	NB5
B4	504	790	775	1.0	4	2.60	
B5	268	775	770	2.5		4.00	NB6
	673	770	760	2.5		4.00	
B6	697	800	790	1.0	2	2.80	NB7
B7	375	810	790	1.0	5	3.00	
B8	545	790	775	1.0		3.00	NB8
	845	775	760	1.0		2.50	
B9	248	790	775	1.0	4	3.65	
B10	221	780	775	1.0	3	4.50	
B11	183	775	770	2.0		3.80	NB9
	244	770	765	2.0		3.20	NB10
	234	765	760	2.0		3.50	
B12	155	760	757	1.5		4.00	
B13	86	757	755	2.5		4.00	NB11

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name.

Table (A.4b)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
	570	755	750	2.5		4.50	
B14	156	757	750	2.0	3	3.60	
B15	991	750	737	2.5		4.40	NB12
	2529	737	715	3.5		5.00	

Table (A.4c): Depths of the Sewer Box Line - C at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
C1	168	878	875	1.0	2	3.40	
C2	506	880	853	1.2	3	2.50	
C3	435	870	853	1.2	3	2.40	
C4	150	853	850	1.5		2.80	NC1
	252	850	837	1.5		3.20	
C5	243	850	842	1.0	2	2.30	
C6	420	842	830	1.5		3.75	
C7	662	850	830	1.2	2	3.00	
C8	558	830	818	1.5		2.50	NC2
C9	906	940	910	1.5	2	2.50	NC3
C10	226	925	910	1.0	4	2.40	
C11	840	910	883	2.0		3.60	NC4
	1556	883	833	2.0		3.40	NC5
	473	833	817	2.0		3.50	NC6
C12	421	817	810	3.0		4.80	
C13	96	810	810	1.0	4	5.80	
C14	982	810	790	3.0		4.50	NC7
	530	790	780	3.0		4.00	NC8
	513	780	769	3.0		4.30	NC9
	192	769	765	3.0		4.50	
C15	294	847	840	1.0	3	3.50	NC10
	266	840	833	1.0		2.30	NC11
C16	294	850	833	1.0	5	3.70	
C17	2063	833	790	2.0		4.00	NC12
	620	790	775	2.0		4.50	NC13
	522	775	765	2.0		4.90	

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name.

Table (A.4c)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
C18	257	765	763	3.0		6.00	NC14
	510	763	755	3.0		4.00	NC15
	462	755	750	3.0		4.20	NC16
C19	589	760	749	2.0	3	5.80	
C20	241	749	745	2.2		4.00	NC17
C21	303	763	759	1.0	2	2.65	
C22	833	759	745	2.2		3.60	
C23	573	745	735	2.2		3.80	
C24	1064	787	765	2.5	3	3.75	
	554	765	755	2.5		4.70	NC18
	544	755	750	2.5		4.00	NC19
	421	750	735	2.5		4.60	NC20
C25	484	737	725	2.5		3.80	NC21

Table (A.4d): Depths of the Sewer Box - Line D at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
D1	947	754	735	2.0	3.00	3.00	ND1

Table (A.4e): Depths of the Sewer Box line - E at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
E1	1174	1000	945	2.0	3.5	3.35	NE1
	1244	945	885	2.0		4.30	NE2
	624	885	863	2.0		4.00	NE3
	756	863	840	2.0		3.80	NE4
	660	840	825	2.0		4.00	
E2	2045	1035	920	2.0	4	3.50	NE5
	420	920	890	2.0		3.90	NE6
	154	890	890	2.0		4.65	
E2-1	1251	890	840	2.5		3.90	NE7
E3	297	853	845	2.5	4	4.40	NE8
	310	845	840	2.5	2	5.60	
E4	538	840	820	2.5		4.20	NE9
E5	452	820	810	2.5		4.60	
E6	880	930	880	1.5	3	2.90	NE10
	650	880	857	1.5		2.60	
E7	942	965	920	1.5	2	2.80	NE11
	324	920	905	1.5		3.00	

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name

Table (A.4c)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
E8	906	950	905	1.0	2	2.40	
E9	316	905	893	1.5		3.00	NE12
	506	893	875	1.5		2.60	
E10	856	950	910	2.5	4	4.20	
E11	247	955	945	1.0	2	2.40	NE13
	884	945	910	1.0	3.5	3.10	
E12	341	910	900	2.5		3.60	
E13	1277	1025	950	2.5	3.5	3.80	NE14
	833	950	908	2.5		4.40	
E14	394	918	908	1.0	4.5	5.40	
E15	828	908	900	2.5		6.00	NE15
E16	549	900	882	2.5		5.50	
E17	224	887	882	2.0	3.5	3.20	
E18	643	882	875	2.5		5.60	
E19	230	875	865	2.5		4.80	NE16
	299	865	857	2.5		4.30	
E20	541	857	840	2.5		3.50	NE17
	850	840	815	2.5		4.00	NE18
	171	815	810	2.5		4.20	
E21	404	810	800	2.5		3.90	NE19
	563	800	785	2.5		4.60	NE20
	592	785	775	2.5		4.70	NE21
	631	775	765	2.5		4.20	
E22-1	662	837	820	1.1	3.5	4.10	
E22	1233	820	788	1.5		4.20	
E23	490	795	788	1.0	2.5	2.40	
E24	511	788	775	2.0		4.00	NE22
	486	775	765	2.0		3.70	
E25	1270	765	745	2.0		4.00	NE23
	519	745	735	2.0		4.40	NE24
E25-1	1023	735	715	2.8		3.80	NE25

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name

Table (A.4f): Depths of the Sewer Box F - at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
F1	769	850	826	1.5	2	2.60	NF1
F2	497	825	826	1.0	1.5	6.00	
F3	494	830	820	1.5		3.00	NF2
F4	401	843	833	1.5	6	6.30	NF3
	738	833	820	1.5		6.60	
F5	585	820	810	3.0		4.80	NF4
	312	810	797	3.0		4.60	NF5
	936	797	780	3.0		4.45	NF6
	380	780	767	3.0		4.75	NF7
	464	767	757	3.0		5.00	NF8
	1024	757	740	3.0		4.00	NF9
F6	1213	765	740	1.0	4	4.00	
F7	1924	740	725	3.0		4.70	NF10
	1286	725	705	4.0		5.00	NF11

Table (A.4g): Depths of the Sewer Box Line - G at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
G1	482	1010	1012	1.0	2	7.40	NG1
	1949	1012	993	1.0		4.00	
G1-1	439	993	970	2.0		4.00	NG2
	1283	985	895	2.0		3.50	NG3
G2	192	1005	985	1.0	6	2.30	
	192	985	965	1.0	5	2.60	
	192	965	945	1.0	4.5	2.50	NG4
	192	945	940	1.0		3.20	
G3	121	940	940	1.0	2.5	3.75	
G4	362	940	933	1.0		3.30	NG5
G5	513	995	955	1.0	4.5	2.45	NG6
G6	543	995	955	1.0	2.5	2.60	
G7	365	955	933	1.0		2.50	

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name

Table (A.4g)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
G8	241	933	920	3.0		4.30	NG7
	1004	920	895	3.0		4.40	NG8
	328	895	893	3.0		5.00	NG9, NG10
G9	328	893	877	3.0		5.43	
G10	725	945	893	1.5	4	2.80	
G11	502	920	893	2.5	2	3.60	
G12	209	893	885	3.0		4.40	NG11
	161	885	877	3.0		4.40	
G13	102	877	873	2.5		3.80	
	351	873	860	2.5		4.20	NG12
G14	945	885	877	3.0	2	6.45	NG13
	617	877	860	3.0		4.90	NG14
G15	236	860	853	2.5		4.00	NG15
	451	853	837	2.5		5.15	NG16
	947	837	807	2.5		4.50	
G16	936	835	807	2.5	3	3.70	
G17	247	807	803	2.5		3.90	NG17
G18	619	823	803	2.5	3	4.00	NG18
G19	246	803	797	3.0		4.70	NG19
	740	797	775	3.0		5.40	NG23
	1367	775	745	3.0		4.60	NG24
G20	340	867	865	1.0	2	3.70	
G21	557	865	850	1.0		2.00	NG20
G22	434	857	850	1.0	2	2.50	
G23	391	850	835	1.0		2.40	NG21
G24	222	870	863	2.5	3.5	4.15	
	1052	863	835	2.5		4.50	
G25	1362	835	795	2.5		5.40	NG22
G26	785	820	795	2.0	2.5	3.00	
G27	2178	795	745	2.5		3.65	NG23

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name

Table (A.4h): Depths of the Sewer Box Line - H at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
H1	1058	1105	1025	2.5	4	5.85	
H2	668	1075	1025	2.0	5	7.10	
H3	764	1025	970	2.5		5.60	NH1
	600	970	930	2.5		4.60	NH2
	732	930	890	2.5		4.85	
H4	755	930	890	1.0	3	3.30	NH3
H5	167	890	880	2.5		4.00	NH5
H6	2204	955	900	1.5	2	5.80	NH4
	262	900	880	1.5		4.60	
H7	940	880	852	2.5		5.90	NH6
	850	852	820	2.5		4.40	NH7
	554	820	805	2.5		4.90	NH8
	301	805	795	2.5		5.45	NH9
	650	795	775	2.5		2.70	NH10
	344	775	780	2.5		9.40	NH11
	327	780	775	2.5		6.00	NH12

Table (A.4i): Depths of the Sewer Box Line - I at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
I1	850	930	885	2.0	3	3.35	NI1
I2	740	928	885	1.5	3	2.95	
I3	1066	885	844	2.0		4.00	NI2
	1304	844	805	2.0		4.10	NI3
	414	805	797	2.0		3.60	NI4
	300	797	795	2.0		3.70	NI5
	190	795	787	2.0		3.30	
I4	175	825	815	1.5	2	2.80	NI6
	384	815	802	1.5		3.30	NI7
	234	802	793	1.5		2.65	
I5	175	834	830	1.5	3	3.15	NI8
	433	830	810	1.5		3.50	NI9
	292	810	793	1.5		3.15	

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name

Table (A.4i)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
I6	278	793	788	2.0	2	3.65	NI10
	410	788	787	2.0		4.65	NI11
I7	430	787	777	2.0		3.70	NI12
I8	1054	845	810	1.5	2.5	2.70	NI13
	308	810	797	1.5		2.65	NI14
	514	797	788	1.5		2.90	NI15
	359	788	777	1.5		2.70	
I9	161	777	773	2.0		3.50	NI16
	734	773	750	2.0		3.50	NI17

Table (A.4j): Depths of the Sewer Box Line - J at Nominated Areas, (Researcher)

Line	Length, m	S.L[a]	E.L[b]	Height, m	S.D[c]	E.D[d]	NA[e]
J1	60	929	924	1.0	4	3.45	
	51	924	921	1.0		3.25	
	50	921	919	1.0		4.00	
J1	696	930	895	2.0	3	3.40	NJ1
	714	895	867	2.0		4.00	NJ2
	1447	867	815	2.0		4.00	
J2	512	835	820	1.0	2	3.40	
J3	357	835	820	1.0	2	2.30	
J4	269	820	816.6	1.0		3.25	NJ3
J5	5520	805	660	2.0	2	3.65	NJ4

a; S.L. = Start Ground Level, b; E.L. = End Ground level , c; S.D.=Start depth of the Sewer Box, d; E.D=End depth of the Sewer Box, e; NA = Nominated Area Name

Table (A.5a) : Area Suitability (m^2) of Nominated Areas of Line A, (Researcher)

NA[a]	R[b]	M.S[c]	S[d]	V.S[e]	H.S[f]	E.S[g]	Total Area, m^2
NA1	4,147	0.0	0.0	2,488	905	0.0	7,540
NA2	1,624	0.0	0.0	3,519	271	433	5,413
NA3	574	0.0	3,442	344	1,205	172	5,736
NA4	1,153	0.0	412	6,506	0.0	165	8,236
NA5	726	0.0	0.0	111	1,0978	0.0	11,815
NA6	3,085	0.0	0.0	1,311	1,774	1,542	7,712

a : NA= Nominated Area, b: R = Restricted, c; M.S= Moderately Suitable, d; S = Suitable, e; V.S. = Very Suitable, f; H.S. = Highly Suitable, g; E,S. = Extremely Suitable,

Table (A.5b) : Area Suitability (m^2) of Nominated Areas of Line B, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NB1	0.0	0.0	0.0	3797	936	468	5,202
NB2	0.0	0.0	1020	2980	3843	0.0	7,843
NB3	307	0.0	0.0	0.0	4353	461	5,121
NB4	223	0.0	0.0	4755	1932	520	7,430
NB5	8	0.0	0.0	0.0	8328	0.0	8,337
NB6	0.0	0.0	51	3687	1383	0.0	5,121
NB7	277	0.0	222	1885	3160	0.0	5,544
NB8	0.0	0.0	0.0	5665	337	742	6,744
NB9	10	0.0	4910	0.0	0.0	0.0	4,919
NB10	31	0.0	698	1397	2239	0.0	4,365
NB11	0.0	0.0	0.0	402	7472	161	8,034
NB12	285	0.0	0.0	0.0	0.0	8921	9,207
NB13	0.0	0.0	1845	1054	2373	0.0	5,272

Table (A.5c) : Area Suitability (m^2) of Nominated Areas of Line C, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NC1	238	59	0.0	2,676	0.0	0.0	2,974
NC2	0.0	0.0	202	3,234	606	0.0	4,042
NC3	434	0.0	217	3,693	0.0	0.0	4,345
NC4	0.0	0.0	3,974	729	1,921	0.0	6,623
NC5	0.0	0.0	1,887	1,234	508	0.0	3,629
NC6	0.0	0.0	1,133	1,700	0.0	0.0	2,833
NC7	0.0	160	763	334	518	0.0	1,774
NC8	0.0	0.0	0.0	3,389	424	39	3,851
NC9	0.0	0.0	538	1,166	90	0.0	1,794
NC10	0.0	0.0	851	0.0	500	0.0	1,351
NC11	262	0.0	3,065	0.0	0.0	0.0	3,327
NC12	81	0.0	230	1,793	1,041	0.0	3,145
NC13	0.0	0.0	2,165	1,499	500	0.0	4,163
NC14	0.0	0.0	1,223	2,598	0.0	0.0	3,821
NC15	0.0	0.0	0.0	754	2,523	0.0	3,276
NC16	0.0	1099	0.0	1,744	948	0.0	3,790
NC17	0.0	1545	0.0	3,522	1,112	0.0	6,180
NC18	0.0	0.0	1,052	2,526	0.0	0.0	3,508

NA= Nominated Area, b: R = Restricted, c; M.S= Moderately Suitable, d; S = Suitable, e; V.S. = Very Suitable, f; H.S. = Highly Suitable, g; E,S. = Extremely Suitable,

Table (A.5c)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NC19	0.0	0.0	2,242	285	1032	0.0	3,559
NC210	0.0	0.0	3,734	1245	948	0.0	5,928
NC21	0.0	105	3,348	0.0	1,570	209	5,232

Table (A.5d) : Area Suitability (m^2) of Nominated Areas of Line D, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
ND1	0.0	0.0	0.0	0.0	0.0	10,686	10,686

Table (A.5e) : Area Suitability (m^2) of Nominated Areas of Line E, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NE1	30	0.0	0.0	1139	3,780	0.0	4,950
NE2	585	0.0	0.0	5,444	0.0	0.0	6,028
NE3	313	0.0	3,557	0.0	578	0.0	4,446
NE4	0.0	0.0	832	2495	0.0	0.0	3,327
NE5	0.0	0.0	905	1,590	247	0.0	2,742
NE6	480	0.0	0.0	2,194	651	103	3,427
NE7	32	0.0	2,237	0.0	671	256	3,196
NE8	0.0	0.0	0.0	844	4,781	0.0	5,625
NE9	261	0.0	1,417	1,343	709	0.0	3,730
NE10	0.0	0.0	1,260	5,403	0	0.0	6,663
NE11	65	0.0	500	677	371	0.0	1,613
NE12	171	0.0	2,026	0.0	0	0.0	2,198
NE13	252	0.0	0.0	3,347	0	0.0	3,599
NE14	0.0	471	1,020	1,843	588	0.0	3,921
NE15	147	0.0	339	471	515	0.0	1,472
NE16	0.0	0.0	331	331	5,952	0.0	6,613
NE17	0.0	0.0	318	1063	0	0.0	1,381
NE18	54	0.0	295	590	1,743	0.0	2,681
NE19	299	0.0	0.0	2635	60	0.0	2,994
NE20	0.0	0.0	385	1540	0	0.0	1,925
NE21	0.0	1147	0.0	2248	1,193	0.0	4,587
NE22	2,651	0.0	0.0	0.0	6,200	0.0	8,851
NE23	1,401	0.0	0.0	0.0	10,262	0.0	11,663
NE24	1,043	0.0	2,310	0.0	4,099	0.0	7,453

a : NA= Nominated Area, b: R = Restricted, c; M.S= Moderately Suitable, d; S = Suitable,
e; V.S. = Very Suitable, f; H.S. = Highly Suitable, g; E,S. = Extremely Suitable,

Table (A.5f) : Area Suitability (m^2) of Nominated Areas of Line F, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NF1	81	0.0	0.0	0.0	696	12,551	13,327
NF2	0.0	0.0	0.0	0.0	5,445	3,063	8,508
NF3	0.0	0.0	333	1,018	0.0	0.0	1,351
NF4	71	0.0	0.0	1,373	3,373	0.0	4,819
NF5	146	0.0	0.0	438	3,065	0.0	3,649
NF6	0.0	0.0	0.0	94	2,872	1,742	4,708
NF7	42	0.0	0.0	3,779	0.0	332	4,153
NF8	0.0	0.0	0.0	169	4,827	3,472	8,468
NF9	0.0	0.0	0.0	2,341	1,040	954	4,335
NF10	186	466	0.0	2,608	3,912	2,142	9,315
NF11	0.0	0.0	231	1,978	1,088	0.0	3,296
NF12	111	0.0	0.0	4,012	16,460	0.0	20,575

Table (A.5g) : Area Suitability (m^2) of Nominated Areas of Line G, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NG1	0.0	514	1,028	2,134	277	0.0	3,952
NG2	589	0.0	0.0	2,550	6,670	0.0	9,809
NG3	0.0	223	556	0.0	4,229	556	5,565
NG4	153	0.0	306	3,362	0.0	0.0	3,821
NG5	132	182	0.0	1,339	0.0	0.0	1,653
NG6	74	0.0	1,450	121	290	0.0	1,936
NG7	50	565	595	0.0	0.0	0.0	1,210
NG8	20	0.0	886	101	496	0.0	1,502
NG9	27	0.0	1,523	0.0	1,122	0.0	2,671
NG10	0.0	0.0	2,520	1,079	1,470	0.0	5,071
NG11	139	0.0	696	1,447	501	0.0	2,782
NG12	330	0.0	165	2,472	330	0.0	3,296
NG13	97	682	0.0	1,493	974	0.0	3,246
NG14	0.0	0.0	315	1,258	0.0	0.0	1573
NG15	0.0	251	879	1,381	0.0	0.0	2,510
NG16	50	655	1,482	0.0	0.0	0.0	2,188
NG17	283	0.0	339	5,033	0.0	0.0	5,655
NG18	0.0	812	0.0	4,521	464	0.0	5,796

a : NA= Nominated Area, b: R = Restricted, c; M.S= Moderately Suitable, d; S = Suitable,
e; V.S. = Very Suitable, f; H.S. = Highly Suitable, g; E,S. = Extremely Suitable,

