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Evaluating the Effecting Factors on the Sizing Design of Municipal Wastewater Treatment Plants

**A Thesis
Submitted to the College of Engineering
of the University of Babylon in Partial
Fulfillment of the Requirements
for The Degree of Master of
Science in Environmental
Engineering**

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وزارة التعليم العالي والبحث العلمي
جامعة بابل
كلية الهندسة
قسم الهندسة المدنية

تقييم العوامل المؤثرة على تصميم محطات معالجة مياه الفضلات البلدية

رسالة
مقدمة إلى كلية الهندسة في جامعة بابل
كجزء من متطلبات نيل درجة الماجستير في علوم
الهندسة البيئية

من قبل

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ABSTRACT

This study presents a computer program designed in Visual Basic software 6.0. This program is flexible and easy to be used by practiced engineer for designing different types of wastewater treatment plants which are complete mixing activated sludge, extended aeration, oxidation ditches, aerated lagoons, aerobic ponds, facultative ponds, anaerobic ponds, trickling filters, rotating biological contactor, and bio-towers. Also, the program deals with the different environmental factors that affect the design of municipal wastewater treatment steps.

The results of the program are verified with the results obtained from hand calculations procedures for the most important factor which is population. This verification shows a good agreement with average correlation coefficients about 0.999.

The relationships between independent and dependent variables are found by multiple non - linear regression analysis is employed by using statistical program "Data Fit version 8.0". Independent variables include population, temperature, sewage contribution per person, BOD and TSS concentrations, area served by network, infiltration rate, design period, recirculation ratio, and dispersion factor. While dependent variables are considered as (volume of treatment units, volume of air required, and quantity of gas generated from digestion process).

The population is found to be the most significant variable affecting design of all wastewater treatment units for all types of present wastewater treatment plants.

When the sewage contribution per capita is increased by (29.65%) that leads to increase volume of finall settling tank for conventional activated sludge about (20.94%), oxidation ditch about (30.97), extended

aeration about (29.87%), aerated lagoon about (12.77%), and (attached growth system) about (26.55%).

When temperature increasing by (25%) that leads to increase volume of biological treatment unit for conventional activated sludge about (17.86%), extended aeration about (15.69%), trickling filter about (20.79%), and rotating biological contactors about (17.94%).

When BOD concentration increasing by (28.57%) that leads to increase volume of total air required for conventional activated sludge about (26.17%), extended aeration about (25.61%), oxidation ditch about (28.57%), and aerated lagoon about (29.35%).

When TSS concentration increasing by (22.22%) that leads to increase volume of thickener, digester, and quantity of gas produced for conventional activated sludge about (19.17%, 4.12%, and 11.39%), stabilization pond about (22.22%, 15.84%, and 12.4%), and attached growth system about (15.86%, 2.15%, and 10.19%) respectively.

Recirculation ratio is found to be a significant variable on the models of volume of biological units for trickling filters and bio-towers.

Dispersion factor is found to be a significant variable on the models of volume of biological units for facultative ponds only.

Finally, the infiltration rate increases the plant influent flow and decrease the concentration of BOD in the sewage because the infiltration caused by the high water table and defects in the network pipes. The area served by network and design period have a significance effects on design requirements.

الخلاصة

تُقدم هذه الدراسة برنامج حاسوبي مصمم بـ 6.0 بلغة فيجوال بيسك. إن البرنامج مرّن وسهل الاستخدام من قبل المهندس المتمرس وكذلك دقيق في النتائج لتصميم أنواع مختلفة من محطات معالجة مياه الفضلات وهي: الحمأة المُشَطَّة، التهوية المستمرة، حفر الأكسدة، البحيرات المهواة، البرك المهواة، البرك اللامهواة، البرك المخيرة، المرشحات بالتنقيط، الملامسات البايولوجية الدوارة، والأبراج الحيوية. كذلك يهدف البرنامج إلى تحليل العوامل البيئية المختلفة المؤثرة على تصميم خطوات معالجة مياه الفضلات البلدية.

تم اختبار صلاحية النماذج المستخرجة بإجراء مقارنة بين النتائج المستحصلة من البرنامج مع النتائج الناتجة من الحسابات اليدوية لأكثر عامل أهمية وتأثيراً وهو عدد السكان باستعمال معادلة التحديد والتي أعطت نتائج جيدة.

لإيجاد العلاقات الإحصائية بين عدد من المتغيرات المستقلة والمتغيرات المعتمدة تم استخدام طريقة التحليل الانحدار اللاخطي المتعد باستعمال برنامج الإحصائي Data Fit "8.0". وتشمل المتغيرات المستقلة كل من (عدد السكان المنطقة المخدومة، درجة حرارة الفضلات، مساهمة الشخص لمياه الفضلات الواحد، تركيز كل من BOD TSS and، ومساحة المنطقة المخدومة بالشبكة المجاري، نسبة التسرب، العمر التصميمي، نسبة إعادة التدوير، وعامل الانتشار) بينما المتغيرات المعتمدة التي أخذت بعين الاعتبار هي (حجم وحدات المعالجة، حجم الهواء المطلوب وكمية الغاز المتولدة من عملية الهضم).

أظهرت الدراسة أن أكثر العوامل أهميه وتأثيراً على تصميم وحدات المعالجة لكل الأنواع المدروسة من المحطات هو عدد السكان.

وعند زيادة مساهمة الشخص لمياه الفضلات بمقدار (29.65%) فإن هذا يؤدي إلى زيادة حجم المروق الثانوي للحمأة المُشَطَّة بمقدار (20.94%)، لحفر الأكسدة بمقدار (30.97%)، للتهوية المستمرة بمقدار (29.87%)، للبحيرات المهواة بمقدار (12.77%)، وللنمو العالق بمقدار (26.55%).

عند زيادة درجة الحرارة بمقدار (25%) فإن هذا يؤدي إلى زيادة حجم وحدة المعالجة البايولوجية للحمأة المُشَطَّة بمقدار (17.86%)، للتهوية المستمرة بمقدار (15.69%)، للمرشحات بالتنقيط بمقدار (20.79%)، ولللامسات البايولوجية الدوارة بمقدار (17.94%).

عند زيادة تركيز طلب الأوكسجين البايولوجي بمقدار (28.57%) فإن هذا يؤدي إلى زيادة كمية الهواء اللازمة لتهوية الحمأة المُشَطَّة بمقدار (26.17%)، لتهويه المستمرة بمقدار (25.61%)، لبحيرات الأكسدة بمقدار (28.57%)، وللبحيرات المهواة بمقدار (29.35%).