Table (A.5g)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NG19	139	0.0	278	3,061	0.0	0.0	3,478
NG20	0.0	0.0	1,497	3,548	499	0.0	5,544
NG21	122	0.0	1,478	190	922	0.0	2,712
NG22	135	473	0.0	0.0	5,268	878	6,754
NG23	541	0.0	270	4,393	8,314	0.0	13,518
NG24	0.0	0.0	7,379	0.0	3,801	0.0	11,180

Table (A.5h) : Area Suitability (m^2) of Nominated Areas of Line H, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NH1	0.0	0.0	2,520	3,256	0.0	0.0	5,776
NH2	0.0	0.0	0.0	8,568	1,059	0.0	9,627
NH3	104	0.0	1,661	311	0.0	0.0	2,077
NH4	998	0.0	4,123	141	0.0	0.0	5,262
NH5	84	0.0	2,055	1,677	377	0.0	4,194
NH6	0.0	276	829	0.0	4,419	0.0	5,524
NH7	177	0.0	0.0	3,009	1,239	0.0	4,425
NH8	0.0	0.0	3,339	629	871	0.0	4,839
NH9	509	0.0	1,908	3,944	0.0	0.0	6,361
NH10	0.0	0.0	1,489	1,747	0.0	0.0	3,236
NH11	762	0.0	1,385	4,363	413	0.0	6,926
NH12	1,833	0.0	2,596	9,622	1,222	0.0	15,272

Table (A.5i) : Area Suitability (m^2) of Nominated Areas of Line I, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NI1	163	0.0	373	1,723	70	0.0	2,329
NI2	207	0.0	0.0	0.0	1,506	0.0	1,714
NI3	96	0.0	1,266	334	693	0.0	2,389
NI4	0.0	0.0	0.0	4,926	4,368	0.0	9,295
NI5	0.0	753	251	3,514	502	0.0	5,020
NI6	0.0	0.0	688	213	350	0.0	1,250
NI7	76	248	0.0	1,619	0.0	0.0	1,905
NI8	0.0	0.0	2,835	232	1,580	0.0	4,647
NI9	89	0.0	254	762	165	0.0	1,270
NI10	0.0	0.0	512	563	205	0.0	1,280

a : NA= Nominated Area, b: R = Restricted, c; M.S= Moderately Suitable, d; S = Suitable,
e; V.S. = Very Suitable, f; H.S. = Highly Suitable, g; E,S. = Extremely Suitable,

Table (A.5i)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NI11	10	615	1139	0.0	0.0	0.0	1,764
NI12	0.0	0.0	285	854	0.0	0.0	1,139
NI13	334	0.0	1,502	100	1,401	0.0	3,337
NI14	107	0.0	1,288	0.0	558	193	2,147
NI15	0.0	0.0	0.0	118	2,251	0.0	2,369
NI16	36	0.0	1,142	607	0.0	0.0	1,784
NI17	0.0	437	3,058	312	2,434	0.0	6,240

Table (A.5j) : Area Suitability (m^2) of Nominated Areas of Line J, (Researcher)

NA [a]	R [b]	M.S [c]	S [d]	V.S [e]	H.S [f]	E.S [g]	Total Area, m^2
NJ1	0.0	1,663	0.0	11,582	971	0.0	14,214
NJ2	0.0	0.0	0.0	2,841	5,406	916	9,163
NJ3	356	713	0.0	1,069	7,636	407	10,182
NJ4	9161	0.0	0.0	458	36,188	0.0	45,807

a : NA= Nominated Area, b: R = Restricted, c; M.S= Moderately Suitable, d; S = Suitable,
e; V.S. = Very Suitable, f; H.S. = Highly Suitable, g; E,S. = Extremely Suitable,

Table (A.6): Normalized WAV of the Nominated Areas, (Researcher)

NA [a]	WAV [b]	NWAV [c]	NA [a]	WAV [b]	NWAV [c]
NA1	10	0.00	NB10	22	0.48
NA2	17	0.47	NB11	26	0.70
NA3	16	0.39	NB12	32	1.00
NA4	17	0.48	NB13	21	0.40
NA5	25	1.00	NC1	18	0.47
NA6	16	0.41	NC2	21	0.67
NB1	22	0.49	NC3	18	0.44
NB2	22	0.49	NC4	18	0.46
NB3	26	0.66	NC5	17	0.42
NB4	22	0.47	NC6	17	0.41
NB5	27	0.71	NC7	18	0.45
NB6	22	0.45	NC8	21	0.68
NB7	23	0.49	NC9	18	0.49
NB8	22	0.46	NC10	18	0.48
NB9	13	0.00	NC11	12	0.00

a; NA = Nominated Areas, b; WAV = Weighted Average Value %,
c; NWAV = Normalized weighted Average Value

Table (A. 6)

NA [a]	WAV [b]	N. WAV [c]	NA [a]	WAV	N. WAV [c]
NC12	21	0.71	NF4	24	0.43
NC13	17	0.41	NF5	25	0.46
NC14	18	0.45	NF6	29	0.74
NC15	25	1.00	NF7	21	0.19
NC16	18	0.45	NF8	29	0.76
NC17	18	0.45	NF9	25	0.44
NC18	18	0.49	NF10	25	0.46
NC19	18	0.44	NF11	22	0.25
NC20	17	0.37	NF12	25	0.49
NC21	18	0.46	NG1	17	0.39
ND1	33	1.00	NG2	23	0.74
NE1	25	0.93	NG3	25	0.84
NE2	18	0.43	NG4	19	0.48
NE3	14	0.15	NG5	17	0.39
NE4	18	0.45	NG6	15	0.29
NE5	18	0.46	NG7	10	0.00
NE6	19	0.49	NG8	18	0.44
NE7	18	0.40	NG9	19	0.49
NE8	26	0.98	NG10	19	0.48
NE9	17	0.38	NG11	19	0.47
NE10	19	0.48	NG12	18	0.46
NE11	19	0.48	NG13	19	0.48
NE12	12	0.00	NG14	19	0.48
NE13	19	0.47	NG15	16	0.35
NE14	18	0.40	NG16	11	0.06
NE15	19	0.49	NG17	19	0.48
NE16	26	0.98	NG18	19	0.48
NE17	18	0.46	NG19	19	0.48
NE18	23	0.80	NG20	19	0.49
NE19	18	0.44	NG21	18	0.43
NE20	19	0.48	NG22	26	0.87
NE21	18	0.46	NG23	23	0.73
NE22	19	0.48	NG24	18	0.44
NE23	23	0.82	NH1	17	0.47
NE24	19	0.49	NH2	21	0.75
NF1	33	1.00	NH3	14	0.21
NF2	29	0.75	NH4	11	0.00
NF3	18	0.00	NH5	17	0.46

a; NA = Nominated Areas, b; WAV = Weighted Average Value %,
c; NWAV = Normalized weighted Average Value

Table (A. 6)

NA [a]	WAV [b]	NWAV [c]	NA[a]	WAV	NWAV [c]
NH6	24	0.97	NI11	11	0.00
NH7	21	0.77	NI12	18	0.49
NH8	17	0.43	NI13	18	0.45
NH9	16	0.42	NI14	18	0.46
NH10	17	0.46	NI15	26	1.00
NH11	17	0.45	NI16	15	0.29
NH12	17	0.46	NI17	18	0.49
NI1	18	0.45	NJ1	19	0.00
NI2	23	0.83	NJ2	25	1.00
NI3	18	0.44	NJ3	24	0.82
NI4	23	0.81	NJ4	21	0.39
NI5	18	0.49			
NI6	18	0.48			
NI7	18	0.46			
NI8	18	0.48			
NI9	18	0.48			
NI10	18	0.49			

a; NA = Nominated Areas, b; WAV = Weighted Average Value %,
c; NWAV = Normalized weighted Average Value

Table (A.7): The Optimized 31 Nominated Areas, (Researcher)

No.	Old NA[a] Name	Optimized NA	Sewer Line
1	NA5	OA1	**A**
2	NB3	OB1	**B**
3	NB5	OB2	
4	NB11	OB3	
5	NB12	OB4	
6	NC2	OC1	**C**
7	NC8	OC2	
8	NC12	OC3	
9	NC15	OC4	
10	ND1	OD1	**D**
11	NE1	OE1	**E**
12	NE8	OE2	
13	NE16	OE3	
14	NE18	OE4	
15	NE23	OE5	

a; NA = Nominated Areas,

Table (A. 7)

No.	Old NA[a] Name	Optimized NA	Sewer Line
16	NF1	OF1	**F**
17	NF2	OF2	
18	NF6	OF3	
19	NF8	OF4	
20	NG2	OG1	**G**
21	NG3	OG2	
22	NG22	OG3	
23	NG23	OG4	
24	NH2	OH1	**H**
25	NH6	OH2	
26	NH7	OH3	
27	NI2	OI1	**I**
28	NI4	OI2	
29	NI15	OI3	
30	NJ2	OJ1	**J**
31	NJ3	OJ2	

a; NA = Nominated Areas,

Table (A.8a): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line A, (Researcher)

ONA[a]	District Names	Pop.[b]	Area $m^2 \times 10^3$	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OA1	Qaiwan 514	4,932	686	686,973	1.0	4,932	986
	Hawari Shar 508	2,559	381,695	381,695	1.0	2,559	512
	Qaiwan 510	2,503	209,209	209,209	1.0	2,503	501
	Chnarol 172	8,317	746,714	485,364	0.7	5,406	1,081
						Total Flow	**3,080**

a;ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area,
d; Q_{av} = Average Daily Flow

Table (A.8b): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line B, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av}[d] m^3/d
OB1	Gundi Kalakn504	7,420	546,041	546,041	1.0	7,420	1,484
	Gundi Kalakn162	7,819	422,660	346,581	0.8	6,412	1,282
	Zirak 168	6,160	126,000	126,000	1.0	6,160	1,232
	Peramagrwn 164	3,989	719,464	719,464	1.0	3,989	798
	Baxtawari 170	1,565	371,540	222,924	0.6	939	188
						Total Flow	**4,984**
OB2	Baxtawari 170	1,565	371,540	148,616	0.4	626	125
	farmanbaran 154	6,150	487,903	195,161	0.4	2,460	492
	Bekas 156	5,340	490,005	490,005	1.0	5,340	1,068
	chnarok 172	8,317	746,714	224,014	0.3	2,495	499
	Naghada 152	3,345	980,089	539,049	0.6	1,840	368
						Total Flow	**2,552**
OB3	Naghada 152	3,345	980,089	343,031	0.45	1,505	301
	Kani speka 130	20,275	537,720	188,202	1.00	20,275	4,055
	Farmanbaran 154	6,150	487,903	292,742	0.60	3,690	738
	Kwestan 150	4,300	375,452	375,452	1.00	4,300	860
	Nergz 148	2,204	339,070	339,070	1.00	2,204	441
	Mashxalan 128	11,321	431,491	215,745	1.00	11,321	2,264
	Sarchinar 119	9,036	541,747	151,689	0.28	2,530	506
	Sarchinar 121	11,441	379,292	113,788	1.00	11,441	2,288
						Total Flow	**11,453**
OB4	Harawazi 123	14,401	1,237,340	915,632	0.7	10,637	2,127
						Total Flow	**2,127**

a;ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area, d; Q_{av} = Average Daily Flow

Table (A.8c): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line C, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OC1	Hemin 146	4,921	546,271	300,449	1.00	4,921	984
	Zerin 144	5,910	445,677	200,555	0.72	4,255	851
					Total Flow		**1,835**
OC2	Kalakn 158	3,435	663,467	663,467	1.00	3,435	687
	Kurdsat 138	4,275	423,543	220,243	0.52	2,223	444
	Bastan 160	4,769	616,610	616,609	1.00	4,769	953
	garaKi 162	7,819	422,660	77,206	0.18	1,428	285
	Sardaw 140	5,362	581,454	261,654	0.45	2,413	482
	Rozh City						807
	Zrein 144	5,910	445,677	124,789 .	0.28	1,655	330
	Sarwari 142	5,497	581,454	314,209	1.00	5,497	1,099
	Zargata 126	19,356	855,239	427,619	0.50	9,678	1,935
	Sarchnar 119	9,036	541,747	119,184 .	0.72	6,506	1,301
	New Baxtiyari 117	16,095	409,050	224,977	0.55	8,852	1,770
	Jaff Towers					-	217
					Total Flow		**10,316**
OC3	Badinan 124	7,477	382,643	210,454	1.00	7,477	1,495
	Besarani 122	16,241	538,511	296,181	1.00	16,241	3,248
	Shirwana 115	1,000	376,513	376,513	1.00	1,000	200
	Hakari 113	9,209	305,502	305,502	1.00	9,209	1,842
	Zargata 126	19,356	855,239	427,619	0.50	9,678	1,936
					Total Flow		**8,721**
OC4	New Baxtiyari 117	16,095	409,050	184,073	0.45	7,243	1,449
	Baxtiyari 111	3,984	409,050	143,168	1.00	3,984	797
	Handren 109	4,635	315,088	315,088	1.00	4,635	927
	Harawazi 123	14,401	1,237,340	125,000	0.16	2,286	457
	Pak City						672
	Baharn City						91
					Total Flow		**4,393**

a; ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area, d; Q_{av} = Average Daily Flow

Table (A.8d): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line D, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OD1	Shakraka 125	9,745	304,082	304,082	1.0	9,745	1,949
	Hawarazi 123	14,401	1,237,340	289,923	0.23	3,374	675
						Total Flow	**2,624**

Table (A.8e): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line E, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OE1	502 Zone	3,507	1,118,363	1,118,363	1.00	3,507	701
	Kurdsat 136	3,627	853,675	85,367	0.10	363	73
	Kurdsat 138	4,275	423,543	101,650	0.24	1,026	205
	Barzaiakani Slemani						2,200
						Total Flow	**3,179**
OE2	Kurdsat 138	4,275	423,543	101,650	0.24	1,026	205
	Kurdsat 136	3,627	853,675	128,051	0.90	3,265	653
	Sardaw 140	5,362	851,528	468,340	0.55	2,949	590
	Swren 120	12,185	483,331	314,165	1.00	12,185	2,437
	Kareza Wshk 118	5,956	301,305	147,639	1.00	5,956	1,191
	Baxan 108	5,170	746,290	149,258	0.55	2,843	569
	Garden City						662
	Xwar Kurdsat 134	1,902	1,683,354	1,683,354	0.99	1,883	377
	Kareza Wshk 114	11,936	307,684	307,684	1.00	11,936	2,387
	Ashti 106	11,954	343,767	25,000	0.61	7,324	1,465
	Ashti 104	9,985	454,686	93,000	0.38	3,794	759
						Total Flow	**11,295**

a; ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area, d; Q_{av} = Average Daily Flow

Table (A. 8e)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av}[d] m^3/d
OE3	Dabashan 116	11,160	613,528	515,363	1.00	11,160	2,232
	Haware Taza 219	7,021	294,538	60,000	1.00	7,021	1,404
	Majid Bag 215	6,546	313,864	235,398	1.00	6,546	1,309
	Majid Bag 213	14,322	371,854	70,652	0.19	2,721	544
	Gundi, Almani						1,723
	Nali 221	1,650	60,000	60,000	1.00	1,650	330
	Shary Daik						365
	Hawara Barza 218	22,152	679,846	535,000	1.00	22,152	4,430
	Ali Kamal 212	12,005	385,418	385,418	1.00	12,005	2,401
	Twi Malik 211	12,420	460,396	276,238	1.00	12,420	2,484
	Azmar 217	8,502	599,419	599,419	1.00	8,502	1,700
	Mamostayan 112	8,322	288,524	237,470	1.00	8,322	1,664
	Qazi Mohammed 110	10,414	434,803	86,961	0.20	2,083	417
OE3	Ashti 104	9,985	454,686	45,469	0.10	998	200
						Total Flow	**21,204**
OE4	Ashti 104	9,985	454,686	227,343	0.50	4,992	998
	Bakhan 108	5,170	746,290	335,830	0.45	2,326	465
	Baranan 107	5,864	403,098	307,000	0.76	4,466	893
	Ashti 106	11,954	343,767	134,069	0.39	4,662	932
						Total Flow	**3,288**
OE5	Baranan 107	5,864	403,098	96,000	0.24	1,397	279
	Andazyran 105	4,199	361,515	234,984	0.65	2,729	546
	Rizgari 408	11,656	426,957	286,957	0.67	7,834	1,567
	Ablakh 410	2,189	281,720	126,774	0.45	985	197
	Ali Naji 103	5,612	437,352	437,352	1.00	5,612	1,122
	Shorsh 101	8,115	534,268	363,302	0.68	5,518	1,104
	Qazi Mihamed 110	10,414	434,803	347,843	0.80	8,331	1,666
	Raparin 102	6,510	695,358	347,679	0.50	3,255	651
	Shekh Mohiden 404	21,418	570,457	570,457	1.00	21,418	4,284
	Mushirawa 406	11,108	483,194	483,194	1.00	11,108	2,222
	Mazari Shahid Jabar 414	3,155	554,663	375,000	0.68	2,133	427
	From Chwar Chra New city						550
						Total Flow	**14,614**

a; ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area,
d; Q_{av} = Average Daily Flow

Table (A.8f): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line F, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OF1	New Sulaimani 209	8,724	396,651	396,651	1.0	8,724	1,745
	Kani Askan 205	9,775	349,511	192,231	0.55	5,376	1,075
	Raparin 102	6,510	695,358	347,679	0.50	3,255	651
						Total Flow	**3,471**
OF2	Shorsh 101	8,115	534,268	106,854	0.20	1,623	325
	Kani Askan 205	9,775	349,511	157,280	0.45	4,399	880
						Total Flow	**1,204**
OF3	Shorsh 101	8,115	534,268	64,112	0.12	974	195
	Wais 402	3,370	352,614	352,614	1.00	3,370	674
	Chwar Bakh 401	14,106	567,183	567,183	1.00	14,106	2,821
	Sharawani 407	18,364	586,084	240,294	1.00	18,364	3,673
	Garmeyan 403	1,432	146,899	146,899	1.00	1,432	286
	Awa barik 405	13,405	452,926	452,926	1.00	13,405	2,681
						Total Flow	**10,330**
OF4	Chwar Chra 413	6,525	690,624	241,718	0.70	4,568	914
	Chwar Chra 411	2,045	550,521	275,260	1.00	2,045	409
						Total Flow	**1,323**

a;ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area, d; Q_{av} = Average Daily Flow