وعند زيادة تركيز المواد الصلبة الكلية بمقدار (22.22%) فإن هذا يؤدي إلى زيادة حجم المثخنات، هاضمات الحمأة، وكمية الغاز المنبعثة للحمأة المُشَطَّة بمقدار (19.17%)، (4.12%)، (11.39%)، وللبرك بمقدار (22.22%)، (15.84%)، (12.4%)، وللنمو الملتصق بمقدار (15.86%)، (2.15%)، (10.19%) على التوالي.

و كانت نسبة إعادة توزيع مياه الفضلات من المتغيرات المؤثرة على وحدات المعالجة البايولوجية لكل من المرشحات بالتنقيط والأبراج الحيوية.

و كان عامل الانتشار من المتغيرات المؤثرة على وحدات المعالجة البايولوجية للبرك الاختيارية.

وأخيراً، تسبب كمية مياه الرش زيادة في كمية التصريف الواصل للمحطة وبالتالي تسبب في تخفيف المحتوى العضوي للفضلات وجد أن لهذا العامل تأثير هام على عملية التصميم، ينتج الرش عن انتقال المياه في الأماكن ذات المستوى العالي للمياه إلى داخل أنابيب

شبكة المجاري. إن المساحة المخدمة والعمر التصميمي لهما تأثير واضح على متطلبات التصميم.

Chapter one

Introduction

1.1 General

Wastewater treatment is an essential public service. Wastewater treatment plants are large energy users with excellent conservation potential. The quantity of wastewater produced varies in different communities and countries, depending on a number of factors such as water uses, climate, lifestyle, economics, etc. Municipal wastewater is derived largely from the water supply. Some water uses, such as for street washing, lawn watering, fire fighting, and leakage from water supply pipelines, do not reach the sewers. The volume of wastewater is added to by infiltration and inflows.

The principal objective of wastewater treatment is generally to allow human and industrial effluents to be disposed of without danger to human health or unacceptable damage to the natural environment.

Wastewater treatment facilities are designed to speed up the natural purification process that occurs in natural waters and to remove contaminants in wastewater that might otherwise interfere with the natural process in the receiving waters.

The biological treatment unit is considered to be the most important unit in the wastewater treatment plant, and because of its importance, the wastewater treatment plants were named after the biological treatment method employed.

The selection of a wastewater treatment system to provide a desired waste reduction requires the technical, economic, and environmental evaluation of all available alternatives. It therefore becomes increasingly

important that the most economical, less environmental impact, and efficient process must be selected for each specific problem.

Wastewater treatment plant is a complex dynamic process influenced by many uncertain factors which are been complicate and extensive such as loading and biomass composition, population, organic loads, temperature, area served by network, pH, and others ⁽¹⁾.

1.2 Research Objectives

The aims of this study are:

1. Preparing computer program in (Visual Basic V 6.0 software) to design different types of wastewater treatment plants which are (conventional activated sludge, extend aeration, oxidation ditch, aerated lagoon, aerobic pond, anaerobic pond, facultative pond, trickling filter, rotating biological contractors, and bio-tower) wastewater treatment plant, that contains preliminary, primary, secondary and sludge treatments processing in a minimum time.
2. Discussing and evaluating different factors that affect the design of different steps of the treatment process.
3. Developing regression models to govern the relationships between the design requirements (dependent variables) which are: (volume of each treatment unit, quantity of total air required for aerobic reactors, and volume of gas production) and different factors that affecting the design (independent variables) which are: (population, specific sewage production, temperature, organic load and solid loadings, infiltration rate,

design period, dispersion, recirculation ratio, and catchment's area served by network).

1.3 The Thesis Layout

This thesis consists of five chapters:

- ❖ Chapter One presents an introduction and research objectives.
- ❖ Chapter Two presents a literatures review of both early and recent studies on design of wastewater treatment plants along with factors affecting on the design, theory of wastewater treatment plant.
- ❖ Chapter Three presents the details of the computer program for designing different types of treatment plant by using Visual Basic software 6.0 and the verification of computer program.
- ❖ Chapter Four presents the results for each statistical models under considerations reported.
- ❖ Chapter Five presents the main conclusions obtained from this study and recommendations for future studies.

Chapter two

Theoretical Background And Literatures Review

2.1 Introduction

"Wastewater," also known as "sewage," originates from household wastes, human and animal wastes, industrial wastewaters, storm runoff, and groundwater infiltration. Wastewater, basically, is the flow of used water from a community. It is 99.94 percent water by weight ⁽²⁾. The remaining 0.06 percent is material dissolved or suspended in the water. It is largely the water supply of a community after it has been fouled by various uses.

Needing for wastewater treatment plants will increase in the future due to population growth, increasingly restrictive environmental regulations, and demand for wastewater reuse. In addition, depending on the structure of the surrounding society the composition and amount of the wastewater varies. Thus, the layout and size of the wastewater treatment plants must vary from location to location ⁽³⁾.

2.2 Characteristics of Wastewater

An understanding of physical, chemical, and biological characteristics of wastewater is very important in design, operation, management of collection, treatment, and disposal of wastewater. The nature of wastewater includes physical, chemical, and biological characteristics which depend on the water usage in the community, the industrial and commercial contributions, weather, and infiltration/inflow ⁽⁴⁾. Both constituents and concentrations vary with time and local conditions. Table (2.1) shows typical concentration ranges for various constituents in untreated domestic wastewater.

Wastewater is classified as strong, medium or weak, depending on its contaminant concentration ⁽⁵⁾.

Table (2-1): Typical Composition of Untreated Domestic Wastewater (5)

Contaminants	Unit	Concentration		
		Weak	Medium	Strong
Total solids (TS)	mg/L	350	720	1200
Total dissolved solids (TDS)	mg/L	250	500	850
-Fixed	mg/L	145	300	525
-Volatile	mg/L	105	200	325
Suspended solids	mg/L	100	220	350
-Fixed	mg/L	20	55	75
-Volatile	mg/L	80	165	275
Settleable solids	mL/L	5	10	20
BOD ₅ , 20°C	mg/L	110	220	400
TOC	mg/L	80	160	290
COD	mg/L	250	500	1000
Nitrogen (total as N)	mg/L	20	40	85
Organic	mg/L	8	15	35
Free ammonia	mg/L	12	25	50
Nitrites	mg/L	0	0	0
Nitrates	mg/L	0	0	0
Phosphorus (total as P)	mg/L	4	8	15
Organic	mg/L	1	3	5
Inorganic	mg/L	3	5	10
Chlorides	mg/L	30	50	100
Sulfate	mg/L	20	30	50
Alkalinity (as CaCO ₃)	mg/L	50	100	200
Grease	mg/L	50	100	150

2.3 Design Flow Rates

Variations in wastewater flow rate are experienced according to the time of the day, the day of the week and season of the year. Quantifying these variations is important for the design and the operation of a treatment plant. Using maximum hour, day, month and other periods, peaking factors can be developed. Peaking factors are useful in making estimations for the maximum hydraulic conditions that could be experienced.