Table (A.8g): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line G, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OG1	Mahwi 226	659	750,266	750,266	1.00	659	132
	Kaziwa 234	6,147	1,426,241	350,000	0.25	1,509	302
	Goizha City						1,440
						Total Flow	**1.874**
OG2	Ibrahim Ahmed 224	9,041	446,907	446,907	1.00	9,041	1,808
	Azadi 216	13,261	418,231	154,745	1.00	13,261	2,652
	New Goizhai 220	8,429	261,473	203,949	1.00	8,429	1,686
	Azadi 222	17,899	641,063	108,981	1.00	17,899	3,580
	Shahidan 214	13,103	382,846	99,540	1.00	13,103	2,621
						Total Flow	**12,347**
OG3	Shekhan 203	3,067	214,983	189,185	1.00	3,067	613
	Grdi Joga 207	2,066	204,599	204,599	1.00	2,066	413
	Malkani 206	12,053	414,216	140,833	1.00	12,053	2,411
	Sabwnkaran 204	11,040	308,149	308,149	1.00	11,040	2,208
	Bazrgani 201	1,776	180,515	180,515	1.00	1,776	355
	Sarshaqam 302	7,855	208,513	95,916	0.46	3,613	723
	Sarshaqam 310	7,314	725,291	188,576	0.26	1,902	380
						Total Flow	**7,103**
OG4	Kaziwa 234	6,147	1,426,241	713,121	0.50	3,074	615
	Mama Yara 307	7,295	227,011	227,011	1.00	7,295	1,459
	Rosh Halat 309	8,041	524,674	424,986	1.00	8,041	1,608
	Saywan 301	11,058	383,693	41,019	1.00	11,058	2,212
	Darogha 210	10,435	369,428	118,217	1.00	10,435	2,087
	Goisha 208	9,818	340,023	176,812	1.00	9,818	1,964
	khabat 304	10,799	307,816	169,299	1.00	10,799	2,160
	khabat 306	5,904	309,908	99,171	0.32	1,889	378
	Dargazen 202	1,968	109,697	109,697	1.00	1,968	394
	Sarshaqam 302	7,855	208,513	112,597	0.54	4,241	848
	Sarshaqam 310	7,314	725,291	398,910	1.00	7,314	1,463
	Waluba 326	12,148	628,361	62,836	0.20	2,430	486
						Total Flow	**15,672**

a; ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area,
d; Q_{av} = Average Daily Flow

Table (A.8h): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line H, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OH1	Kwra Kazhaw 325	3,639	1,040,209	859,620	0.83	3,007	601
	Dilan City 1, 2						1,098
	Asaish 323	2,610	333,978	333,978	1.00	2,610	522
	Kani Shakraw 317	3,181	533,427	533,427	1.00	3,181	636
	Danya City						704
					Total Flow		**3,561**
OH2	Hiwa 311	4,870	934,258	344,374	1.00	4,870	974
	Pari 315	6,074	522,765	405,991	1.00	6,074	1,215
	Kwra Kazhaw 325	3,639	1,040,209	176,836	0.17	619	124
	Sana 313	8,650	835,642	835,642	0.75	6,488	1,298
					Total Flow		**3,611**
OH3	Chia 316	4,904	572,582	343,549	0.60	2,943	589
	Kani Ba 318	6,861	662,669	662,669	1.00	6,861	1,372
					Total Flow		**1,960**

Table (A.8i): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line I, (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OI1	Rozh Halat 309	8,041	524,674	419,739	1.00	8,041	1,608
	Kaziwa 234	6,147	1,426,241	356,560	0.25	1,537	307
	Rozh Halat 305	13,904	553,189	553,189	1.00	13,904	2,781
						Total Flow	**4,696**
OI2	Khabat 306	5,904	309,908	210,738	0.68	4,015	803
	Zmnako 308	10,867	387,546	213,150	0.55	5,977	1,195
	Tanjaro 314	19,673	601,360	182,000	0.51	10,033	2,007
						Total Flow	**4,005**

a; ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area,
d; Q_{av} = Average Daily Flow

Table (A. 8i)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OI3	Chia 316	4,904	572,582	229,033	0.40	1,962	392
	Nawroz 320	6,870	640,593	108,901	0.24	1,649	330
	Balambo 324	8,389	316,216	18,973	0.16	1,342	268
	Shoqaqani Yakgrtw 332	1,738	693,732	173,433	0.25	435	87
						Total Flow	**1,078**

Table (A.8j): Available Flow (Q_{av}) at Optimized Nominated Areas of Sewer Line J, , (Researcher)

ONA[a]	District Names	Pop.[b]	Area m^2	Area of Flow, m^2	F[c] %	Pop. of Flow Area	Q_{av} [d] m^3/d
OJ1	Sardam City						308
	Nawzad City						3,850
	Mwkryan 701	8,430	1,119,706	1,119,706	1.00	8,430	1,686
	Aso 703	4,226	609,455	609,455	1.00	4,226	845
	Gwndi Qrga 329	2,354	229,791	229,791	1.00	2,354	471
	Mardin 327	5,944	1,134,155	737,200	1.00	5,944	1,189
	Gwndi Qrga 331	2,527	415,322	415,322	1.00	2,527	505
	Srwsht 333	7,017	1,028,788	1,028,788	1.00	7,017	1,403
						Total Flow	**10,258**
OJ2	Sana 313	8,650	835,642	208,910	0.25	2,163	433
	Kani Ba 318	6,861	662,669	298,201	0.45	3,087	617
	Gwlabakh 335	4,565	669,087	669,087	1.00	4,565	913
	Khastakhanai Shorsh 334	6,481	4,829,240	4,829,240	1.00	6,481	1,296
	Shary Spy						402
	Shary Pzishkan						624
						Total Flow	**4,286**

a; ONA = Optimized nominated area, b ;Pop. = Population, c; F= Fraction of Area of Flow/Total area, d; Q_{av} = Average Daily Flow

Table (A.9): Available Flow (Q_{av}) of Residential Complexes in the Study Area, (Researcher)

No.	City Name	No. of Buildings	No. of Flats	No. of Capita	Flow m^3/d
1	Barzaiakani Slemani(houses)	2,000	-	11,000	2,200
2	Rozh City	84	1,008	5,040	807
3	Baharan	21	114	570	91.2
4	Pak City	15	840	4,200	672
5	Nawroz City	4	192	960	153.6
6	Dream Land	8	408	2,040	326.4
7	Darwaza City 1 (Houses)	300	-	1,650	330
8	Darwaza City 2	23	1,081	5,405	865
9	Darwaza City 3	8	640	3,200	512
10	Gardin City	18	828	4,140	662.4
11	Chwar Chrai new (houses)	500	-	2,750	550
12	Gundi Allmany 1(House)	480	-	2,640	528
13	Gundi Allmany 2	-	424	2,120	422
14	Gundi Allmany 3	-	1,202	6,010	339
15	Shary Daik	50	456	2,280	962
16	Goizha City 1	9	432	2,160	345.6
	Goizha City 2	12	576	2,880	460.8
	Goizha City 3	11	792	3960	633.6
17	Diya City	13	364	1820	291.2
	Diya City - Houses	480	-	2640	331.1
18	Kurd City 1		301	1,655	331.1
	Kurd City 2		960	4,800	960
19	Lubnan City (houses)	624	-	3,120	624
20	Saib City	25	7	1,480	236.8
21	Dilan City 1	25	700	3,500	560
	Dilan City 2	55	672	3,360	537.6
22	Danya City	6	720	3,600	704
		2	160	800	
23	Sardam City(Houses)	280	-	1,540	308
24	Gulli Shar	52	624	3,120	499.2
25	Green City (houses)	500	-	2,750	550
26	Nawzad City (houses)	3,500	-	19,250	3,850
27	Shary Spy	19	228	1,140	182
28	Shary Spy (Houses)	200	-	1,100	220
29	Shary Pzishkan	15	780	3,900	624
30	Shari Roshinbiran	24	338	1,690	270.4
31	Jaff Towers (2 Towers)	2,000	272	1,360	217.6

Table (A. 10): Water Demand of Irrigation (Q_d) of the Green Areas (GRs), (Researcher)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR1	3,563	35.6	GR41	1,495	14.9
GR2	993	9.9	GR42	250	2.5
GR3	11,667	116.7	GR43	403	4.0
GR4	4,313	43.1	GR44	657	6.6
GR5	10,164	101.6	GR45	877	8.8
GR6	788	7.9	GR46	783	7.8
GR7	7,745	77.4	GR47	528	5.3
GR8	629	6.3	GR48	130	1.3
GR9	1,793	17.9	GR49	90	0.9
GR10	1,131	11.3	GR50	99	1.0
GR11	839	8.4	GR51	2,314	23.1
GR12	446	4.5	GR52	2,016	20.2
GR13	1,348	13.5	GR53	359	3.6
GR14	4,274	42.7	GR54	773	7.7
GR15	625	6.2	GR55	495	4.9
GR16	3,863	38.6	GR56	1,199	12.0
GR17	4,150	41.5	GR57	975	9.7
GR18	3,028	30.3	GR58	840	8.4
GR19	4,043	40.4	GR59	132	1.3
GR20	1,295	13.0	GR60	3,509	35.1
GR21	417	4.2	GR61	3,777	37.8
GR22	1,574	15.7	GR62	602	6.0
GR23	5,199	52.0	GR63	18,402	184.0
GR24	3,961	39.6	GR64	547	5.5
GR25	14,918	149.2	GR65	1,057	10.6
GR26	1,084	10.8	GR66	3,774	37.7
GR27	807	8.1	GR67	978	9.8
GR28	6,453	64.5	GR68	4,540	45.4
GR29	1,466	14.7	GR69	607	6.1
GR30	2,446	24.5	GR70	12,688	126.9
GR31	5,427	54.3	GR71	1,659	16.6
GR32	718	7.2	GR72	695	6.9
GR33	287	2.9	GR73	1,203	12.0
GR34	178	1.8	GR74	716	7.2
GR35	13,598	136.0	GR75	1,762	17.6
GR36	1,191	11.9	GR76	1,635	16.4
GR37	795	8.0	GR77	585	5.8
GR38	3,262	32.6	GR78	3,668	36.7
GR39	502	5.0	GA79	379	3.8
GR40	581	5.8	GA80	1,521	15.2

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR81	349	3.5	GR121	10,058	100.6
GR82	230	2.3	GR122	2,856	28.6
GR83	405	4.1	GR123	1,083	10.8
GR84	299	3.0	GR124	3,095	30.9
GR85	798	8.0	GR125	786	7.9
GR86	2,812	28.1	GR126	651	6.5
GR87	613	6.1	GR127	1,522	15.2
GR88	51,767	517.7	GR128	758	7.6
GR89	4,852	48.5	GR129	2,933	29.3
GR90	2,659	26.6	GR130	419	4.2
GR91	940	9.4	GR131	9,045	90.5
GR92	1,052	10.5	GR132	741	7.4
GR93	1,434	14.3	GR133	5,014	50.1
GR94	746	7.5	GR134	1,514	15.1
GR95	3,318	33.2	GR135	1,408	14.1
GR96	1,464	14.6	GR136	304	3.0
GR97	88	0.9	GR137	8,861	88.6
GR98	208	2.1	GR138	321	3.2
GR99	158	1.6	GR139	961	9.6
GR100	371	3.7	GR140	2,903	29.0
GR101	169	1.7	GR141	448	4.5
GR102	1,437	14.4	GR142	2,939	29.4
GR103	2,382	23.8	GR143	1,543	15.4
GR104	416	4.2	GR144	3,721	37.2
GR105	2,996	30.0	GR145	1,325	13.3
GR106	1,360	13.6	GR146	1,116	11.2
GR107	2,338	23.4	GR147	73	0.7
GR108	8,431	84.3	GR148	13,034	130.3
GR109	1,659	16.6	GR149	2,438	24.4
GR110	1,094	10.9	GR150	8,416	84.2
GR111	626	6.3	GR151	2,344	23.4
GR112	126	1.3	GR152	1,919	19.2
GR113	1,151	11.5	GR153	917	9.2
GR114	345	3.5	GR154	1,557	15.6
GR115	524	5.2	GR155	433	4.3
GR116	5,654	56.5	GR156	9,559	95.6
GR117	40	0.4	GR157	5,398	54.0
GR118	133	1.3	GR158	10,782	107.8
GR119	3,888	38.9	GR159	4,422	44.2
GR120	930	9.3	GR160	878	8.8

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR161	111	1.1	GR201	67	0.7
GR162	1,810	18.1	GR202	3,278	32.8
GR163	802	8.0	GR203	38,464	384.6
GR164	930	9.3	GR204	19,027	190.3
GR165	830	8.3	GR205	2,911	29.1
GR166	298	3.0	GR206	531	5.3
GR167	287	2.9	GR207	2,216	22.2
GR168	627	6.3	GR208	10,642	106.4
GR169	664	6.6	GR209	59	0.6
GR170	1,461	14.6	GR210	5,453	54.5
GR171	412	4.1	GR211	1,771	17.7
GR172	2,123	21.2	GR212	189	1.9
GR173	88	0.9	GR213	573	5.7
GR174	4,146	41.5	GR214	3,886	38.9
GR175	449	4.5	GR215	1,670	16.7
GR176	4,490	44.9	GR216	1,294	12.9
GR177	208	2.1	GR217	661	6.6
GR178	146	1.5	GR218	7,023	70.2
GR179	2,676	26.8	GR219	649	6.5
GR180	1,097	11.0	GR220	1,735	17.3
GR181	761	7.6	GR221	474	4.7
GR182	403	4.0	GR222	3,738	37.4
GR183	345	3.4	GR223	1,770	17.7
GR184	1,850	18.5	GR224	1,021	10.2
GR185	4,581	45.8	GR225	5,258	52.6
GR186	1,586	15.9	GR226	40,939	409.4
GR187	470	4.7	GR227	2,385	23.9
GR188	1,787	17.9	GR228	6,210	62.1
GR189	1,664	16.6	GR229	1,894	18.9
GR190	815	8.1	GR230	3,766	37.7
GR191	1,109	11.1	GR231	4,206	42.1
GR192	1,420	14.2	GR232	1,235	12.4
GR193	1,759	17.6	GR233	2,026	20.3
GR194	188	1.9	GR234	3,491	34.9
GR195	156	1.6	GR235	754	7.5
GR196	335	3.4	GR236	310	3.1
GR197	181	1.8	GR237	1,628	16.3
GR198	48	0.5	GR238	3,291	32.9
GR199	154	1.5	GR239	2,012	20.1
GR200	1,811	18.1	GR240	1,910	19.1

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR241	1,711	17.1	GR281	4,066	40.7
GR242	767	7.7	GR282	7,059	70.6
GR243	916	9.2	GR283	8,840	88.4
GR244	2,492	24.9	GR284	3,837	38.4
GR245	2,320	23.2	GR285	2,322	23.2
GR246	939	9.4	GR286	3,498	35.0
GR247	620	6.2	GR287	6,540	65.4
GR248	663	6.6	GR288	1,812	18.1
GR249	1,958	19.6	GR289	453	4.5
GR250	2,041	20.4	GR290	34,214	342.1
GR251	512	5.1	GR291	2,266	22.7
GR252	9,121	91.2	GR292	3,380	33.8
GR253	2,041	20.4	GR293	4,467	44.7
GR254	4,432	44.3	GR294	8,607	86.1
GR255	915	9.1	GR295	1,410	14.1
GR256	34,742	347.4	GR296	19,135	191.4
GR257	5,217	52.2	GR297	410	4.1
GR258	16,569	165.7	GR298	4,441	44.4
GR259	10,233	102.3	GR299	11,410	114.1
GR260	375	3.7	GR300	364	3.6
GR261	2,978	29.8	GR301	952	9.5
GR262	10,072	100.7	GR302	733	7.3
GR263	1,041	10.4	GR303	735	7.3
GR264	1,111	11.1	GR304	2,458	24.6
GR265	4,575	45.8	GR305	1,864	18.6
GR266	28,335	283.4	GR306	801	8.0
GR267	496	5.0	GR307	7,416	74.2
GR268	2,878	28.8	GR308	1,691	16.9
GR269	763	7.6	GR309	2,554	25.5
GR270	3,260	32.6	GR310	2,440	24.4
GR271	3,111	31.1	GR311	6,797	68.0
GR272	92	0.9	GR312	4,415	44.1
GR273	1,718	17.2	GR313	266	2.7
GR274	264	2.6	GR314	4,600	46.0
GR275	406	4.1	GR315	9,054	90.5
GR276	6,955	69.6	GR316	2,028	20.3
GR277	593	5.9	GR317	2,030	20.3
GR278	566	5.7	GR318	672	6.7
GR279	1,491	14.9	GR319	577	5.8
GR280	3,310	33.1	GR320	105	1.0

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR321	86	0.9	GR361	67	0.7
GR322	85	0.8	GR362	80	0.8
GR323	297	3.0	GR363	979	9.8
GR324	176	1.8	GR364	3,676	36.8
GR325	285	2.9	GR365	935	9.3
GR326	192	1.9	GR366	596	6.0
GR327	60	0.6	GR367	540	5.4
GR328	512	5.1	GR368	2,187	21.9
GR329	399	4.0	GR369	2,324	23.2
GR330	156	1.6	GR370	3,234	32.3
GR331	62	0.6	GR371	8,843	88.4
GR332	320	3.2	GR372	9,237	92.4
GR333	332	3.3	GR373	1,667	16.7
GR334	1,466	14.7	GR374	1,512	15.1
GR335	3,654	36.5	GR375	28,382	283.8
GR336	143	1.4	GR376	1,366	13.7
GR337	3,333	33.3	GR377	3,105	31.0
GR338	495	4.9	GR378	1,519	15.2
GR339	526	5.3	GR379	1,168	11.7
GR340	2,382	23.8	GR380	2,474	24.7
GR341	1,064	10.6	GR381	2,558	25.6
GR342	395	3.9	GR382	1,866	18.7
GR343	17,376	173.8	GR383	8,998	90.0
GR344	8,635	86.4	GR384	1,335	13.4
GR345	5,901	59.0	GR385	599	6.0
GR346	31,331	313.3	GR386	1,394	13.9
GR347	831	8.3	GR387	642	6.4
GR348	10,756	107.6	GR388	4,251	42.5
GR349	3,812	38.1	GR389	2,098	21.0
GR350	21,423	214.2	GR390	1,565	15.6
GR351	14,042	140.4	GR391	8,369	83.7
GR352	7,690	76.9	GR392	1,493	14.9
GR353	847	8.5	GR393	6,066	60.7
GR354	6,399	64.0	GR394	228	2.3
GR355	426	4.3	GR395	2,581	25.8
GR356	6	0.1	GR396	798	8.0
GR357	12	0.1	GR397	4,929	49.3
GR358	19	0.2	GR398	361	3.6
GR359	35	0.3	GR399	2,679	26.8
GR360	243	2.4	GR400	171	1.7