In previous studies, there are many values for peak factor, Metcalf and Eddy ⁽⁵⁾ stated that value of maximum flow to average flow rate between 1.3 for large cities and 2 for small cities, AL-Layla ⁽⁶⁾ stated the maximum hourly flow rate vary between 1.5 and 3 times the average

hourly flow rate. The higher value corresponds to small communities and the lower value to large ones. Also Steel ⁽⁷⁾ declared that percent of maximum flow rate about 2.25 of value of average flow rate, and Hammer ⁽⁸⁾ stated maximum flow rate is about 2.5 of average design flow rate. The average design flow rate can be determined as:

$$Q_{avg} = P_f \times \mu_o \quad \dots (2.1)$$

Where;

Q_{avg} . = average design flow, m³/d

P_f = future population, capita

μ_o = specific sewage production, l/c/d

The peaking factor can be estimated from equations (2.2, 2.3 and 2.4) ⁽⁵⁾.

$$PF = 1 + \frac{14}{4 + \sqrt{P_t}} \quad \dots (2.2)$$

And;

$$PF = \frac{4.0}{P_t^{1/6}} \cdot n \quad \dots (2.3)$$

where;

n = factor depends on climate change, in normal conditions n values are (1.2 – 1.4),

P_t = population in thousands.

Therefore Eq. (2.3) can arrange to:

$$PF = \frac{4.8}{P_t^{1/6}} \quad \dots (2.4)$$

PF = the ratios of peak flow to average flow

2.4 Wastewater Treatment Plant Layout

Wastewater treatment facilities are designed to speed up the natural purification process that occurs in natural waters and to remove contaminants in wastewater that might otherwise interfere with the natural process in the receiving waters.

Conventional wastewater treatment systems are combinations of physical and biological (sometimes with chemical) processes to remove its impurities.

A Conventional wastewater treatment plant is accomplished by general steps which are:

- 1- Preliminary treatment.
- 2- Primary treatment
- 3- Secondary treatment,
- 4- Tertiary (advanced) treatment (if it is necessary) , and
- 5- Sludge treatment processes.

2.4.1 Preliminary Treatment

is defined as any physical or chemical process at the wastewater treatment plant that precedes primary treatment. Its function is mainly to protect subsequent treatment units and to minimize operational problems. Pretreatment at the source to render a wastewater acceptable at the domestic wastewater treatment facility is not included ⁽⁹⁾.

A-Bar screens

Screening is usually the first unit operation at wastewater treatment plants. The screening units include racks, coarse screens, and fine screens. The racks and screens used in preliminary treatment are to remove large objects such as rags, plastics, paper, metals, dead animals, and the like. The purpose is to protect pumps and to prevent solids from fouling subsequent treatment facilities. To select the proper screening

size, the sewage flow is divided by the efficiency factor obtained from Table (2.2) will give the wet area required for the screen channel.

Table (2.2): Efficiencies of Bar Spacing for Screen Channel (10)

Bar Size, mm	Openings, c m	Efficiency
6.5	2.54	0.8
8.0	2.54	0.768
9.5	2.54	0.728
11.0	2.54	0.696
12.0	2.54	0.667

The procedure for selecting the proper screening size is as follows ⁽¹⁰⁾:

$$\text{Cross area or channel cross-section wet area} = \frac{\text{Net area, m}^2}{\text{Efficiency coefficient for bars}} \dots(2.5)$$

$$\text{Corresponding depth or width} = \frac{\text{Channel cross-section wet area}}{\text{Maximum desired width or depth}} \dots(2.6)$$

The head loss through the bar rack is calculated from Eqs. (2.7), (2.9), or (2.10). Eq. (2.7) is used to calculate head loss through clean or partly clogged bars, while Eq.(2.8) is used to calculate head loss through clean screen only. Eq.(2.9) is the common orifice formula and is also used to calculate head loss through fine screen ⁽⁵⁾.

$$h_L = \left(\frac{1}{0.7} \right) \frac{V_R^2 - v^2}{2g} \dots (2.7)$$

$$h_L = \beta \left(\frac{w}{b} \right)^{4/3} \frac{v^2}{2g} \sin \theta \dots (2.8)$$

$$h_L = \frac{1}{2} \left(\frac{Q}{cA_e} \right)^2 \dots (2.9)$$

Where;

h_L = head loss, m.

V_R = approach velocity in rack chamber, m/s.

v = approach velocity in upstream channel, m/s.

g = acceleration due to gravity, 9.81 m/s^2 .

w = maximum width of the bar facing the flow, m.

b = minimum clear spacing of bars, m.

θ = angle of the rack to the horizontal.

β = bar shape factor, as follows in table (2.3).

Q = discharge through screen, m^3/s .

c = coefficient of discharge.

A_e = effective submerged open area, m^2 .

Table (2.3): Bar Shape Factor (11)

Bar type	β
Sharpe – edged rectangular	2.42
Rectangular with semicircular face	1.83
Circular	1.79
Rectangular with semicircular upstream and downstream faces	1.67
Tear shape	0.76

B- Grit chambers

Grit is defined as sand, gravel, or other mineral matter having the nominal diameter of 0.15- 0.20 mm or larger. Typically, grit chambers are designed to remove all particles with nominal diameter of 0.20 mm or larger; i.e. solid materials having subsiding velocities or specific gravities significantly greater than those of organic putrescible solids in wastewater. The types of grit chambers are three: horizontal or rectangular horizontal flow, aerated, and vortex ⁽¹²⁾.

Grit chambers should be provided for all wastewater treatment plants, and used on systems required for plants receiving sewage from combined sewers or from sewer systems receiving a substantial amount

of ground garbage or grit. Grit chambers are usually installed ahead of pumps and comminuting devices.

In the present study aerated grit chamber will be selected. In recent years, the aerated grit chamber has been more widely used for selective removal of grits at medium and large size treatment plants ⁽¹¹⁾.

It is important that the designer must give consideration to those factors of the design and performance of the process. Many of these factors include type of grit, detention time, air supply, tank geometry, and baffle arrangement. Some design factors are given below in table (2.4).

$$V_g = Q_p \cdot t \quad \dots (2.10)$$

Where;

V_g = volume of grit chamber, m³.