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR401	1,547	15.5	GR441	1,050	10.5
GR402	450	4.5	GR442	3,602	36.0
GR403	1,000	10.0	GR443	1,470	14.7
GR404	6,133	61.3	GR444	913	9.1
GR405	715	7.1	GR445	4,537	45.4
GR406	2,003	20.0	GR446	5,837	58.4
GR407	2,193	21.9	GR447	3,345	33.4
GR408	3,123	31.2	GR448	1,581	15.8
GR409	1,340	13.4	GR449	2,630	26.3
GR410	1,243	12.4	GR450	1,363	13.6
GR411	7,170	71.7	GR451	5,039	50.4
GR412	5,251	52.5	GR452	1,063	10.6
GR413	13,073	130.7	GR453	3,161	31.6
GR414	3,015	30.1	GR454	2,273	22.7
GR415	2,787	27.9	GR455	5,394	53.9
GR416	2,210	22.1	GR456	2,988	29.9
GR417	6,685	66.8	GR457	4,896	49.0
GR418	2,374	23.7	GR458	320	3.2
GR419	8,738	87.4	GR459	416	4.2
GR420	5,358	53.6	GR460	286	2.9
GR421	39,859	398.6	GR461	355	3.5
GR422	26,262	262.6	GR462	2,310	23.1
GR423	8,780	87.8	GR463	3,432	34.3
GR424	16,458	164.6	GR464	2,384	23.8
GR425	17.3	0.173	GR465	3,468	34.7
GR426	56.22	0.562	GR466	2,829	28.3
GR427	5.4	0.054	GR467	441	4.4
GR428	15.7	0.157	GR468	15,329	153.3
GR429	32,667	326.7	GR469	233	2.3
GR430	36,840	368.4	GR470	2,545	25.5
GR431	61,246	612.5	GR471	4,202	42.0
GR432	18.9	0.189	GR472	1,732	17.3
GR433	29.02	0.29	GR473	211	2.1
GR434	20,841	208.4	GR474	3,658	36.6
GR435	16,681	166.8	GR475	1,047	10.5
GR436	11,323	113.2	GR476	3,522	35.2
GR437	9,419	94.2	GR477	378	3.8
GR438	2,142	21.4	GR478	206	2.1
GR439	2,983	29.8	GR479	466	4.7
GR440	2,543	25.4	GR480	1,143	11.4

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR481	1,401	14.0	GR521	1,247	12.5
GR482	607	6.1	GR522	1,600	16.0
GR483	7,910	79.1	GR523	1,704	17.0
GR484	1,109	11.1	GR524	1,423	14.2
GR485	787	7.9	GR525	558	5.6
GR486	3,135	31.3	GR526	8,718	87.2
GR487	1,025	10.3	GR527	6,524	65.2
GR488	1,004	10.0	GR528	2,224	22.2
GR489	96	1.0	GR529	18,820	188.2
GR490	147	1.5	GR530	3,236	32.4
GR491	291,482	1,457.4	GR531	7,329	73.3
GR492	1,994	19.9	GR532	4,751	47.5
GR493	48,017	480.2	GR533	1,680	16.8
GR494	2,987	29.9	GR534	3,309	33.1
GR495	301	3.0	GR535	1,450	14.5
GR496	991	9.9	GR536	6,036	60.4
GR497	979	9.8	GR537	7,820	78.2
GR498	408	4.1	GR538	1,960	19.6
GR499	439	4.4	GR539	2,870	28.7
GR500	2,250	22.5	GR540	1,268	12.7
GR501	1,035	10.3	GR541	788	7.9
GR502	1,489	14.9	GR542	893	8.9
GR503	1,758	17.6	GR543	1,681	16.8
GR504	2,111	21.1	GR544	3,053	30.5
GR505	6,338	63.4	GR545	1,364	13.6
GR506	1,645	16.4	GR546	3,128	31.3
GR507	877	8.8	GR547	3,555	35.5
GR508	1,896	19.0	GR548	10,521	105.2
GR509	803	8.0	GR549	17,966	179.7
GR510	423	4.2	GR550	4,013	40.1
GR511	1,743	17.4	GR551	1,622	16.2
GR512	217	2.2	GR552	427	4.3
GR513	491	4.9	GR553	521	5.2
GR514	169	1.7	GR554	1,760	17.6
GR515	1,662	16.6	GR555	18,084	180.8
GR516	1,730	17.3	GR556	32.11	0.321
GR517	168	1.7	GR557	20,460	204.6
GR518	1,902	19.0	GR558	11,343	113.4
GR519	5,876	58.8	GR559	19,690	196.9
GR520	4,357	43.6	GR560	16,430	164.3

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR561	385	3.9	GR601	15,272	152.7
GR562	888	8.9	GR602	12,697	127.0
GR563	1,125	11.2	GR603	2,864	28.6
GR564	9,707	97.1	GR604	3,373	33.7
GR565	1,897	19.0	GR605	8,129	81.3
GR566	607	6.1	GR606	1,106	11.1
GR567	1,770	17.7	GR607	592	5.9
GR568	660	6.6	GR608	9,106	91.1
GR569	15,641	156.4	GR609	948	9.5
GR570	594	5.9	GR610	2,637	26.4
GR571	937	9.4	GR611	9,773	97.7
GR572	2,097	21.0	GR612	2,131	21.3
GR573	135	1.4	GR613	881	8.8
GR574	1,351	13.5	GR614	1,755	17.5
GR575	1,125	11.2	GR615	4,918	49.2
GR576	865	8.6	GR616	724	7.2
GR577	834	8.3	GR617	256	2.6
GR578	115	1.2	GR618	213	2.1
GR579	2,708	27.1	GR619	489	4.9
GR580	822	8.2	GR620	476	4.8
GR581	7,221	72.2	GR621	2,616	26.2
GR582	1,972	19.7	GR622	1,066	10.7
GR583	532	5.3	GR623	1,867	18.7
GR584	1,545	15.5	GR624	2,018	20.2
GR585	3,302	33.0	GR625	7,166	71.7
GR586	308	3.1	GR626	4,625	46.2
GR587	472	4.7	GR627	2,835	28.4
GR588	4,585	45.8	GR628	1,455	14.5
GR589	983	9.8	GR629	12,298	123.0
GR590	1,411	14.1	GR630	7,088	70.9
GR591	2,126	21.3	GR631	23,355	233.6
GR592	626	6.3	GR632	1,489	14.9
GR593	4,869	48.7	GR633	5,762	57.6
GR594	60,800	608.0	GR634	652	6.5
GR595	9,742	97.4	GR635	24.6	0.246
GR596	19,285	192.9	GR636	3,999	40.0
GR597	1,559	15.6	GR637	1,368	13.7
GR598	5,394	53.9	GR638	2,494	24.9
GR599	4,346	43.5	GR639	11,415	114.1
GR600	141,713	708.6	GR640	976	9.8

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR641	332	3.3	GR681	3,592	35.9
GR642	9,853	98.5	GR682	1,970	19.7
GR643	5,063	50.6	GR683	1,020	10.2
GR644	2,285	22.9	GR684	765	7.7
GR645	8,340	83.4	GR685	15,651	156.5
GR646	273	2.7	GR686	8,109	81.1
GR647	1,018	10.2	GR687	1,521	15.2
GR648	25,895	259.0	GR688	1,284	12.8
GR649	447	4.5	GR689	4,491	44.9
GR650	2,114	21.1	GR690	7,328	73.3
GR651	186	1.9	GR691	13,388	133.9
GR652	4,144	41.4	GR692	1,283	12.8
GR653	1,231	12.3	GR693	5,548	55.5
GR654	7,951	79.5	GR694	2,314	23.1
GR655	362	3.6	GR695	1,335	13.4
GR656	2,637	26.4	GR696	4,878	48.8
GR657	501	5.0	GR697	581	5.8
GR658	358	3.6	GR698	2,133	21.3
GR659	1,697	17.0	GR699	1,409	14.1
GR660	1,206	12.1	GR700	1,216	12.2
GR661	14.35	0.144	GR701	531	5.3
GR662	507	5.1	GR702	1,720	17.2
GR663	10,247	102.5	GR703	5,935	59.3
GR664	3,402	34.0	GR704	3,527	35.3
GR665	2,262	22.6	GR705	2,838	28.4
GR666	1,755	17.6	GR706	1,294	12.9
GR667	3,500	35.0	GR707	1,111	11.1
GR668	5,250	52.5	GR708	1,926	19.3
GR669	5,012	50.1	GR709	9,209	92.1
GR670	29,171	291.7	GR710	127,155	1,271.6
GR671	19,529	195.3	GR711	10,511	105.1
GR672	12,186	121.9	GR712	33,338	333.4
GR673	1,085	10.8	GR713	483,925	1,451.8
GR674	4,089	40.9	GR714	5,389	53.9
GR675	695	7.0	GR715	2,364	23.6
GR676	9,141	91.4	GR716	3,325	33.3
GR677	8,056	80.6	GR717	23,013	230.1
GR678	5,462	54.6	GR718	3,341	33.4
GR679	1,216	12.2	GR719	12,366	123.7
GR680	1,831	18.3	GR720	4,017	40.2

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR721	20,207	202.1	GR761	1,955	19.6
GR722	15,256	152.6	GR762	3,719	37.2
GR723	19,803	198.0	GR763	2,521	25.2
GR724	103,770	1,037.7	GR764	672	6.7
GR725	11,427	114.3	GR765	2,321	23.2
GR726	11,332	113.3	GR766	8,867	88.7
GR727	23,069	230.7	GR767	11,115	111.2
GR728	23,288	232.9	GR768	4,857	48.6
GR729	32,692	326.9	GR769	3,290	32.9
GR730	10,504	105.0	GR770	10,003	100.0
GR731	23,986	239.9	GR771	3	0.0
GR732	817,060	1,634.1	GR772	4	0.0
GR733	1,046,606	1,569.9	GR773	44,308	443.1
GR734	4,061	40.6	GR774	3	0.0
GR735	1,367	13.7	GR775	10,025	100.3
GR736	2,599	26.0	GR776	9,184	91.8
GR737	2,039	20.4	GR777	685	6.8
GR738	3,215	32.1	GR778	4,533	45.3
GR739	1,579	15.8	GR779	6,777	67.8
GR740	2,376	23.8	GR780	3,881	38.8
GR741	6,884	68.8	GR781	3,603	36.0
GR742	29,707	297.1	GR782	1,192	11.9
GR743	7,385	73.8	GR783	4,680	46.8
GR744	1,183	11.8	GR784	2,467	24.7
GR745	1,196	12.0	GR785	1,879	18.8
GR746	463	4.6	GR786	3,273	32.7
GR747	1,503	15.0	GR787	8,188	81.9
GR748	703	7.0	GR788	2,649	26.5
GR749	968	9.7	GR789	3,567	35.7
GR750	2,813	28.1	GR790	8,221	82.2
GR751	1,116	11.2	GR791	14,358	143.6
GR752	1,078	10.8	GR792	1,323	13.2
GR753	1,197	12.0	GR793	8,468	84.7
GR754	6,212	62.1	GR794	1,607	16.1
GR755	895	9.0	GR795	2,895	29.0
GR756	560	5.6	GR796	1,136	11.4
GR757	694	6.9	GR797	2,181	21.8
GR758	5,221	52.2	GR798	2,249	22.5
GR759	4,554	45.5	GR799	2,400	24.0
GR760	10,436	104.4	GR800	2,935	29.4

a:GR = Green Areas;

Table (A. 10)

GR[a]	Area, m^2	Q_d, m^3/d	GR[a]	Area, m^2	Q_d, m^3/d
GR801	2,541	25.4	GR815	7,361	73.6
GR802	706	7.1	GR816	1,831	18.3
GR803	1,284	12.8	GR817	2,088	20.9
GR804	1,411	14.1	GR818	8,238	82.4
GR805	883	8.8	GR819	625	6.2
GR806	1,273	12.7	GR820	1,746	17.5
GR807	361	3.6	GR821	510	5.1
GR808	246	2.5	GR822	4,178	41.8
GR809	995	9.9	GR823	2,618	26.2
GR810	156	1.6	GR824	13,102	131.0
GR811	1,707	17.1	GR825	791	7.9
GR812	1,634	16.3	GR826	93,851	469.3
GR813	1,073	10.7	GR827	26,505	132.5
GR814	2,101	21.0			

a:GR = Green Areas;

Pipe Cost Calculation Detail

The price list are taken from the market of 2014 as shown in table (A.11)

Table (A. 11): Price list of PE -100, SDR11. PN16, (Local Market)

Pipe Diameter , mm	Unit Price US$/m	Pipe Diameter , mm	Unit Price US$/m
20	0.55	200	7.90
25	0.75	225	9.50
32	0.85	250	10.50
40	1.25	280	11.90
50	1.45	315	13.70
63	1.75	355	16.45
75	2.25	400	19.20
90	2.35	450	22.75
110	4.40	500	26.90
125	4.90	560	32.50
140	5.40	600	39.80
160	5.90	700	49.20
180	6.90		

To find the parameters m and K_m of Equation (4.15) the best fit equation is found using data of table (A.11) as shown below :

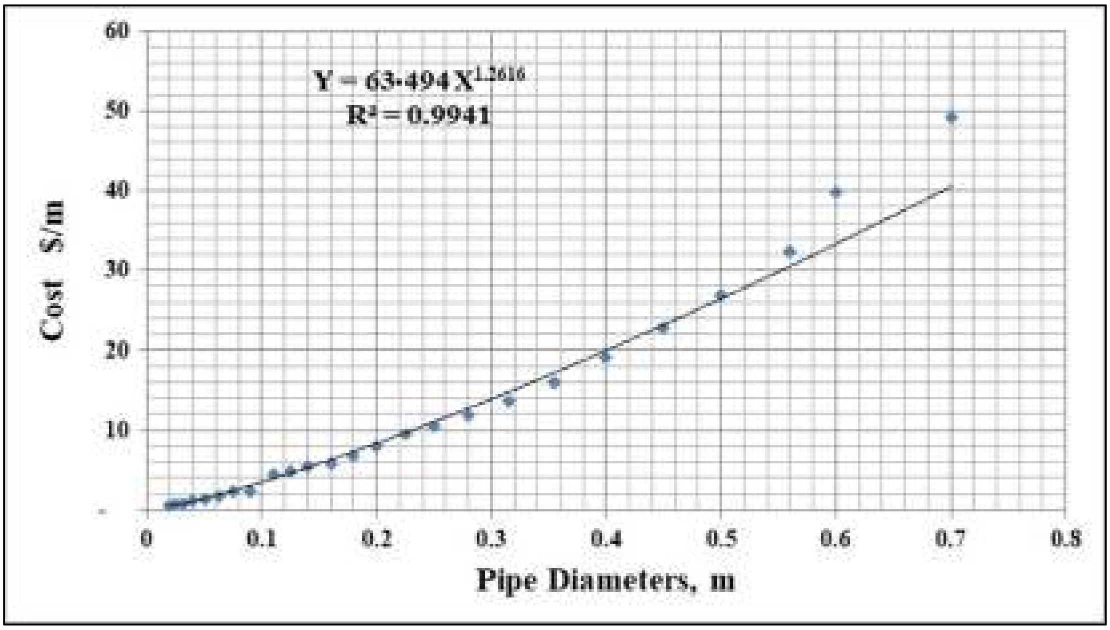

Fig. (A.1): The Best Fit Equation of the Pipe Cost Equation, (Researcher)

Table (A. 12) : Results of the GIS – OD Network Matrix Analysis, , (Researcher)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OA1	GR 411	501	OA1	GR 416	615
OA1	GR 568	122	OA1	GR 216	747
OA1	GR 228	187	OA1	GR 531	807
OA1	GR 214	320	OB1	GR 657	545
OA1	GR 693	391	OB1	GR 223	545
OA1	GR 694	316	OB1	GR 391	694
OA1	GR 717	393	OB1	GR 265	706
OA1	GR 695	360	OB1	GR 252	724
OA1	GR 541	409	OB1	GR 297	766
OA1	GR 696	504	OB1	GR 696	781
OA1	GR 692	526	OB1	GR 566	820
OA1	GR 758	705	OB1	GR 536	824
OA1	GR 252	752	OB1	GR 375	862
OA1	GR 537	806	OB1	GR 226	976
OA1	GR 416	998	OB1	GR 225	976

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OB1	GR 659	990	OC1	GR 690	500
OB2	GR 250	146	OC1	GR 543	462
OB2	GR 251	197	OC1	GR 398	476
OB2	GR 248	355	OC1	GR 403	561
OB2	GR 221	649	OC1	GR 777	597
OB2	GR 766	701	OC1	GR 409	601
OB2	GR 633	889	OC1	GR 39	632
OB2	GR 634	994	OC1	GR 40	659
OB2	GR 35	605	OC1	GR703	684
OB2	GR312	686	OC1	GR171	698
OB2	GR705	576	OC1	GR691	703
OB3	GR345	50	OC1	GR375	721
OB3	GR143	154	OC1	GR226	800
OB3	GR732	382	OC1	GR225	800
OB3	GR 478	461	OC1	GR794	726
OB3	GR 344	522	OC1	GR352	758
OB3	GR 754	860	OC1	GR448	774
OB3	GR 289	965	OC1	GR150	786
OB4	GR 733	148	OC1	GR719	786
OB4	GR 586	349	OC1	GR589	810
OB4	GR 592	443	OC1	GR236	845
OB4	GR 593	604	OC1	GR590	887
OB4	GR 346	606	OC1	GR702	892
OB4	GR 249	812	OC1	GR566	913
OB4	GR 63	957	OC1	GR395	929
OC1	GR812	132	OC2	GR 7	612
OC1	GR311	179	OC2	GR763	648
OC1	GR 614	209	OC2	GR309	831
OC1	GR 629	420	OC2	GR 70	863
OC1	GR 253	254	OC2	GR 88	77
OC1	GR 615	254	OC2	GR137	197
OC1	GR 392	286	OC2	GR149	272
OC1	GR 222	412	OC2	GR483	388
OC1	GR 728	394	OC2	GR484	377
OC1	GR 217	351	OC2	GR437	143
OC1	GR 84	361	OC2	GR148	420
OC1	GR616	402	OC2	GR765	140
OC1	GR351	452	OC2	GR764	99