Q_p = peak design flow, m³/d.

t = detention time, min.

Table (2.4): Design Factors and Typical Design Values for Aerated Grit Chamber (11).

Design Factor	Range of Value
Depth, m	2 - 5
Length, m	7.5 - 20
Width, m	2.5 - 7.0
Width/depth ratio	1:1 - 5:1
Length/width ratio	2.5:1 - 5:1
Detention time, min	2 - 5
Transverse velocity at surface, m/s	0.6-0.8
Air supply, l/s.m of tank length	4.6 - 12.4

2.4.2 Primary treatment

After the wastewater passes the preliminary processes, it enters sedimentation tanks. The suspended solids that are too light to fall out in the grit chamber will settle in the tank over a few hours.

Sedimentation tanks are designed to operate continuously. They are usually rectangular or circular and have hoppers for sludge collection. Most sedimentation tanks are constructed with gently sloped bottoms and have sludge hoppers with relatively steep sides. Non—mechanized settling tanks are used only in very small installations; the sludge moves to hoppers by gravity, where it is removed ⁽⁹⁾.

The design overflow rates for various types of clarifiers facilities are given in table (2.5). Primary sedimentation facilities are generally designed for an overflow rate of 35 m³.m².d at average design flow, and detention time usually 1-3 h (typically 2 h). Surface overflow rates (SOR) are calculates as:

$$\text{SOR} = \frac{Q_{\text{avg}}}{A} \quad \dots (2.11)$$

Where;

SOR = surface overflow rate, m³/m².d.

Q_{avg} = average design flow, m³/d.

A = surface area of tank, m².

Table (2.5): Surface Loading Rates for Primary Settling Tank (11)

Condition	Surface Loading Rate m ³ /m ² .d	
	Range	Typical
Primary clarification prior to secondary treatment		
Average flow	30-50	40
Peak flow	70-130	100
Primary clarification with waste activated sludge return		
Average flow	25-35	30
Peak flow	45-80	60

2.4.3 Secondary treatment

Biological treatment systems are designed to maintain a large active mass and a variety of microorganisms, principally bacteria (and fungi, protozoa, rotifers, algae, etc.), within the confined system under favorable environmental conditions, such as dissolved oxygen, nutrient, etc.

The biological treatment process depends on one of the two concepts:

- Suspended–growth process:

In which the microorganisms are maintained in suspension within the liquid during the biological process, the main types of this process are activated sludge and oxidation ponds.

- Attached – growth process:

In which the microorganisms are attached to some inert medium such as rocks, slag, or plastic and also know as fixed – film process. The main types of this process are trickling filter and rotating biological contactor ⁽¹⁾.

A- Suspended–growth process

1- Activated Sludge Process:

Activated Sludge process is very flexible and can be adapted to almost any type of biological wastewater treatment problem.

The activated sludge process is one of the most widespread biological wastewater purification technologies. In this process, wastewater is mixed with a concentrated bacterial biomass suspension (the activated sludge) which degrades the pollutants. Originally, the concern was mainly to remove the organic carbon substances from the wastewater, which could be obtained rather easily by simple process designs ⁽¹³⁾.

The empirical design of activated sludge is based on BOD loading, food-to-microorganism ratio (F/M), sludge age, and aeration period. Empirical design concepts are still acceptable.

The organic (BOD₅) loading on an aeration basin can be computed using the BOD in the influent wastewater without including the return sludge flow. BOD loadings are expressed in terms of kg BOD applied per day per 1000 m³(kg/(d • m³)) of liquid volume in the aeration basin.

The $\frac{F}{M}$ ratio is used to express BOD loadings with regard to the microbial mass in the process. The value of the $\frac{F}{M}$ ratio can be calculated by the following equation:

$$\frac{F}{M} = \frac{Q * BOD}{V * MLSS} \quad \dots(2.12)$$

Where

$\frac{F}{M}$ = food-to-microorganism ratio, kg of BOD per day per kg of MLSS

Q= wastewater flow, m³/d.

BOD = wastewater 5-day BOD, mg/L.

V = liquid volume of aeration tank, m³.

MLSS = mixed liquor suspended solids in the aeration tank, mg/l.

The aeration period is the retention time of the influent wastewater flow in the aeration basin and is expressed in hours. It is computed from the basin volume divided by the average daily flow excluding the return sludge flow.

For a complete-mix system, the mean hydraulic retention time (HRT) θ for the aeration tank is

$$\theta = V/Q \quad \dots(2.13)$$

Where;

θ = hydraulic retention time, day

V = reactor volume, m^3

Q = wastewater influent flow rate, m^3/d

Fig.(2.1) shows a schematic diagram of complete-mix activated sludge reactor with sludge wasting from the aeration tank; and the other method sludge wasting from return sludge line.

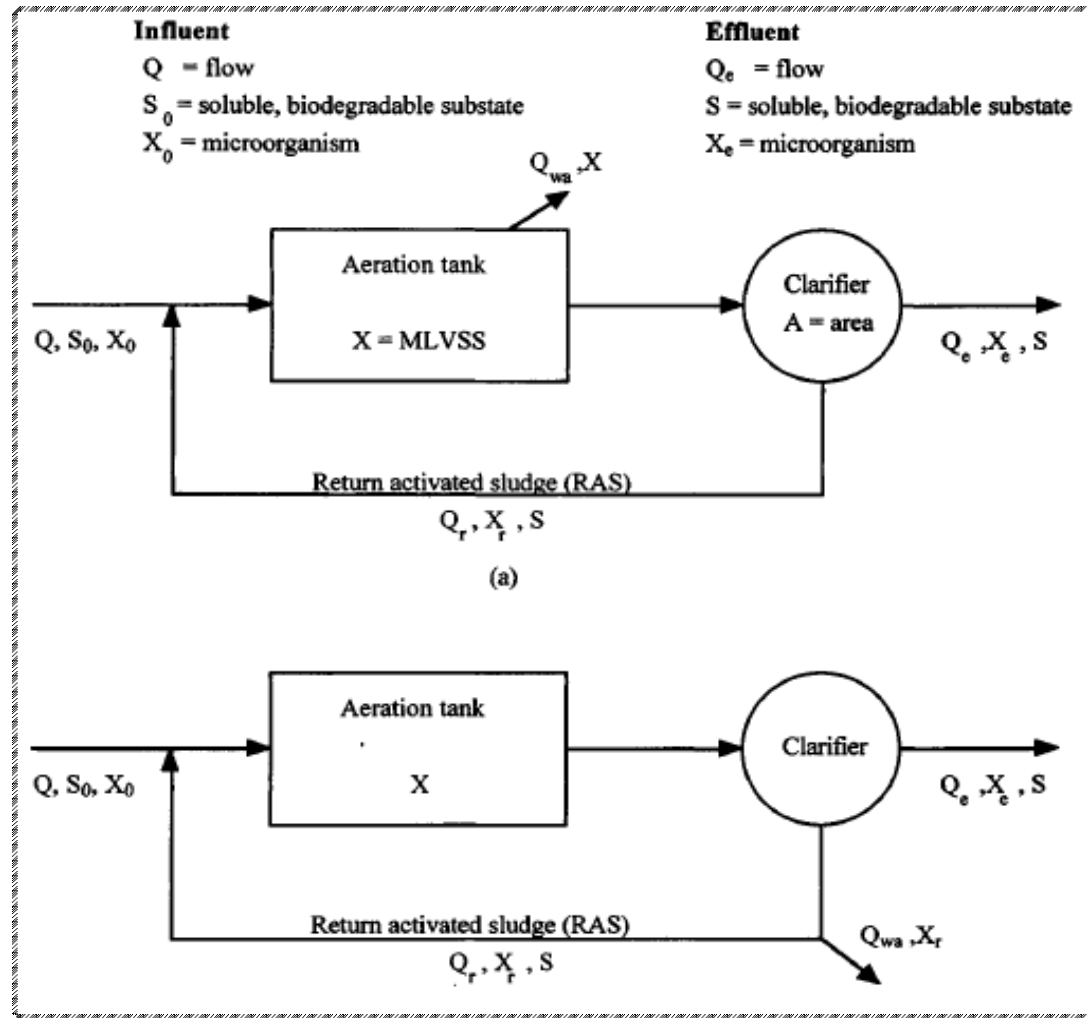


Fig. (2.1): Schematic Chart of Complete - Mix Activated Sludge Reactor: (a) Sludge Wasting from the Aeration Tank; (b) Sludge Wasting from Return Sludge Line (4).

Referring to Fig. (2.1a), the mean cell residence time θ_c (or sludge age or SRT) in the system is defined as the mass of organisms in the aeration tank divided by the mass of organisms removed from the system per day, and is expressed as:

$$\theta_c = \frac{X}{(\Delta X / \Delta t)} \quad \dots (2.14)$$

$$\theta_c = \frac{VX}{Q_{wa}X + Q_e X_e}$$

Where;

θ_c = mean cell residence time based on solids in the tank, d.

X = concentration of MLVSS maintained in the tank, mg/L.

$(\Delta X / \Delta t)$ = growth of biological sludge over time period, mg/L.d.

Q_{wa} = flow of waste sludge removed from the aeration tank, m³/d.

Q_e = flow of treated effluent, m³/d.

X_e = microorganism concentration (VSS) in effluent, mg/L.

For system-drawn waste sludge from the return sludge line (Fig. 2.1b), the mean cell residence time would be

$$\theta_c = \frac{VX}{Q_{wa}X_r + Q_e X_e} \quad \dots (2.15)$$

Where;

Q_{wa} = flow of return sludge from sludge line, m³/d.

X_r = return concentration in effluent, mg/L.

The mass balance for the microorganisms in the entire activated sludge system is expressed as the rate of accumulation of the microorganisms in the inflow plus net growth, minus that in the outflow. Mathematically, it is expressed as ⁽¹⁴⁾.

$$V \frac{dX}{dt} = Q_{in} X_o + V(r'_g) - (Q_{wa} X + Q_e X_e) \quad \dots (2.16)$$

Where;

$\frac{dX}{dt}$ = rate of change of microorganisms concentration (VSS), mg/L.m³.d.

X_o = microorganisms concentration (VSS) in influent, mg/L.

r'_g = net rate of microorganism growth (VSS), mg/L.d.

Other terms are as in the above equations.

$$r'_g = Yr_{su} - K_d X \quad \dots (2.17)$$

Where;

Y = maximum yield coefficient over finite period of log growth, mg/mg.

r_{su} = substrate utilization rate, mg/m³.d.

K_d = endogenous decay coefficient, day⁻¹.

The mass concentration of microorganisms X in the aeration tank is:

$$X = \frac{QY(S_o - S)\theta_c}{V(1 + K_d\theta_c)} \quad \dots (2.18)$$

Where

$(S_o - S)$ = mass concentration of substrate utilized, mg/L.

S_o = substrate concentration in influent, mg/L.

S = substrate concentration in effluent, mg/L.

θ = hydraulic retention time (Eq. (2.13)), d.

The reactor (aeration tank) volume

$$V = \frac{QY(S_o - S)\theta_c}{X(1 + K_d\theta_c)} \quad \dots (2.19)$$

The substrate concentration in effluent S can also be determined from the substrate mass balance by the following equation

$$S = \frac{K_s(1 + \theta_c K_d)}{\theta_c(Yk - K_d) - 1} \quad \dots (2.20)$$

where

S = effluent substrate (soluble BOD₅) concentration, mg/L.

K_s = half-velocity constant, substrate concentration at one-half the maximum growth rate, mg/L.

k = maximum rate of substrate utilization per unit mass of microorganism, per day.

The amount of sludge generated (increased) per day affects the design of the sludge treatment and disposal facilities. It can be calculated by

$$P_x = Y_{obs} Q(S_0 - S) \quad \dots (2.21)$$

where

P_x = net waste activated sludge (VSS), kg/d.

Q = influent wastewater flow, m³/d.

S_0 = influent soluble BOD₅ concentration, mg/L.

S = effluent soluble BOD₅ concentration, mg/L.

Y_{obs} = observed yield, g/g.

$$Y_{obs} = \frac{Y}{1 + K_d \theta_c} \quad \dots (2.22)$$

The theoretical oxygen requirement in the activated sludge is determined from BOD₅ of the wastewater and the amount of microorganisms wasted per day from the process. Oxygen requirements generally depend on the design peak hourly BOD₅, MLSS, and degree of treatment.

The DO of the waste activated sludge may be estimated as (1.42 P_x). The mass of oxygen required may be estimated as ⁽¹⁴⁾:

$$O_2 = \frac{Q(S_0 - S)}{f} - 1.42 P_x \quad \dots (2.23)$$

Where;

O_2 = theoretical air required, kg/d.