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OC2	GR 37	698	OC4	GR 675	765
OC2	GR 28	472	OC4	GR 676	765
OC2	GR 85	276	OC4	GR 720	810
OC2	GR370	461	OC4	GR 738	810
OC3	GR763	12	OC4	GR 601	821
OC3	GR309	170	OC4	GR 372	827
OC3	GR 7	370	OC4	GR 156	832
OC3	GR85	276	OC4	GR 36	903
OC3	GR370	444	OC4	GR291	971
OC3	GR 70	546	OC4	GR 25	997
OC3	GR765	521	OD1	GR372	318
OC3	GR764	562	OD1	GR586	384
OC3	GR739	620	OD1	GR592	478
OC3	GR 28	635	OD1	GR738	492
OC3	GR 5	691	OD1	GR593	638
OC3	GR37	698	OD1	GR346	640
OC3	GR91	737	OD1	GR675	655
OC3	GR762	760	OD1	GR 63	668
OC3	GR 4	762	OD1	GR371	706
OC3	GR92	792	OD1	GR720	751
OC3	GR484	811	OD1	GR676	763
OC3	GR438	867	OD1	GR647	764
OC3	GR451	981	OD1	GR733	787
OC3	GR439	875	OD1	GR249	847
OC3	GR483	890	OD1	GR290	917
OC3	GR149	900	OD1	GR291	912
OC3	GR501	967	OD1	GR157	913
OC3	GR 3	973	OD1	GR480	980
OC3	GR 368	986	OE1	GR722	104
OC4	GR 157	59	OE1	GR473	132
OC4	GR 158	498	OE1	GR539	221
OC4	GR 642	423	OE1	GR472	242
OC4	GR 647	429	OE1	GR726	248
OC4	GR 480	556	OE1	GR605	399
OC4	GR 648	674	OE1	GR606	328
OC4	GR 290	705	OE1	GR531	349
OC4	GR 481	694	OE1	GR609	449
OC4	GR 63	727	OE1	GR727	397
OC4	GR371	765	OE1	GR613	405

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OE1	GR 800	426	OE2	GR 71	781
OE1	GR 379	441	OE2	GR430	991
OE1	GR 611	464	OE2	GR673	873
OE1	GR 364	469	OE2	GR242	907
OE1	GR 383	522	OE2	GR212	907
OE1	GR 540	528	OE2	GR151	935
OE1	GR 610	578	OE2	GR1	942
OE1	GR 378	626	OE2	GR 255	948
OE1	GR 380	683	OE2	GR 52	952
OE1	GR 26	688	OE2	GR 761	955
OE1	GR220	697	OE2	GR 580	959
OE1	GR 381	761	OE2	GR 59	983
OE1	GR 803	785	OE3	GR307	92
OE1	GR 608	807	OE3	GR656	348
OE1	GR 455	838	OE3	GR343	641
OE1	GR 382	872	OE3	GR 9	464
OE1	GR 724	963	OE3	GR153	487
OE1	GR 725	950	OE3	GR444	505
OE1	GR 376	943	OE3	GR 8	525
OE1	GR 377	964	OE3	GR21	561
OE1	GR 292	880	OE3	GR430	987
OE1	GR 293	880	OE3	GR387	640
OE1	GR 723	952	OE3	GR 58	655
OE1	GR 607	971	OE3	GR373	664
OE1	GR 215	997	OE3	GR152	686
OE2	GR 187	19	OE3	GR749	725
OE2	GR 283	96	OE3	GR655	753
OE2	GR 452	45	OE3	GR 77	778
OE2	GR 453	114	OE3	GR300	785
OE2	GR 62	116	OE3	GR748	804
OE2	GR301	180	OE3	GR410	818
OE2	GR303	213	OE3	GR811	825
OE2	GR188	237	OE3	GR456	904
OE2	GR491	338	OE3	GR267	910
OE2	GR 54	484	OE3	GR306	919
OE2	GR343	505	OE3	GR195	920
OE2	GR492	569	OE3	GR151	925
OE2	GR767	695	OE3	GR 53	927
OE2	GR475	702	OE3	GR 78	931

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OE3	GR 71	938	OE5	GR 770	647
OE3	GR 52	943	OE5	GR 649	764
OE3	GR299	948	OF1	GR 178	367
OE3	GR308	949	OF1	GR 299	537
OE3	GR713	63	OF1	GR 34	495
OE3	GR580	984	OF1	GR267	499
OE4	GR579	286	OF1	GR306	546
OE4	GR475	322	OF1	GR670	553
OE4	GR492	455	OF1	GR300	624
OE4	GR767	520	OF1	GR159	673
OE4	GR476	535	OF1	GR654	747
OE4	GR673	599	OF1	GR713	747
OE4	GR431	655	OF1	GR710	765
OE4	GR 97	710	OF1	GR 58	876
OE4	GR429	729	OF1	GR749	946
OE4	GR477	750	OF1	GR655	975
OE4	GR 1	767	OF2	GR670	391
OE4	GR577	775	OF2	GR159	493
OE4	GR580	784	OF2	GR 34	556
OE4	GR430	802	OF2	GR 19	687
OE4	GR 51	810	OF2	GR 98	658
OE4	GR 56	837	OF2	GR178	661
OE4	GR 53	841	OF2	GR496	727
OE4	GR 2	842	OF2	GR497	729
OE4	GR347	884	OF2	GR525	784
OE4	GR453	910	OF2	GR160	809
OE4	GR189	926	OF2	GR199	849
OE4	GR748	963	OF2	GR 33	873
OE4	GR462	980	OF2	GR138	883
OE5	GR205	417	OF2	GR710	889
OE5	GR413	426	OF3	GR203	201
OE5	GR781	443	OF3	GR639	249
OE5	GR632	449	OF3	GR294	297
OE5	GR774	467	OF3	GR103	410
OE5	GR773	528	OF3	GR515	541
OE5	GR775	528	OF3	GR502	554
OE5	GR776	528	OF3	GR652	578
OE5	GR637	541	OF3	GR 31	586
OE5	GR630	576	OF3	GR523	587

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To RGA [b]	Length , m
OF3	GR 454	606	OF4	GR 632	715
OF3	GR 519	616	OF4	GR 781	720
OF3	GR 102	659	OF4	GR 423	727
OF3	GR 206	702	OF4	GR 413	737
OF3	GR 423	708	OF4	GR 721	871
OF3	GR 384	741	OF4	GR 304	941
OF3	GR 304	761	OF4	GR 384	961
OF3	GR 500	777	OG1	GR 276	200
OF3	GR 721	791	OG1	GR 729	356
OF3	GR 353	798	OG1	GR 132	425
OF3	GR 394	811	OG1	GR 281	678
OF3	GR 107	950	OG1	GR 622	800
OF3	GR 355	964	OG1	GR 549	806
OF3	GR 576	812	OG1	GR 808	832
OF3	GR 524	819	OG1	GR 470	866
OF3	GR 575	838	OG1	GR 534	884
OF3	GR 517	913	OG1	GR 528	919
OF3	GR 495	915	OG1	GR 420	976
OF3	GR 354	920	OG1	GR 730	978
OF3	GR 200	921	OG1	GR 486	986
OF3	GR 561	925	OG1	GR 621	987
OF3	GR 196	959	OG1	GR 600	797
OF3	GR 516	990	OG2	GR 671	63
OF4	GR 652	284	OG2	GR 130	192
OF4	GR 649	399	OG2	GR 123	293
OF4	GR 636	412	OG2	GR 275	308
OF4	GR 650	459	OG2	GR 133	339
OF4	GR 542	507	OG2	GR 122	672
OF4	GR 529	512	OG2	GR 131	675
OF4	GR 770	516	OG2	GR 170	790
OF4	GR 782	571	OG2	GR 467	805
OF4	GR 630	588	OG2	GR 582	822
OF4	GR 801	594	OG2	GR 270	823
OF4	GR 639	613	OG3	GR 826	110
OF4	GR 637	622	OG3	GR 518	234
OF4	GR 773	636	OG3	GR 736	238
OF4	GR 774	696	OG3	GR 106	264
OF4	GR 651	698	OG3	GR 108	289
OF4	GR 638	712	OG3	GR 521	306

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OG3	GR 520	325	OH1	GR 686	268
OG3	GR 751	334	OH1	GR 412	413
OG3	GR 105	343	OH1	GR 678	503
OG3	GR 109	448	OH1	GR 600	538
OG3	GR 507	474	OH1	GR 348	544
OG3	GR 499	506	OH1	GR 687	712
OG3	GR 362	510	OH1	GR 712	743
OG3	GR 100	513	OH1	GR 798	769
OG3	GR 361	515	OH1	GR 563	811
OG3	GR 99	541	OH1	GR 599	862
OG3	GR498	542	OH1	GR 796	882
OG3	GR173	607	OH1	GR 820	899
OG3	GR497	723	OH1	GR 760	943
OG3	GR496	725	OH1	GR 715	961
OG3	GR355	743	OH1	GR 627	931
OG3	GR107	752	OH1	GR 574	950
OG3	GR 19	815	OH2	GR 821	224
OG3	GR664	838	OH2	GR 526	590
OG3	GR663	866	OH2	GR 597	602
OG3	GR354	852	OH2	GR 786	609
OG3	GR827	880	OH2	GR 598	737
OG3	GR516	969	OH2	GR 388	642
OG3	GR353	975	OH2	GR 665	708
OG4	GR109	486	OH2	GR 296	754
OG4	GR105	591	OH2	GR 683	767
OG4	GR108	645	OH2	GR 268	773
OG4	GR518	700	OH2	GR 287	789
OG4	GR827	880	OH2	GR 810	834
OG4	GR203	971	OH2	GR 532	848
OG4	GR519	717	OH2	GR 286	873
OG4	GR174	719	OH2	GR 760	967
OG4	GR257	740	OH2	GR 715	951
OG4	GR258	789	OH2	GR 627	980
OG4	GR256	887	OH2	GR 527	1000
OG4	GR 80	959	OH2	GR 417	1000
OG4	GR 30	726	OH2	GR 596	1000
OG4	GR664	940	OH3	GR 532	85
OG4	GR102	971	OH3	GR 743	352
OH1	GR679	225	OH3	GR 261	556

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GR [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OH3	GR 286	577	OI2	GR 827	433
OH3	GR 821	623	OI2	GR 202	478
OH3	GR 685	644	OI2	GR 258	583
OH3	GR 677	676	OI2	GR 663	656
OH3	GR 787	686	OI2	GR 257	693
OH3	GR 737	728	OI2	GR 507	758
OH3	GR 668	761	OI2	GR 30	853
OH3	GR 667	809	OI2	GR109	865
OH3	GR 349	914	OI2	GR751	884
OH3	GR 263	775	OI2	GR518	986
OH3	GR 665	841	OI2	GR108	957
OH3	GR 262	826	OI2	GR105	970
OH3	GR 269	857	OI3	GR 43	431
OH3	GR 526	860	OI3	GR506	613
OH3	GR 735	885	OI3	GR709	567
OH3	GR 407	894	OI3	GR262	661
OI1	GR 66	2	OI3	GR571	668
OI1	GR177	73	OI3	GR743	756
OI1	GR 65	132	OI3	GR263	711
OI1	GR175	158	OI3	GR741	732
OI1	GR 64	229	OI3	GR266	764
OI1	GR268	421	OI3	GR572	783
OI1	GR421	538	OI3	GR711	839
OI1	GR407	854	OI3	GR202	884
OI1	GR211	855	OI3	GR121	866
OI1	GR176	873	OI3	GR256	888
OI1	GR269	891	OI3	GR569	905
OI1	GR287	779	OI3	GR 80	918
OI1	GR508	920	OI3	GR573	953
OI1	GR662	970	OJ1	GR778	451
OI1	GR535	991	OJ1	GR625	525
OI2	GR174	159	OJ1	GR624	579
OI2	GR711	279	OJ1	GR623	674
OI2	GR121	306	OJ1	GR626	873
OI2	GR266	809	OJ1	GR595	986
OI2	GR709	887	OJ1	GR779	812
OI2	GR256	329	OJ1	GR818	978
OI2	GR 80	359	OJ1	GR594	994
OI2	GR664	379	OJ1	GR792	997

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 12)

From ONA [a]	To GA [b]	Length , m	From ONA [a]	To GR [b]	Length , m
OJ2	GR 349	330	OJ2	GR 231	726
OJ2	GR 350	411	OJ2	GR 598	984
OJ2	GR 667	431	OJ2	GR 597	987
OJ2	GR 668	587	OJ2	GR 626	985
OJ2	GR 735	460	OJ2	GR 595	991
OJ2	GR 787	538	OJ2	GR 779	675
OJ2	GR 685	580			
OJ2	GR 548	606			

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 13) : Details of the Demand of the Grouping Green Areas, (Researcher)

Group No.	From ONA [a]	To GR [b] - Group	Length , m	Group Demand m^3/d
1	OA1	GR 411	501	72
2	OA1	GR 717	393	230
3	OA1	GR 693	391	55
4	OA1	GR 758	705	52
5	OA1	GR 252	752	91
6	OA1	GR 228	187	62
7	OA1	GR 537	807	78
1	OB1	GR 391	694	84
2	OB1	GR 536	824	61
3	OB1	GR 226	976	380
1	OB2	GR 35	686	136
2	OB2	GR 766	780	89
3	OB2	GR 633	889	58
4	OB2	GR 250	146	20

a;ONA =Optimized Nominated Areas, b; GR = Green Areas

Table (A. 13)

Group No.	From ONA [a]	To GR [b] - Group	Length , m	Group Demand m^3/d
1	OB3	GR 143	154	15
2	OB3	GR 732	382	1634
3	OB3	GR 344	522	86
1	OB4	GR733	148	1532
2	OB4	GR 63	957	38
3	OB4	GR346	606	177
1	OC1	GR 311	220	68
2	OC1	GR 629	420	123
3	OC1	GR 691	800	134
4	OC1	GR 222	412	37
5	OC1	GR 728	394	233
6	OC1	GR 719	786	124
7	OC1	GR 226	800	30
8	OC1	GR 150	786	84
8	OC1	GR 351	452	140
10	OC1	GR 703	684	59
1	OC2	GR 88	77	518
2	OC2	GR 28	472	65
3	OC2	GR370	461	12
1	OC3	GR370	444	20
2	OC3	GR 7	370	102
3	OC3	GR 5	691	102
4	OC3	GR 3	973	117
5	OC3	GR70	546	127
1	OC4	GR290	705	155
2	OC4	GR157	59	54
3	OC4	GR648	674	259
4	OC4	GR 25	997	149

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 13)

Group No.	From ONA [a]	To GR [b] - Group	Length , m	Group Demand m^3/d
5	OC4	GR 158	498	108
6	OC4	GR 63	763	67
1	OD1	GR733	787	31
2	OD1	GR290	917	188
3	OD1	GR346	640	137
4	OD1	GR 63	668	79
1	OE1	GR722	104	153
2	OE1	GR724	963	1038
3	OE1	GR605	449	81
4	OE1	GR608	807	91
5	OE1	GR383	522	90
6	OE1	GR611	500	98
1	OE2	GR343	505	23
2	OE2	GR491	338	1458
3	OE2	GR767	695	88
4	OE2	GR283	96	88
1	OE3	GR343	641	152
2	OE3	GR430	987	325
3	OE3	GR713	63	1446
4	OE3	GR456	904	30
5	OE3	GR307	92	74
6	OE3	GR299	948	60
1	OE4	GR 431	655	613
2	OE4	GR 429	729	327
3	OE4	GR 430	802	36
4	OE4	GR 579	286	27
5	OE4	GR 767	520	23
1	OE5	GR 413	426	62
2	OE5	GR 773	528	443
1	OF1	GR 299	537	40.6
2	OF1	GR 713	747	54.4

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 13)

Group No.	From ONA [a]	To GR [b] - Group	Length , m	Group Demand m^3/d
1	OF2	GR 19	687	40
2	OF2	GR159	493	44
3	OF2	GR670	391	292
1	OF3	GR 31	700	55
2	OF3	GR294	553	87
3	OF3	GR203	201	26
4	OF3	GR639	583	51
5	OF3	GR423	708	48
6	OF3	GR519	616	24
7	OF3	GR103	786	25
8	OF3	GR354	920	22
1	OF4	GR423	727	40
2	OF4	GR639	700	64
3	OF4	GR721	871	202
4	OF4	GR529	512	188
5	OF4	GR413	737	68
1	OG1	GR 729	356	327
2	OG1	GR 281	678	41
3	OG1	GR 730	978	105
4	OG1	GR 549	806	180
5	OG1	GR 600	797	639
1	OG2	GR 671	63	195
2	OG2	GR 270	823	33
3	OG2	GR 131	675	90
4	OG2	GR 122	672	29
5	OG2	GR 170	790	15
1	OG3	GR354	920	42
2	OG3	GR826	110	469
3	OG3	GR827	880	49
4	OG3	GR108	289	22
5	OG3	GR520	325	43
6	OG3	GR663	866	86

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 13)

Group No.	From ONA [a]	To GR [b] - Group	Length , m	Group Demand m^3/d
1	OG4	GR519	717	36
2	OG4	GR108	645	42
3	OG4	GR 30	726	24
4	OG4	GR827	328	27
5	OG4	GR203	971	360
1	OH1	GR600	538	70
2	OH1	GR678	503	54
3	OH1	GR712	743	334
4	OH1	GR686	350	81
5	OH1	GR348	544	108
6	OH1	GR760	943	47
1	OH2	GR596	1000	193
2	OH2	GR296	754	191
3	OH2	GR598	737	16
4	OH2	GR526	731	29
5	OH2	GR286	873	36
6	OH2	GR760	943	57
1	OH3	GR526	860	59
2	OH3	GR532	85	48
3	OH3	GR262	826	50
4	OH3	GR677	676	81
5	OH3	GR665	841	15
6	OH3	GR685	644	52
7	OH3	GR349	914	35
1	OI1	GA 421	538	399
2	OI1	GA 176	873	45
3	OI1	GA 407	854	22
4	OI1	GA 268	421	17
5	OI1	GA 287	779	66

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 13)

Group No.	From ONA [a]	To GR [b] - Group	Length , m	Group Demand m^3/d
6	OI1	GR 66	2	38
1	OI2	GR 108	957	21
2	OI2	GR 827	433	57
3	OI2	GR 256	329	347
4	OI2	GR 663	656	16
5	OI2	GR 711	279	105
6	OI2	GR 266	764	146
1	OI3	GR 262	661	52
2	OI3	GR 266	764	137
3	OI3	GR 506	613	16
4	OI3	GR 741	732	69
5	OI3	GR 572	783	21
1	OJ1	GR 778	451	45
2	OJ1	GR 625	525	72
3	OJ1	GR 595	986	19
4	OJ1	GR 779	812	36
5	OJ1	GR 594	994	608
1	OJ2	GR 598	984	38
2	OJ2	GR 685	580	105
3	OJ2	GR 595	991	78
4	OJ2	GR 779	675	32
5	OJ2	GR 350	411	214

a; ONA = Optimized Nominated Areas, b; GR = Green Areas

Table (A. 14) : Elevations of Green Areas (GRs) of the Study Area, , (Researcher)

GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl
GR1	814	GR41	823	GR81	864	GR121	793
GR2	806	GR42	799	GR82	868	GR122	897
GR3	800	GR43	775	GR83	897	GR123	892
GR4	802	GR44	775	GR84	815	GR124	942
GR5	800	GR45	774	GR85	783	GR125	960
GR6	770	GR46	772	GR86	851	GR126	921
GR7	788	GR47	775	GR87	864	GR127	948
GR8	858	GR48	775	GR88	780	GR128	940
GR9	860	GR49	775	GR89	790	GR129	932
GR10	857	GR50	810	GR90	801	GR130	893
GR11	919	GR51	841	GR91	806	GR131	921
GR12	930	GR52	822	GR92	787	GR132	945
GR13	948	GR53	836	GR93	801	GR133	901
GR14	981	GR54	811	GR94	800	GR134	943
GR15	898	GR55	803	GR95	803	GR135	889
GR16	910	GR56	881	GR96	787	GR136	760
GR17	959	GR57	826	GR97	791	GR137	781
GR18	792	GR58	858	GR98	821	GR138	849
GR19	819	GR59	790	GR99	815	GR139	875
GR20	765	GR60	790	GR100	813	GR140	871
GR21	859	GR61	830	GR101	809	GR141	873
GR22	740	GR62	755	GR102	788	GR142	836
GR23	806	GR63	850	GR103	792	GR143	759
GR24	768	GR64	840	GR104	801	GR144	771
GR25	759	GR65	849	GR105	798	GR145	795
GR26	925	GR66	774	GR106	808	GR146	785
GR27	970	GR67	769	GR107	802	GR147	781
GR28	781	GR68	781	GR108	800	GR148	780
GR29	800	GR69	780	GR109	795	GR149	781
GR30	784	GR70	847	GR110	965	GR150	845
GR31	795	GR71	924	GR111	900	GR151	843
GR32	865	GR72	918	GR112	915	GR152	850
GR33	820	GR73	919	GR113	926	GR153	860
GR34	832	GR74	908	GR114	926	GR154	795
GR35	780	GR75	870	GR115	900	GR155	848
GR36	754	GR76	860	GR116	888	GR156	764
GR37	785	GR77	874	GR117	887	GR157	759
GR38	821	GR78	879	GR118	885	GR158	765
GR39	830	GR79	795	GR119	941	GR159	827
GR40	832	GR80	864	GR120	912	GR160	850

Table (A. 14)

GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl
GR161	879	GR201	840	GR241	851	GR281	966
GR162	946	GR202	798	GR242	841	GR282	910
GR163	980	GR203	783	GR243	880	GR283	826
GR164	973	GR204	750	GR244	910	GR284	788
GR165	976	GR205	756	GR245	961	GR285	917
GR166	893	GR206	784	GR246	905	GR286	838
GR167	909	GR207	776	GR247	904	GR287	869
GR168	933	GR208	951	GR248	786	GR288	770
GR169	912	GR209	799	GR249	756	GR289	769
GR170	918	GR210	830	GR250	781	GR290	767
GR171	836	GR211	820	GR251	769	GR291	760
GR172	789	GR212	845	GR252	812	GR292	987
GR173	809	GR213	860	GR253	819	GR293	991
GR174	793	GR214	821	GR254	878	GR294	788
GR175	839	GR215	974	GR255	843	GR295	960
GR176	872	GR216	781	GR256	794	GR296	881
GR177	841	GR217	837	GR257	789	GR297	814
GR178	845	GR218	859	GR258	791	GR298	956
GR179	772	GR219	799	GR259	774	GR299	831
GR180	779	GR220	924	GR260	888	GR300	830
GR181	779	GR221	785	GR261	825	GR301	832
GR182	791	GR222	843	GR262	795	GR302	923
GR183	775	GR223	825	GR263	796	GR303	834
GR184	778	GR224	909	GR264	825	GR304	773
GR185	775	GR225	855	GR265	830	GR305	881
GR186	870	GR226	864	GR266	795	GR306	830
GR187	825	GR227	843	GR267	831	GR307	845
GR188	834	GR228	819	GR268	860	GR308	875
GR189	814	GR229	1001	GR269	845	GR309	797
GR190	927	GR230	1035	GR270	860	GR310	820
GR191	871	GR231	802	GR271	932	GR311	832
GR192	925	GR232	798	GR272	926	GR312	780
GR193	887	GR233	757	GR273	896	GR313	775
GR194	865	GR234	765	GR274	797	GR314	739
GR195	864	GR235	760	GR275	883	GR315	901
GR196	803	GR236	811	GR276	954	GR316	877
GR197	924	GR237	860	GR277	949	GR317	882
GR198	886	GR238	817	GR278	939	GR318	884
GR199	846	GR239	847	GR279	745	GR319	882
GR200	780	GR240	850	GR280	797	GR320	880

Table (A. 14)

GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl
GR321	881	GR361	814	GR401	859	GR441	776
GR322	880	GR362	812	GR402	905	GR442	779
GR323	876	GR363	951	GR403	850	GR443	782
GR324	875	GR364	924	GR404	900	GR444	856
GR325	875	GR365	824	GR405	884	GR445	840
GR326	875	GR366	923	GR406	983	GR446	840
GR327	876	GR367	923	GR407	848	GR447	945
GR328	878	GR368	802	GR408	785	GR448	804
GR329	868	GR369	798	GR409	823	GR449	821
GR330	868	GR370	786	GR410	868	GR450	778
GR331	867	GR371	760	GR411	808	GR451	772
GR332	867	GR372	750	GR412	951	GR452	828
GR333	872	GR373	851	GR413	753	GR453	823
GR334	870	GR374	953	GR414	820	GR454	790
GR335	865	GR375	850	GR415	875	GR455	967
GR336	868	GR376	995	GR416	796	GR456	875
GR337	765	GR377	999	GR417	899	GR457	890
GR338	764	GR378	918	GR418	810	GR458	784
GR339	761	GR379	925	GR419	845	GR459	784
GR340	765	GR380	919	GR420	1040	GR460	785
GR341	765	GR381	910	GR421	856	GR461	775
GR342	767	GR382	915	GR422	770	GR462	785
GR343	846	GR383	928	GR423	804	GR463	783
GR344	757	GR384	770	GR424	778	GR464	762
GR345	765	GR385	786	GR425	1044	GR465	781
GR346	750	GR386	784	GR426	1043	GR466	922
GR347	790	GR387	860	GR427	1018	GR467	905
GR348	925	GR388	871	GR428	1241	GR468	953
GR349	840	GR389	881	GR429	830	GR469	957
GR350	823	GR390	844	GR430	840	GR470	1027
GR351	818	GR391	815	GR431	835	GR471	921
GR352	849	GR392	838	GR432	1164	GR472	932
GR353	800	GR393	908	GR433	1239	GR473	944
GR354	800	GR394	799	GR434	972	GR474	912
GR355	802	GR395	808	GR435	968	GR475	807
GR356	785	GR396	815	GR436	1046	GR476	798
GR357	785	GR397	797	GR437	786	GR477	807
GR358	785	GR398	844	GR438	776	GR478	758
GR359	785	GR399	828	GR439	778	GR479	759
GR360	771	GR400	845	GR440	770	GR480	762

Table (A. 14)

GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl
GR481	742	GR521	808	GR561	782	GR601	739
GR482	775	GR522	815	GR562	792	GR602	891
GR483	779	GR523	795	GR563	900	GR603	905
GR484	777	GR524	784	GR564	1095	GR604	891
GR485	920	GR525	845	GR565	1088	GR605	922
GR486	1040	GR526	844	GR566	842	GR606	922
GR487	972	GR527	888	GR567	864	GR607	910
GR488	875	GR528	985	GR568	805	GR608	908
GR489	862	GR529	763	GR569	796	GR609	944
GR490	914	GR530	761	GR570	781	GR610	927
GR491	825	GR531	961	GR571	792	GR611	933
GR492	818	GR532	828	GR572	798	GR612	946
GR493	966	GR533	935	GR573	795	GR613	933
GR494	936	GR534	990	GR574	913	GR614	831
GR495	811	GR535	814	GR575	805	GR615	824
GR496	816	GR536	841	GR576	791	GR616	834
GR497	816	GR537	796	GR577	800	GR617	859
GR498	814	GR538	913	GR578	812	GR618	860
GR499	811	GR539	945	GR579	795	GR619	807
GR500	801	GR540	929	GR580	820	GR620	839
GR501	795	GR541	816	GR581	775	GR621	1037
GR502	794	GR542	761	GR582	933	GR622	1019
GR503	761	GR543	810	GR583	934	GR623	855
GR504	764	GR544	769	GR584	938	GR624	856
GR505	770	GR545	760	GR585	918	GR625	855
GR506	801	GR546	757	GR586	748	GR626	857
GR507	805	GR547	894	GR587	778	GR627	890
GR508	833	GR548	816	GR588	790	GR628	1072
GR509	849	GR549	1005	GR589	810	GR629	829
GR510	930	GR550	878	GR590	800	GR630	749
GR511	936	GR551	785	GR591	826	GR631	940
GR512	920	GR552	778	GR592	748	GR632	744
GR513	916	GR553	777	GR593	750	GR633	794
GR514	794	GR554	805	GR594	875	GR634	804
GR515	793	GR555	775	GR595	864	GR635	812
GR516	800	GR556	759	GR596	880	GR636	760
GR517	795	GR557	762	GR597	852	GR637	754
GR518	800	GR558	767	GR598	850	GR638	769
GR519	794	GR559	764	GR599	989	GR639	775
GR520	809	GR560	778	GR600	977	GR640	780

Table (A. 14)

GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl
GR641	920	GR681	1097	GR721	791	GR761	845
GR642	760	GR682	984	GR722	944	GR762	791
GR643	735	GR683	876	GR723	991	GR763	790
GR644	755	GR684	882	GR724	965	GR764	780
GR645	1031	GR685	830	GR725	973	GR765	786
GR646	1015	GR686	936	GR726	953	GR766	787
GR647	750	GR687	916	GR727	960	GR767	812
GR648	744	GR688	923	GR728	835	GR768	775
GR649	753	GR689	934	GR729	952	GR769	766
GR650	761	GR690	814	GR730	1039	GR770	749
GR651	768	GR691	816	GR731	877	GR771	744
GR652	764	GR692	810	GR732	751	GR772	744
GR653	890	GR693	815	GR733	739	GR773	745
GR654	840	GR694	818	GR734	770	GR774	745
GR655	823	GR695	810	GR735	835	GR775	743
GR656	834	GR696	810	GR736	801	GR776	740
GR657	825	GR697	861	GR737	835	GR777	839
GR658	816	GR698	855	GR738	751	GR778	891
GR659	816	GR699	856	GR739	808	GR779	836
GR660	910	GR700	841	GR740	778	GR780	902
GR661	988	GR701	815	GR741	775	GR781	745
GR662	815	GR702	812	GR742	767	GR782	764
GR663	804	GR703	810	GR743	815	GR783	890
GR664	805	GR704	790	GR744	783	GR784	1066
GR665	835	GR705	789	GR745	780	GR785	909
GR666	840	GR706	786	GR746	789	GR786	868
GR667	815	GR707	880	GR747	824	GR787	829
GR668	815	GR708	828	GR748	824	GR788	897
GR669	800	GR709	789	GR749	826	GR789	1065
GR670	835	GR710	828	GR750	847	GR790	930
GR671	899	GR711	795	GR751	801	GR791	850
GR672	873	GR712	981	GR752	886	GR792	847
GR673	810	GR713	845	GR753	769	GR793	835
GR674	963	GR714	874	GR754	768	GR794	843
GR675	750	GR715	886	GR755	774	GR795	1086
GR676	752	GR716	893	GR756	771	GR796	910
GR677	831	GR717	815	GR757	770	GR797	790
GR678	924	GR718	908	GR758	810	GR798	989
GR679	929	GR719	847	GR759	746	GR799	855
GR680	1024	GR720	754	GR760	898	GR800	926

Table (A. 14)

GR	Elevation amsl	GR	Elevation amsl	GR	Elevation amsl
GR801	765	GR811	869	GR821	849
GR802	810	GR812	830	GR822	936
GR803	916	GR813	1012	GR823	960
GR804	870	GR814	995	GR824	893
GR805	865	GR815	882	GR825	914
GR806	894	GR816	1024	GR826	795
GR807	1058	GR817	841	GR827	785
GR808	1025	GR818	869		
GR809	900	GR819	1080		
GR810	876	GR820	899		

Table (A. 15) : Elevations of the Optimized Nominated Areas, , (Researcher)

No.	ONA [a]	Elevation , amsl	No.	ONA [a]	Elevation amsl
1	OA1	810	17	OF2	833
2	OB1	840	18	OF3	780
3	OB2	780	19	OF4	759
4	OB3	756	20	OG1	970
5	OB4	738	21	OG2	897
6	OC1	830	22	OG3	800
7	OC2	780	23	OG4	775
8	OC3	788	24	OH1	925
9	OC4	756	25	OH2	854
10	OD1	732	26	OH3	827
11	OE1	940	27	OI1	849
12	OE2	825	28	OI2	796
13	OE3	841	29	OI3	790
14	OE4	799	30	OJ1	864
15	OE5	745	31	OJ2	816
16	OF1	846			

a; ONA = Optimized Nominated Areas

Table (A. 16) : Results of The Reclaimed Water Pipe Details, , (Researcher)

Pipe	Diameter, mm	Q, m^3/s	Velocity, m/s	Length, m
OA1 - GR 411	40	0.00083	0.99	501
OA1 - GR 717	75	0.00266	0.90	393
OA1 - GR 693	40	0.00064	0.77	391
OA1 - GR 758	40	0.00060	0.72	705
OA1 - GR 252	50	0.00105	0.80	752
OA1 - GR 228	40	0.00072	0.86	187
OA1 - GR 537	40	0.00091	1.08	807
OB1 - GR 391	50	0.00097	0.74	694
OB1 - GR 536	40	0.00070	0.84	824
OB1- GR 226	90	0.00440	1.03	976
OB2 - GR 35	63	0.00157	0.76	686
OB2 - GR 766	50	0.00103	0.78	780
OB2 - GR 633	40	0.00067	0.80	889
OB2 - GR 250	25	0.00024	0.72	146
OB3 -GR143	25	0.00018	0.60	154
OB3 -GR 732	180	0.01891	1.11	382
OB3 -GR 344	40	0.00100	1.20	522
OB4 -GR 733	180	0.01781	1.05	148
OB4 -GR 63	32	0.00044	0.83	957
OB4 -GR 346	63	0.00205	0.99	606
OC1 - GR 311	40	0.00079	0.94	220
OC1 - GR 629	50	0.00142	1.09	420
OC1 - GR 691	50	0.00155	1.18	800
OC1 - GR 222	32	0.00043	0.81	412
OC1 - GR 728	75	0.00270	0.91	394
OC1 - GR 719	50	0.00143	1.09	786
OC1 - GR 226	25	0.00034	1.05	800
OC1 - GR 150	50	0.00097	0.75	786
OC1 - GR 351	63	0.00163	0.78	452
OC1 - GR 703	40	0.00069	0.82	684
OC2 - GR 88	110	0.00599	0.94	77
OC2 - GR 28	40	0.00075	0.90	472
OC2 - GR 370	20	0.00014	0.70	461

Table (A. 16)

Pipe	Diameter, mm	Q, m^3/s	Velocity, m/s	Length, m
OC3 - GR 370	20	0.00023	1.16	444
OC3 - GR 7	50	0.00118	0.90	370
OC3 - GR 5	50	0.00118	0.90	691
OC3 - GR 3	50	0.00135	1.03	973
OC3 - GR 70	50	0.00147	1.12	546
OC4 -GR290	63	0.00179	0.86	705
OC4 -GR 157	40	0.00063	0.75	59
OC4 -GR 648	75	0.00300	1.01	674
OC4 -GR 25	63	0.00173	0.83	997
OC4 -GR 158	50	0.00125	0.95	498
OC4 -GR 63	40	0.00078	0.93	763
OD1 -GR 733	25	0.00036	1.11	787
OD1 -GR 290	63	0.00217	1.05	917
OD1 -GR 346	63	0.00158	0.76	640
OD1 -GR 63	50	0.00091	0.70	668
OE1 -GR 722	63	0.00177	0.85	104
OE1 -GR 724	160	0.01201	0.89	963
OE1 -GR 605	50	0.00094	0.72	449
OE1 -GR 608	50	0.00105	0.81	807
OE1 -GR 383	50	0.00104	0.80	522
OE1 -GR 611	50	0.00113	0.86	500
OE2 -GR 343	25	0.00026	0.78	505
OE2 -GR 491	180	0.01687	0.99	338
OE2 -GR 767	50	0.00102	0.78	695
OE2 -GR 283	50	0.00102	0.78	96
OE3 -GR 343	63	0.00176	0.85	641
OE3 -GR 430	90	0.00376	0.88	987
OE3 -GR 713	180	0.01674	0.98	63
OE3 -GR 456	25	0.00035	1.06	904
OE3 -GR 307	40	0.00086	1.03	92
OE3 -GR 299	40	0.00069	0.83	948
OE4 -GR 431	110	0.00709	1.11	655
OE4 -GR 429	90	0.00378	0.89	729
OE4 -GR 430	32	0.00042	0.79	802

Table (A. 16)

Pipe	Diameter, mm	Q, m^3/s	Velocity, m/s	Length, m
OE4 -GR 579	25	0.00031	0.96	286
OE4 -GR 767	25	0.00027	0.82	520
OE5 -GR 413	40	0.00072	0.87	426
OE5 -GR773	110	0.00513	0.81	528
OF1 - GR 299	32	0.00047	0.88	537
OF1 - GR 713	40	0.00063	0.75	747
OF2 -GR19	32	0.00046	0.88	687
OF2 -GR 670	90	0.00338	0.79	391
OF2 -GR 159	32	0.00051	0.96	493
OF3 -GR 31	40	0.00063	0.75	700
OF3 -GR 294	40	0.00100	1.19	553
OF3 -GR 203	25	0.00029	0.89	201
OF3 -GR 639	32	0.00057	1.07	583
OF3 -GR 423	32	0.00056	1.05	708
OF3 -GR 519	25	0.00027	0.81	616
OF3 -GR 103	25	0.00028	0.84	786
OF3 -GR 354	25	0.00025	0.76	920
OF4 - GR 423	32	0.00046	0.86	727
OF4 - GR 639	40	0.00075	0.90	700
OF4 - GR 721	63	0.00234	1.10	871
OF4 - GR 529	63	0.00218	1.05	512
OF4 - GR 413	40	0.00079	0.95	737
OG1 - GR 729	90	0.00378	0.89	356
OG1 - GR 281	32	0.00047	0.89	678
OG1 - GR 730	50	0.00122	0.93	978
OG1 - GR 549	63	0.00208	1.00	806
OG1 - GR 600	110	0.00740	1.16	797
OG2 -GR671	63	0.00226	1.09	63
OG2 -GR 270	25	0.00038	1.15	823
OG2 -GR 131	50	0.00105	0.80	675
OG2 -GR 122	25	0.00033	1.01	672
OG2 -GR 170	20	0.00017	0.84	790

Table (A. 16)