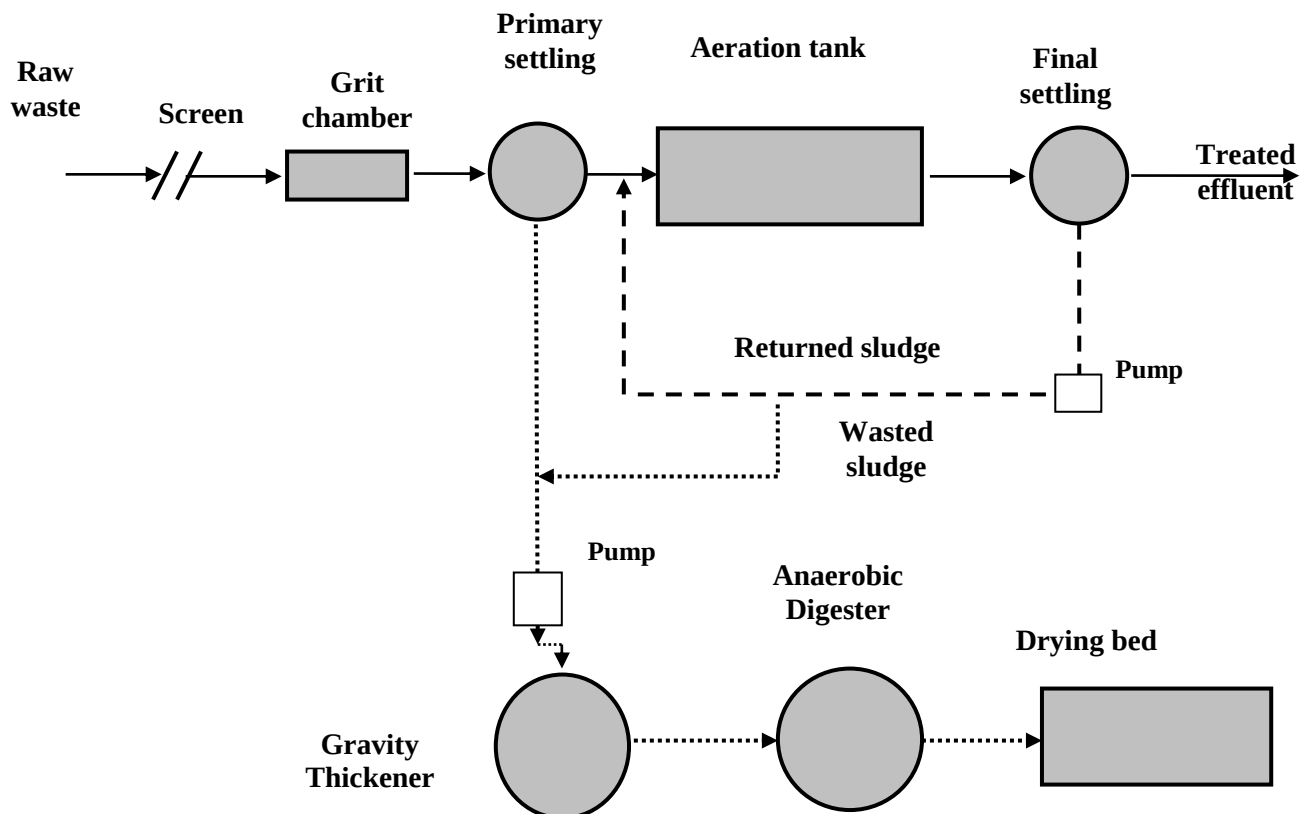
P_x = net mass of cell tissue produced, kg/d.

f = conversion factor = 0.68.

Grady.et.al ⁽¹⁵⁾ declared that low dissolved oxygen concentration could limit the growth of microorganisms and induce the predominance of filamentous bacteria with subsequent deterioration of the effluent quality. On the other hand, high DO concentration represents a high-

energy waste, generates excess turbulence and may break up the biological flock resulting in poor settling characteristics and concentration of solids in the effluent.

Fig.(2.2) shows a schematic chart of Conventional Activated Sludge Wastewater Treatment Plant.



Fig(2.2): A Conventional Wastewater Treatment Plant in the Present Study

2- Extended aeration

The extended aeration process consists of an aeration with a longer detention time than the conventional activated-sludge process and is followed by a settling tank of several hours of detention period ⁽⁶⁾.

Extended aeration is low in capital cost, simple to operate, and produce a well oxidize sludge but this process is applicable only to small treatment plants with capacities of less than 3800 m³/day ⁽¹⁶⁾.

As in the case of activated sludge design discussed earlier, the first step in designing any extended aeration plant is to select a suitable value

of the mean cell residence time θ_c and, consequently, $\frac{F}{M}$. the major factor affecting this choice is sludge stabilization and nitrification ⁽¹⁷⁾.

Most extended aeration system are generally designed as complete mixing reactors.

$$\frac{F}{M} = \frac{K_d f_b}{0.77Y} \quad \dots(2.24)$$

$$\theta_c = \frac{0.77}{K_d f_b} \quad \dots(2.25)$$

Where:

f_b = biodegradable fraction of VSS.

Fig.(2.3) shows a schematic chart of Extended aeration.

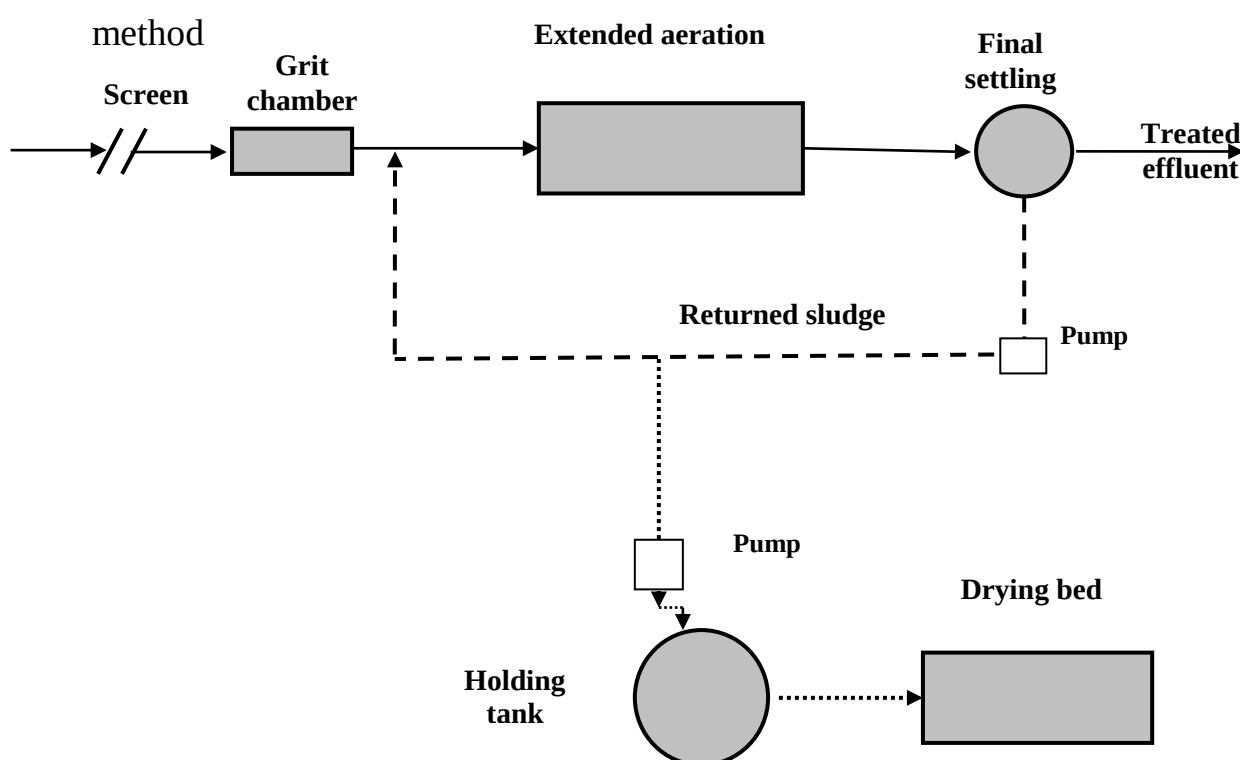


Fig. (2.3): An Extended Aeration Wastewater Treatment Plant in the Present Study.

3- Oxidation ditch

It is an earthen tank of special shape with arrangements for a sufficient supply of oxygen. Raw wastewater is brought into the tank and aerated for an extended period of time. The activated sludge in the

oxidation ditch removes the organic matter and converts it to cell protoplasm. These solids (or all protoplasm) will be degraded if aeration is continued for long periods of time (30-60 days mean cell residence time). The sludge formed in the oxidation ditch is mineralized to such an extent that it can be dried on sand beds without odor problems. This method of treatment achieves a high degree of purification. A rotor (Kessener brush) is installed inside the ditch to aerate the mixed liquor and circulate the contents. The mixed liquor is made to circulate inside the ditch, and as a result it passes the rotor at intervals for aeration ⁽⁶⁾.

The organic load applied to the plant depends on the wastewater flow $Q(\text{m}^3/\text{day})$, BOD_5 of the influent (g/m^3), and the mass of sludge $\text{MLSS} (\text{g}/\text{m}^3)$ in the aeration system.

$$\frac{\text{BOD}_5 \text{ loading}}{\text{Unit volume of tank}} = \frac{\text{QBOD}_5}{V} \quad \dots(2.26)$$

Where V = volume of the oxidation ditch (m^3). If the influence of suspended activated sludge solids is also taken into consideration, then

$$\text{Sludge loading} = \frac{\text{QBOD}_5}{V(\text{MLSS})} \quad \dots(2.27)$$

It is advisable to maintain a constant sludge loading to provide a constant food supply for the activated sludge, which is essential for process to operate in the endogenous phase.

$$\text{Sludge loading rate} = \text{constant} = L_{ds}$$

So,

$$V = \frac{QB}{L_{ds}(\text{MLSS})} \quad \dots(2.28)$$

Fig.(2.4) shows a schematic chart of oxidation ditch method

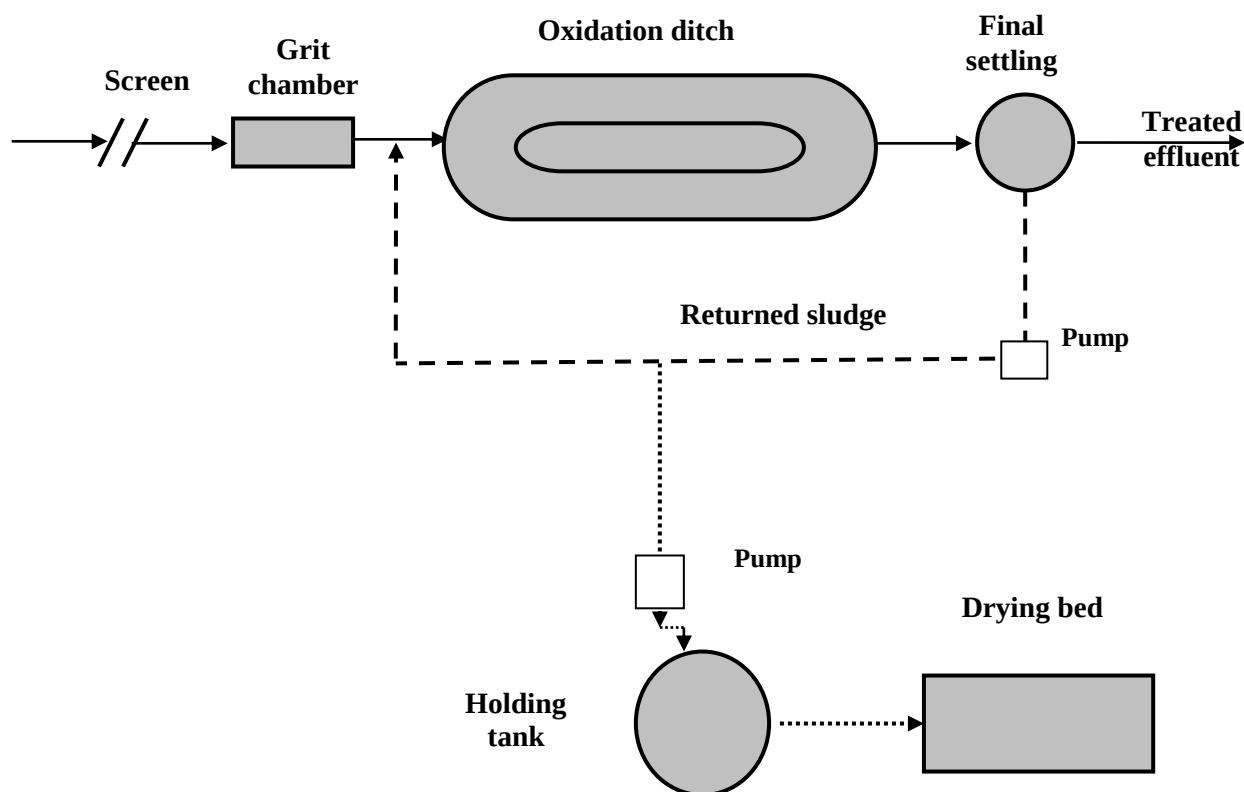


Fig. (2.4): An Oxidation Ditch Wastewater Treatment Plant in the Present Study

4- Aerated lagoon

The aerated lagoons are suspended growth reactors in earthen basins with no sludge recycle. Mechanical aerators are normally used for mixing and supplying oxygen demand. Since the aerated lagoons have a large detention period (2-6 days), a certain amount of nitrification is achieved ⁽¹¹⁾.