Pipe	Diameter, mm	Q, m^3/s	Velocity, m/s	Length, m
OG3 -GR 354	32	0.00049	0.93	920
OG3 -GR 826	110	0.00543	0.85	110
OG3 -GR 827	32	0.00057	1.08	880
OG3 -GR 108	25	0.00025	0.76	289
OG3 -GR 520	32	0.00050	0.95	325
OG3 -GR 663	50	0.00099	0.76	866
OG4 - GR 519	32	0.00041	0.78	717
OG4 - GR 108	32	0.00049	0.92	645
OG4 - GR 30	25	0.00028	0.87	726
OG4 – GR 827	25	0.00031	0.96	328
OG4 – GR 203	90	0.00416	0.98	971
OH1 - GR 600	40	0.00080	0.96	538
OH1 - GR 678	40	0.00063	0.76	503
OH1 - GR 712	90	0.00386	0.91	743
OH1 - GR 686	40	0.00094	1.12	350
OH1 - GR 348	50	0.00125	0.95	544
OH1 - GR 760	32	0.00054	1.02	943
OH2 - GR 596	63	0.00223	1.08	1000
OH2 - GR 296	63	0.00222	1.07	754
OH2 - GR 598	20	0.00018	0.89	737
OH2 - GR 526	25	0.00033	1.02	731
OH2 - GR 286	32	0.00041	0.76	873
OH2 - GR 760	40	0.00067	0.80	943
OH3 -GR 526	40	0.00068	0.81	860
OH3 -GR 532	32	0.00055	1.04	85
OH3 -GR 262	32	0.00057	1.07	826
OH3 -GR 677	40	0.00093	1.12	676
OH3 -GR 665	20	0.00017	0.83	841
OH3 -GR 685	32	0.00060	1.13	644
OH3 -GR 349	32	0.00040	0.76	914
OI1 -GR421	90	0.00461	1.08	538
OI1 -GR 176	32	0.00052	0.98	873
OI1 -GR 407	25	0.00025	0.78	854
OI1 -GR 268	20	0.00019	0.93	421
OI1 -GR 287	40	0.00076	0.91	779

Table (A. 16)

Pipe	Diameter, mm	Q, m^3/s	Velocity, m/s	Length, m
OI1 -GR 66	32	0.00044	0.82	2
OI2 -GR108	25	0.00024	0.73	957
OI2 -GR 827	40	0.00065	0.78	433
OI2 -GR 256	90	0.00395	0.93	329
OI2 -GR 663	20	0.00019	0.97	656
OI2 -GR 711	50	0.00122	0.93	279
OI2 -GR 266	63	0.00169	0.82	764
OI3 - GR 262	32	0.00060	1.12	661
OI3 - GR 266	63	0.00159	0.76	764
OI3 - GR 506	20	0.00019	0.94	613
OI3 – GR 741	40	0.00080	0.95	732
OI3 – GR 572	25	0.00024	0.74	783
OJ1 –GR 778	32	0.00052	0.99	451
OJ1 -GR 625	40	0.00083	0.99	525
OJ1 -GR 595	20	0.00022	1.07	986
OJ1 -GR 779	32	0.00041	0.77	812
OJ1 -GR 594	110	0.00704	1.11	994
OJ2 -GR 598	32	0.00044	0.84	984
OJ2 -GR 685	50	0.00121	0.93	580
OJ2 -GR 595	40	0.00091	1.09	991
OJ2 -GR 779	25	0.00037	1.14	675
OJ2 -GR 350	63	0.00248	1.19	411

Table (A. 17) : Results of the Pump Heads of the Pressurized Pipes, , (Researcher)

Pipe	ELD [a]	hf m	Pump Head (H) , m	Pipe	ELD [a]	hf m	Pump Head (H) , m
OA1 - GR 411	-2.0	4.3	9	OC4 - GR 25	-7	27	41
OA1 - GR 717	-9.0	6.5	18	OC4 - GR 158	-14	14	32
OA1 - GR 693	-9.0	18.6	33	OC4 - GR 63	-3	34	46
OA1 - GR 758	-3.0	35.1	48	OD1 - GR 290	-32	21	60
OA1 - GR 252	-4.0	27.5	39	OD1 - GR 346	-22	18	46
OA1 - GR 228	-12.0	6.4	22	OD1 - GR 63	-27	27	61
OA1 - GR 537	11.0	32.5	30	OE1 - GR 724	-24	8	35
OB1 - GR391	21	27	14	OE1 - GR 605	14	12	2
OB1 - GR536	-5	39	54	OE1 - GR 608	28	30	9
OB1- GR 226	-28	15	47	OE1 - GR 383	8	19	17
OB2 - GR 35	-4	20	30	OE1 - GR 611	3	18	20
OB2 - GR766	-11	29	48	OE2 - GR 343	-25	46	82
OB2 - GR633	-18	43	73	OE2 - GR 491	-4	2	9
OB2 - GR 250	-5	14	24	OE2 - GR 767	9	26	25
OB3 - GR143	-8	18	31	OE3 - GR 343	-8	17	31
OB3 - GR 344	-6	19	30	OE3 - GR 430	18	16	4
OB4 - GR 63	-20	61	95	OE3 - GR 456	-38	67	120
OB4 - GR 346	-16	15	35	OE3 - GR 307	-7	4	14
OC1 - GR 311	-6	10	19	OE3 - GR 299	7	45	50
OC1 - GR 629	-3	7	14	OE4 - GR 431	-15	7	26
OC1 - GR 691	11	23	19	OE4 - GR 429	-15	12	31
OC1 - GR 222	-17	18	41	OE4 - GR 430	-25	53	90
OC1 - GR 728	-8	7	19	OE4 - GR 579	0	23	29
OC1 - GR 719	-21	24	52	OE4 - GR 767	-17	45	74
OC1 - GR 226	-38	60	112	OE5 - GR 413	-11	20	37
OC1 - GR 150	-19	30	57	OE5 - GR 773	-4	7	14
OC1 - GR 351	8	12	8	OF1 - GR 299	11	23	19
OC1 - GR 703	17	33	25	OF1 - GR 159	15	41	36
OC2 - GR 28	-5	23	35	OF2 - GR19	10.4	51.8	53.8
OC2 - GR 370	-10	45	67	OF3 - GR 31	-19	36	64
OC3 - GR 370	0	31	39	OF3 - GR 294	-12	21	39
OC3 - GR 5	-14	24	44	OF3 - GR 203	-6	17	28
OC3 - GR 3	-14	30	53	OF3 - GR 639	1	32	39
OC3 - GR 70	6	13	12	OF3 - GR 423	-25	39	74
OC4 - GR290	-8	18	32	OF3 - GR 519	-17	54	85
OC4 - GR 157	-7	2	11	OF3 - GR 103	-15	68	99
OC4 - GR 648	7	13	10	OF3 - GR 354	-24	85	128

a: ELD = Elevation difference between the nominated area and the green area – 4m, depth of the treatment unit.

Table (A. 17)

Pipe	ELD [a]	hf m	Pump Head (H) , m	Pipe	ELD [a]	hf m	Pump Head (H) , m
OF4 - GR 423	-47	45	103	OH3 - GR 262	28	45	27
OF4 - GR 639	-20	32	60	OH3 - GR 677	-8	27	42
OF4 - GR 721	-25	20	51	OH3 - GR 665	-12	95	127
OF4 - GR529	-8	12	25	OH3 - GR 685	-6	34	49
OF4 - GR 413	2	32	39	OH3 - GR 349	-17	62	93
OG1 - GR 281	1	41	51	OI1 - GR421	-10	8	21
OG1 - GR 730	-73	33	114	OI1 - GR 176	-26	50	88
OG1 - GR 549	-39	19	64	OI1 - GR 407	-2	117	82
OG1 - GR 600	6	9	6	OI1 - GR 268	-14	30	53
OG2 - GR 270	33	58	38	OI1 - GR 287	-23	35	67
OG2 - GR 131	-28	25	59	OI2 - GR 108	-8	90	118
OG2 - GR 122	-4	51	68	OI2 - GR 827	7	22	21
OG2 - GR 170	-25	82	125	OI2 - GR 256	-2	5	11
OG3 - GR 354	-4	55	72	OI2 - GR 663	-9	66	90
OG3 - GR 827	11	48	48	OI2 - GR 711	-3	9	16
OG3 - GR 108	-4	27	38	OI2 - GR 266	0	21	27
OG3 - GR 520	-13	19	38	OI3 - GR 262	0	35	44
OG3 - GR 663	-8	33	50	OI3 - GR 266	0	22	28
OG4 - GR 519	-22	48	81	OI3 - GR 506	-6	54	72
OG4 - GR 108	-29	39	77	OI3 - GR 741	20	32	20
OG4 - GR 30	-12	62	88	OI3 - GR 572	-3	73	93
OG4 - GR 827	-14	26	47	OJ1 - GR778	-30	26	63
OG4 - GR 203	-11	15	31	OJ1 - GR 625	5	22	24
OH1 - GR 600	-39	23	70	OJ1 - GR 595	-1	100	123
OH1 - GR 678	-3	25	36	OJ1 - GR 779	24	54	43
OH1 - GR 712	-54	12	71	OJ1 - GR 594	-10	11	25
OH1 - GR 686	-16	14	34	OJ2 - GR 598	-34	62	111
OH1 - GR 348	-5	18	28	OJ2 - GR 685	-14	19	39
OH1 - GR 760	22	53	43	OJ2 - GR 595	-46	40	95
OH2 - GR 596	-30	23	60	OJ2 - GR 779	-20	48	80
OH2 - GR 296	-15	17	38	OJ2 - GR 350	-7	9	19
OH2 - GR 598	0	73	89				
OH2 - GR 526	6	56	63				
OH2 - GR 286	12	59	61				
OH2 - GR 760	-48	46	105				
OH3 - GR526	-21	42	73				

a: ELD = Elevation difference between the nominated area and the green area – 4m, depth of the treatment unit.

APPENDIX B

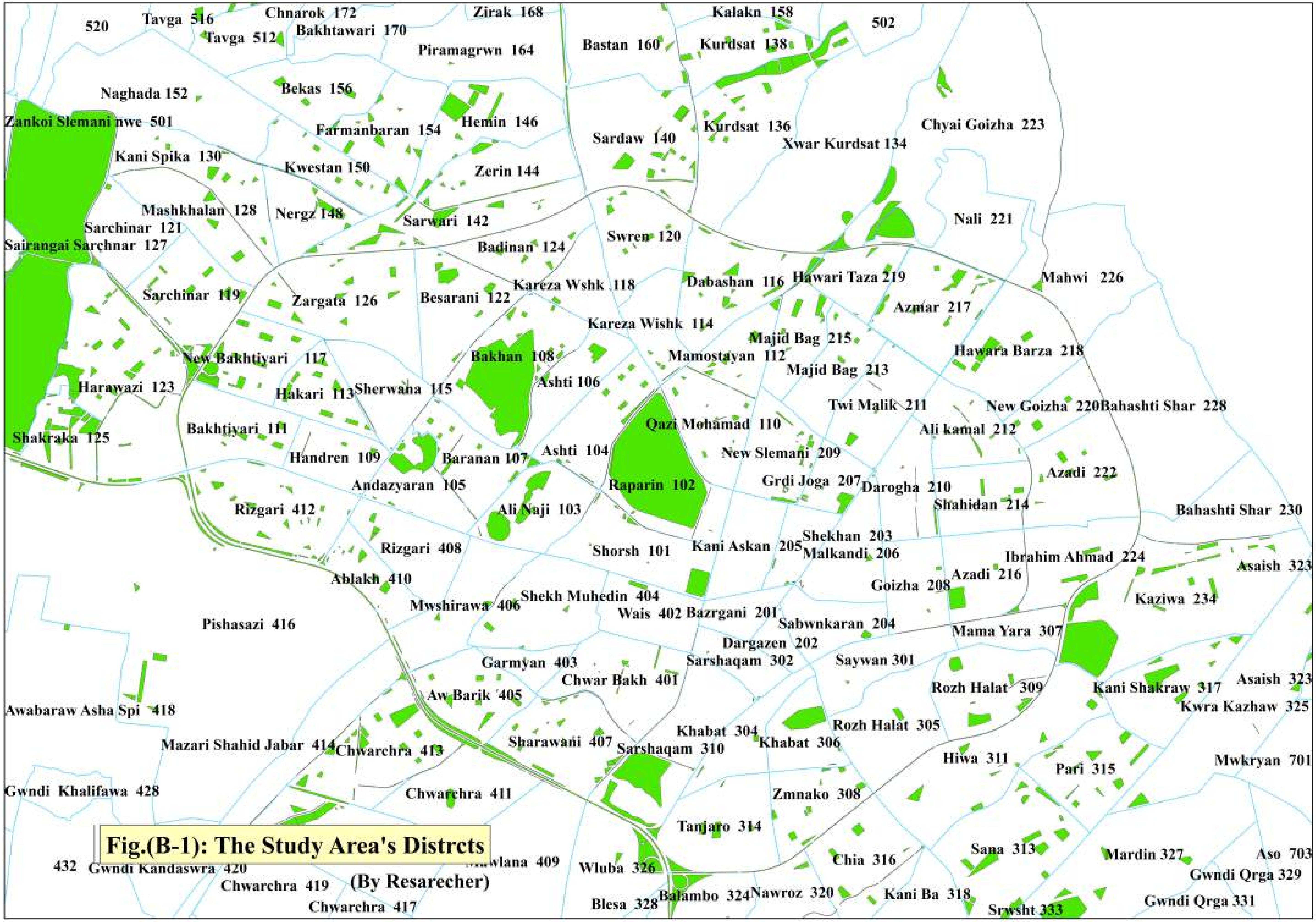

Fig.(B-1): The Study Area's Distrcts

(By Resarecher)

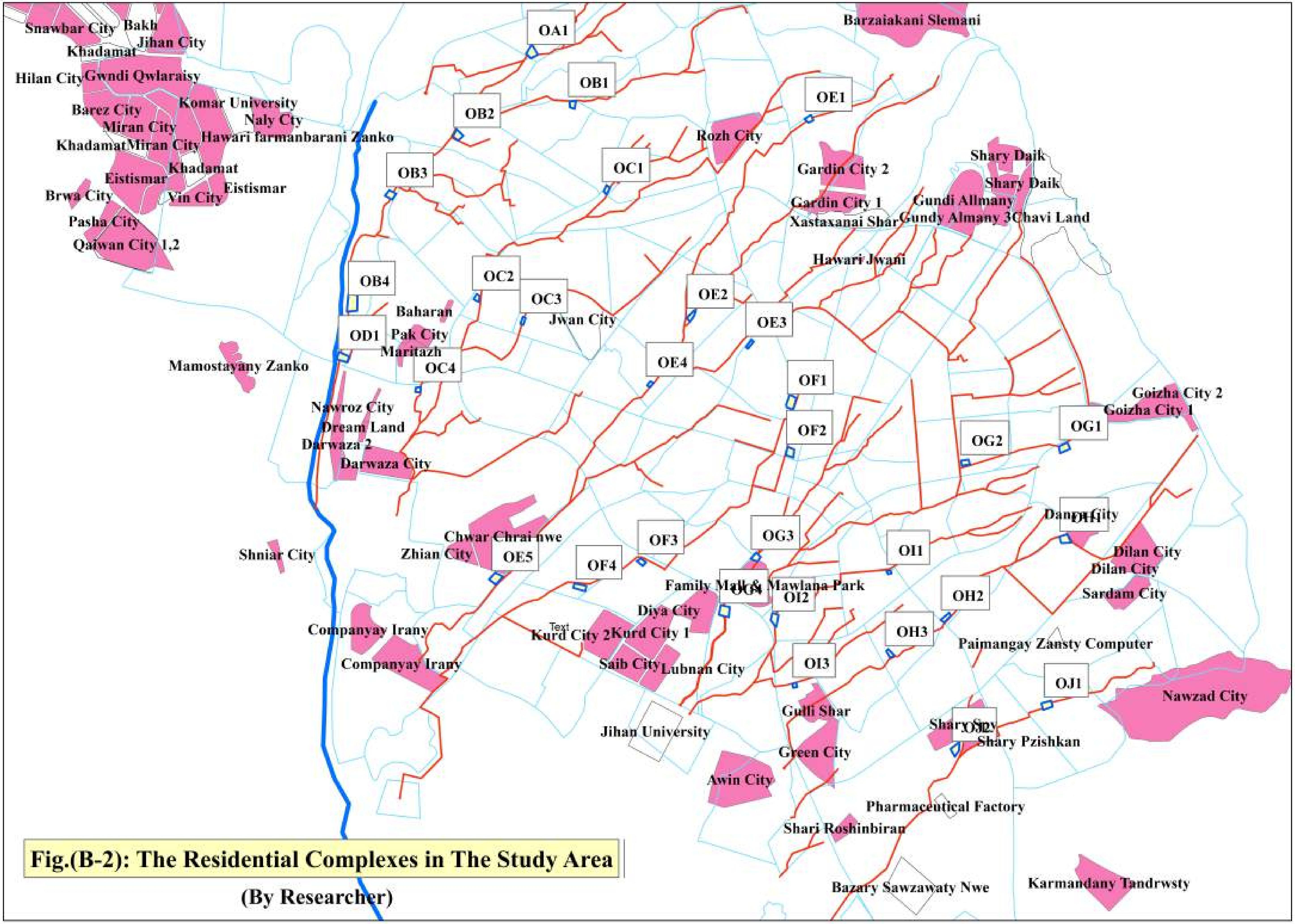

Fig.(B-2): The Residential Complexes in The Study Area

(By Researcher)

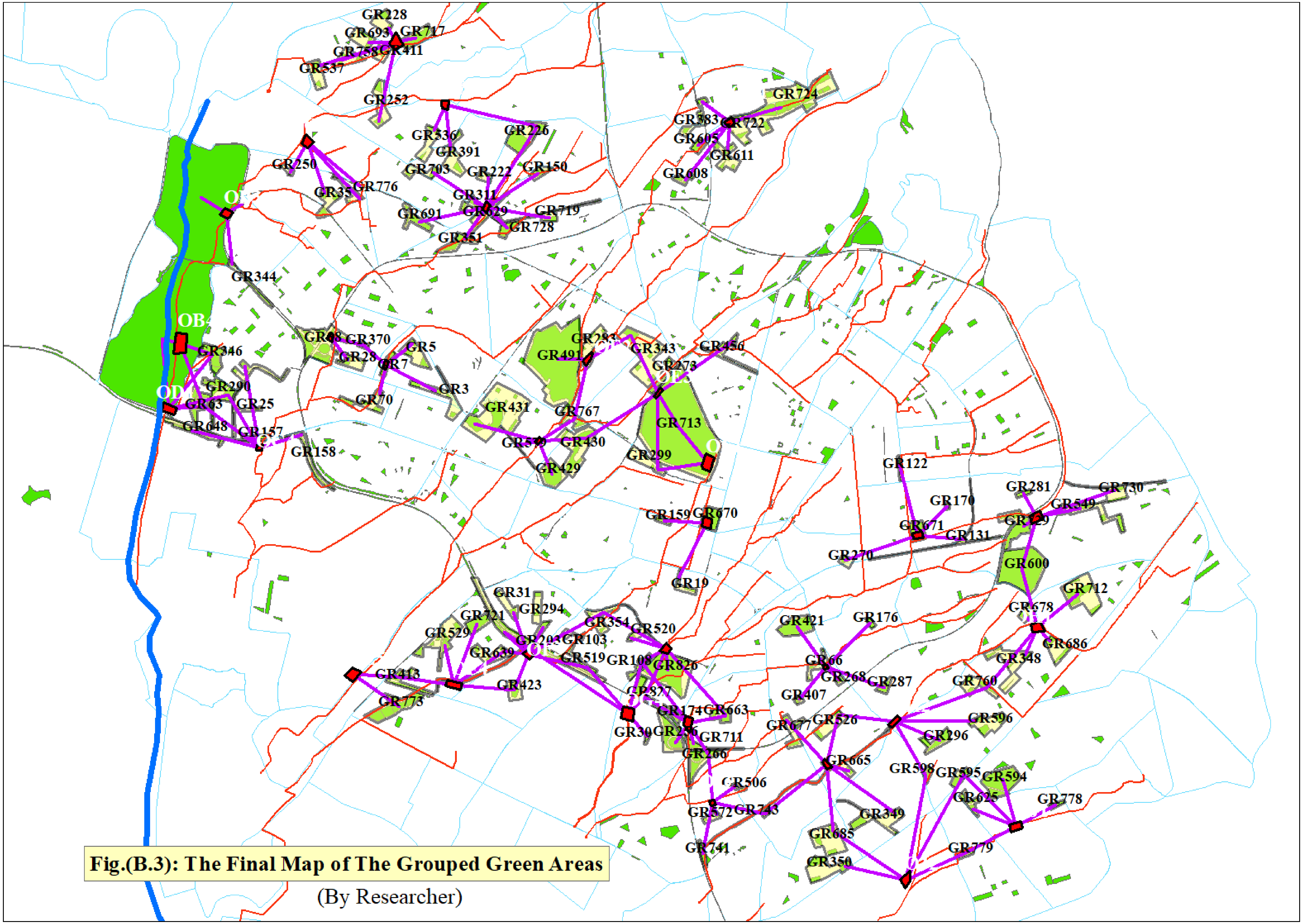

Fig.(B.3): The Final Map of The Grouped Green Areas

(By Researcher)

إختيار أفضل المواقع لمياه المجاري المنزلية البلدية اللامركزية في مدينة السليمانية بإستخدام GIS وAHP مع معالجات فعالة محتملة و قابلية إعادة الإستعمال

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الخلاصــــــــــــة

تعاني مدينة السليمانية من نقص كبير في احتياجات المياه اليومية بسبب تزايد اعداد السكان والتغيرات المناخية والافراط الكبيرفي الاستخدام للمياه . ومن الحلول المقترحة لحل مشكلة نقص المياه هو انشاء عدد من محطات التصفية اللامركزية - (Decentralized Treatment Units DWWTUs) والتي تتميز بكفائتها في المعالجة و بكلفتها القليلة وسهولة انشاءها و تشغيلها ومن ثم اعادة استخدام مياه الصرف الصحي المُعالجة فيها لاغراض سقي الحدائق الخضراء المجاورة لها في المدينة . لايوجد محطة لمعالجة مياه الصرف الصحي في مدينة السليمانية و أن مياه الصرف الصحي تصرف الى جدول قايلسان من عدة مخارج تصريف (Sewer Outlets) دون معالجة مما سبب مشاكل بيئية كبيرة في المنطقة . تم اختيار نوع محطات معالجة الحمأة المنشطة ذات التهوية الممتدة (Activated Sludge Extended Aeration Package Plants) في هذه الدراسة.

أحد الاهداف الرئيسية للبحث هو اختيار المواقع المناسبة لهذه المحطات في المدينة وعليه تم انتقاء 134 موقع داخل المدينة كخطوة اولية اعتماداً على معايير معينة و تمت نمذجة موديل رياضي باستخدام الـ GIS و باستخدام عملية التسلسل الهرمي AHP و ذلك لتقييم المواقع المختارة و تصنيفها . تم اعتماد خمسة معايير في النموذج و هي (1) مساحات المواقع المختارة ،(2) بُعد موقع المحطات عن المساحات الخضراء المجاورة لها , (3) الكثافة السكانية في المناطق التي تقع فيها المحطات ، (4) ميل سطح أرض المواقع المختارة ، (5) عمق انابيب مياه الصرف الصحي الرئيسية عند كل مساحة مختارة . المحددات التي تم اخذها بنظر الاعتبار في النموذج هي : (1) المسافات من موقع المحطات الى الابنية السكنية يجب ان لاتقل عن 30 م ، (2) المسافة من خط انبوب مياه الصرف الرئيسي وموقع المحطة يجب ان لا تزيد عن 50 م . نتائج تصنيف المساحات الكلية كانت بالشكل التالي: 10 % من المساحات محضور انشاء اي محطة فيها ، 2% من المواقع معتدل الملائمة ، 14% ملائم ، 30 % ملائم جداً ، 37% ملائم للغاية ، 7 % ملائم جداً جداً . لكل موقع مختار(134 موقع) هناك اكثر من تصنيف و لكل موقع تم حساب المعدل الوزني المعياري (NWAV) Normalized Weighted Average لنسب الملائمة (Suitability %)لكل مساحة مختارة . تم اختيار المساحات التي قيم الـNWAV لها اكثر من 0.5 و في المحصلة من مجموع الـ 134 موقع تم الحصول على 31 موقع مناسب لانشاء المحطات فيها.

الهدف الثاني من البحث هو انشاء نموذج رياضي لايجاد أقل كلفة F_{min} لمعالجة مياه الصرف الصحي من محطات التصفية اللامركزية ونقلها الى المناطق الخضراء المجاورة لها. حسابات الكلفة تتضمن كلفة انشاء وتشغيل و صيانة المحطات اللامركزية وكلف ضخ المياه المُعالجة و كلفة شبكات انابيب نقل مياه الصرف المُعالجة. ان عدد المناطق الخضراء للمدينة 827 حديقة مختلفة المساحات وتبلغ المساحة الكليه لاجمالي المناطق الخضراء $4.74\ Km^2$. إن نقل مياه الصرف المُعالجة الى المناطق الخضراء سوف يتم بواسطة انابيب ذو تدفق أما مضغوط Pressurized Flow Pipes أو انابيب تدفق بواسطة الجريان بالجاذبية Gravitational Pipe Flow و ذلك إعتماداً على مقدار فروقات المناسيب وخسائر الاحتكاك Head losses .

تم انشاء نموذج لمصفوفة نقل Transportation Matrix Model بحجم [*31 x 827*] لتمثيل كلفة نقل مياه الصرف المُعالجة من محطات التصفية اللامركزية (Origin) الى الحدائق الخضراء المجاورة (Destinations) . تم استخدام برنامج الـ (ArcGIS 10.2) لحساب ألاطوال و المسارات المثلى للانابيب الناقلة للمياه المُعالجة و ذلك باستخدام طريقة Network Analysis – OD Matrix و ارتفاعات المناسيب لمواقع المحطات اللامركزية و المناطق الخضراء تم حسابها من خرائط الـ DTM في برنامج الـ (ArcGIS 10.2) .

استخدمت طريقة الخوارزمية الجينية على شكل مصفوفة Genetic Algorithm in a Matrix Form لحل النموذج الرياضي لايجاد الكلفة المثلى لـ F_{min} (Optimum) و باستخدام رموز برنامج الـ Matlab 2018 a Programming Code . تم ايجاد عدد من الحلول العشوائية Random Solutions اعتماداً على كميات مختلفة من تصاريف المياه المعالجة و أعداد المجتمع Population (Np) التي تم اعتمادها كان مساوياً الى : [Np = 100, 200, 300, 400, 500, 600, 700, 800, 900 and 1000] و كل حل من هذه الحلول يمثل كروموسوم جيني . لكل Np تم عمل Run للبرنامج ثلاث مرات مع تكرار أربع مرات Iterations لكل Run و أقل قيمة لـ Np والتي عندها اصبحت النتائج مستقرة Stable Results كانت عند $Np = 500$ و ايضاً تم تغيير الـ Crossovering Points (PCO) للوصول الى الحل الامثل لقيمة F_{min} و الذي كان عند $Np = 500$ و $PCO = 632$.

اعتماداً على قيمة لـ F_{min} و التي تم ايجادها من الحل الامثل تم ايجاد احجام محطات التصفية اللامركزية الـ 31 و التي تتراوح ما بين $150\ m^3/day – 2,100\ m^3/day$ و اجمالي مياه الصرف الصحي المُعالجة في اليوم يساوي $26,150\ m^3$ و اطوال الانابيب الناقلة لمياه الصرف المُعالجة هي $96,792\ m$ باستخدام انابيب بولي اثيلين عالية الكثافة HDPE .

الحمأة الناتجة من عملية التصفية يتم معالجتها في الهاضمات الهوائية Aerobic Digesters ومن ثم تنقل الى حوض التجفيف الرملي Sand Drying Bed الذي تم تصميمه وتم اختيار موقع مناسب له في جنوب غرب مدينة السليمانية.

حكومة اقليم كردستان - العراق
وزارة التعليم العالي و البحث العلمي
جامعة السليمانية
كلية الهندسة

إختيار أفضل المواقع لمياه المجاري المنزلية البلدية اللامركزية في مدينة السليمانية بإستخدام GIS وAHP مع معالجات فعالة محتملة و قابلية إعادة الإستعمال

اطروحـــة
مقدمة الى مجلس كلية الهندسة في جامعة السليمانية وهي جزء من متطلبات نيل شهادة الدكتوراه في فلسفة الهندسة البيئية

من قبل
زيرين جمال عفور أحمد

باشراف

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استاذ مساعد

أذار 2020 ميلادية 12 ره شه مه 2719 كوردي رجب 1441 هجرية

هه ڵبژاردنی باشترینی جێگاكان بۆ ئاوه رۆی ماڵانی ناوشاری سلێمانی نا نا ناوندی به بەكارهێناني GIS وAHP به چاره سه ركردنیێكی كاریگه رییانه گونجاو و دووباره به كارهێانەوەی

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پوختـــــــــــــــــــه

کەمبوونی ئاو یەکێکە لە کێشە سەرەکیەکان لە شاری سلێمانی کە بێگومان بەهۆی زیاد بوونی ڕێژەی دانیشتوانەکەی ، گۆڕانی کەش وهەوا ، زیادەڕەوی لە بەکارهێنانی ئاو ، وە هەروەها چەند هۆکارێکی ترەوە سەری هەڵداوە. بە مەبەستی چارەسەرکردنی بەشێکی ئەو کێشانە ، یەکە نا سەنترەڵەکان بۆ چارەسەری ئاوەڕۆ (DWWTUs) هەڵبژێردران لەم توێژینەوەیەدا وە کە بەهۆیانەوە ئاوەڕۆ چارەسەرکراوەکە بەکاربهێنرێتەوە بۆ ئاودێری ناوچە سەوزەکان لە ناو شاری سلێمانیدا . بێجگە لەوانەش هەمووی ، چەمی قلیاسان بۆتە قەرارگەی هەموو ئەو دەرچە ئاوەڕۆیانەی کە کۆتا خاڵی کۆکردنەوەی ئاوەڕۆکانی شاری سلێماندا کە ئەمەش بێگومان کارەساتەکە زیاتر ئەکا بە تایبەتی بەهۆی زۆرێک لە تێکچون و کێشە ژینگەییەکان. ئەو یەکە نا سەنترەڵانە لە شیوازی هەوا دوور مەودا ی پاشماوە نیشتووە چالاکەکان بووە .
لێرەوە ئامانجە سەرەکیەکەی ئەم توێژینەوەی دکتۆرایە ئەوەیە کە باشترین جێگە هەڵبژێرێت کە گونجاوتریان بێت لە ڕوی شوێنەکانیانەوە . لە سەرەتادا 134 جێگە سەرەتاییەکان دەستنیشانکران لە شوێنی جیاجیا و بە پانتایی شارەکە . لە ڕێی مۆدێلێکەوە وەبەکارهێنانی پرۆگرامی ArcGIS و تە کنیکی (Analytical Hierchey Process AHP) . لە بەکارهێنانی مۆدێلەدا،کاریگەری پێنج فاکتەر وەرگیراون و کەپێکهاتبوون لە (1) ڕووبەری پێویست بۆ یەکەکان ، (2) دووری جێگەی یەکەکان لە باخچەو ڕووبەرە سەوزەکانەوە، (3) ژمارەی دانیشتوان ، (4) لێژی زەوی شوێنی یەکەکان ، (5) قوڵاي ئەو بۆری ئاوەڕۆیانەی کە دەبنە سەرچاوەی یەکەکانی چارەسەرکرنی ئاوەڕۆکان . لەم توێژینەوەدا بۆ بەکارهێنانی ئەو مۆدێلە ، دوومەرجی سەرەکی دانراون (یەکەم) دەبێت ماڵەکانی دەوري شوێني یەکەکان بەلایەنی کەمەوە 30 متر دوربن ، (دووەم) دووری بۆری ئاوەڕۆکان لە یەکەکانەوە لە 50 متر زیاتر نەبێت . لەئەنجامی مۆدێلە کەوە کە شەش جۆر لە ئاست گونجاویی (Suitability Classificationsشوێنی یەکەکان بەکار هێنا ، توانرا (134) جێی گونجاو دیاری بکرێت و کە لەوانە 10% ئاستیان نە گونجاوە ن بۆ شوێنی یەکەی پاڵاوتنی ئاوی ئاوەڕۆکان و 2% ئاستی مامناوەندیە ، 14% گونجاوەو 30% زۆر گونجاون ، 37 % پلەی گونجاویەکەی زۆر باشە و 7 % پلەی گونجاویەکەی نایابە. بۆ ئەو 134 شوێنی یەکانە ژمارەی Normalized Weighted Average Value (NWAV) دۆزراوەتەوە و ئەو زەویانەی کە NWAV ی

له 0.5 زیاتره وەرگیرا و له ئه نجامي كۆتاییدا 31 شوێن گونجاو بۆ دروستكردنی یەكە ناسەنترەڵاكانی چارەسەری ئاوەڕۆ دیاری كران.

ئامانجي دووهەمینی ئەم توێژینەوەیە، دۆزینەوەی مۆدێلێكی بیركاریانه بۆ كەمترین بوددجه (F_{min}) كه بریتیه له بەدجەی دامەزراندن و دروستكردن و ئیش پێكردنی یەكەكانه (O&M) و بەددجەی گەیاندنی ئەو ئاوەی كه پاك ئەكرێتەوه بۆ باخچەكان (بۆری + دروستكردن و دامەزراندن پەمپەكان).ژمارەی باخچەكانی ناو شاري سلێمانی 827 باخچەو شوێنی سەوزاییه وه ڕوبەری گشتیان بریتیه 4.74 Km2. هەروەها مۆدێلێكی گواستنەوەش (Transportation Matrix Model) دروستكرا كه پێكهاتبوو له [31 x 827] بۆ دۆزینەوەی كەمترین نرخی گەیاندن و گواستنەوەی ئاوی ئاودێری باخچەكان . پرۆگرامی ArcGISبەكارهێنرا له دۆزینەوەی كورترین ڕێگا (Least Cost Rout) له یەكەكانەوه بۆ باخچەكان به بەكارهێنانی مۆدێلی (Network Analysis – OD Matrix) . ئاستی زەوی یەكەكان و باخچەكان له نەخشەی DTM له پرۆگرامی ArcGIS . بۆ چارەسەركردنی مودێلەكه و دۆزینەوەی باشترین Fmin (Optimum) ڕێگەیەكی تەكنیكی in a Matrix (Genetic Algorithm Form) بەكارهێنرا وله پرۆگرامی Matlab 2018a Coding دا دۆززرایەوه . ژمارەیەك ئەنجام بەدەستهات (Random Solutions) كەپەیوەندی هەیه به ڕێژەی ئاوەڕۆكەوە(Wastewater Flow Quantity) و ژمارەی كۆمەڵگەی نیشتەجێیەكانەوه (Population Np) كه بەكار هێنرا له تەكنیكی Algorithm Genetic دا .نرخی Np بەكارهاتووەكان بریتی بوون له (100, 200, 300, 400, 500, 600, 700 800,900,1000) كه هەر ئەنجامێك به جین (Gene) ناونراوه وه بۆ هەر Np یەك سێ جار بەرنامەكه ئیش پێكرا (Run =3) وه چوار جار دوباره كراوەتەوه (Iteration It = 4) . لە Np=500 ئەنجامي نرخەكان دامەزرابون و پرۆسەی (Crossovering – PCO – Point of Crossovering Position) ئەنجام درا بۆ مۆدێلەكه و له خاڵی PCO =632 دا و NP = 500 باشترین نرخي F_{min} (Optimum) مان دەست كەوت . لەو ئەنجامەوه (Optimum) F_{min} گەورەیی 31 یەكەكان دەرهێندران و له ڕێژەی m^3/day (150 – 2,100) دا بوون و كۆی ئەو ئاوەڕۆیەی كه ڕۆژانه ئەپاڵێورێن ئەكاته 26,150 m^3 . درێژی ئەو بۆریانەی كەبەكار هێنراون بۆ گواستنەوەی ئاوی باخچەكان ئەكاته 96,792 m جۆری HDPE .

پاشماوەی ئەو پیسیەی كه ئه نیشێت له حەوزەكانی یەكەكاندا له پرۆسەی پاڵاوتنەكەدا ڕەوانەی (Aerobic Digester) دەكرێت و دواتر ئەگوێزرێتەوه بۆ حەوزی لمی ووشكردنەوەكان Sand Drying Beds كه دیزاینی بۆ كراون و شويني گونجاویان بۆ دیاری كراون له ڕۆژئاوای باكوری شاری سلێمانیدا .

حکومه تی هه رێمی کوردستان – عیراق
وه زاره تی خوێندنی باڵا وتوێژنه وه ی زانستی
زانکۆي سلێمانی
کۆلیجی ئەندازیاری

هه ڵبژاردنی باشترینی جێگاکان بۆ ئاوه رۆی ماڵانی ناوشاری سلێمانی نا نا ناوندی به بەکارهێناني GIS وAHP به چاره سه رکردنییێکی کاریگه رییانه گونجاو و دووباره به کارهێانەوەی

نامەیەکە
پێشکەشکراوه به ئه نجومه نی کۆلیجی ئەندازیاری / زانکۆی سلێمانی
وەك بەشێك له پێداویستیەکانی به دەستهێنانی پلەی دکتۆرای
فەلسەفە له ئەندازیاری ژینگه

لەلایەن
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پروفیسوری یاریده ده ر

ئازار 2020 زاینی 12 ره شه مه 2719 کوردی رجب 1441 کۆچی