Design of aerated lagoon is similar to an activated sludge process with no recycle ⁽¹⁴⁾.

$$\frac{S}{S_0} = \frac{1}{1 + k(V/Q)} \quad \dots(2.29)$$

$$T_w = \frac{AfT_a + QT_i}{Af + Q} \quad \dots(2.30)$$

Where:

S =effluent BOD₅ concentration, mg/l.

S_0 =influent BOD₅ concentration, mg/l.

k = first-order BOD₅ removal-rate constant, d⁻¹.

V =volume, m³.

Q =flow rate, m³/d.

T_i =influent waste temperature, °C.

T_w =lagoon water temperature, °C.

T_a =ambient air temperature, °C.

f =proportionality factor, 0.5.

A =surface area, m².

Fig.(2.5) shows a schematic chart of aerated lagoons method

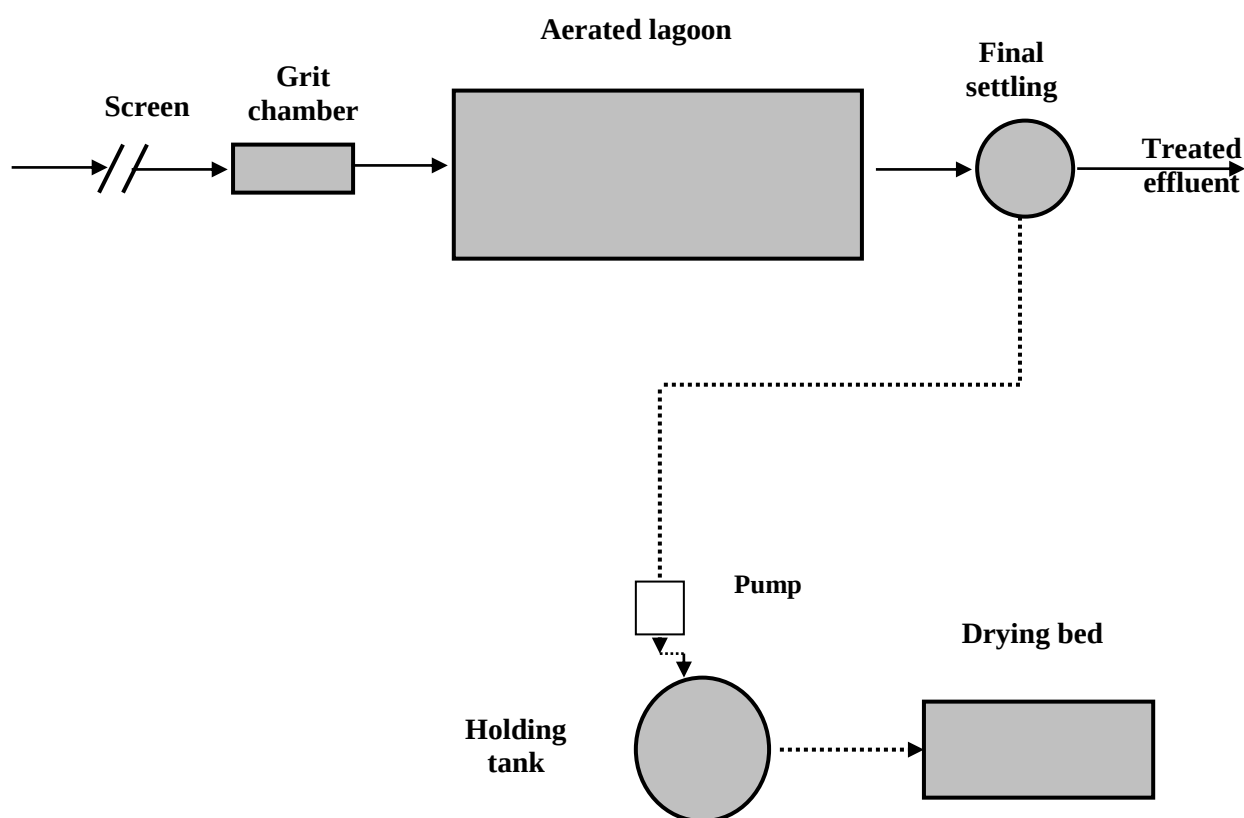


Fig. (2.5): An Aerated Lagoon Wastewater Treatment Plant in the Present Study.

5- Stabilization ponds

A stabilization pond is a relatively shallow body of water contained in an earthen basin of certain shape, designed to treat wastewater. These ponds have become a popular means of wastewater treatment for small communities and industries that produce organic waste streams. The stabilization ponds have the advantage of low construction and operation costs. The major disadvantages are large land area required, odor and insect problem, possible groundwater contamination, and poor effluent quality ⁽¹¹⁾.

The stabilization ponds are usually classified as aerobic, facultative, and anaerobic. This classification is based on the nature of the biological activity taking place.

A- Anaerobic ponds

Anaerobic ponds are commonly 2 – 5 m deep and receive wastewater with high organic loads (i.e., usually greater than 100 g BOD/m³.day, equivalent to more than 3000 kg/ha.day for a depth of 3 m). They normally do not contain dissolved oxygen or algae. In anaerobic ponds, BOD removal is achieved by sedimentation of solids, and subsequent anaerobic digestion in the resulting sludge. The process of anaerobic digestion is more intense at temperatures above 15^o C. The anaerobic bacteria are usually sensitive to pH <6.2. Thus, acidic wastewater must be neutralized prior to its treatment in anaerobic ponds. A properly-designed anaerobic pond will achieve about a 40% removal of BOD at 10^o C, and more than 60% at 20^o C. A shorter retention time of 1.0 - 1.5 days is commonly used ⁽¹⁸⁾.

The anaerobic ponds are designed on the basis of volumetric loading (λ_v , g/m³/d), which is given by:

$$\lambda_v = L_i Q / V_a \quad \dots(2.31)$$

Where:

L_i is influent BOD, (mg/L).

Q is flow rate (m^3/day).

V_a is anaerobic pond volume (m^3).

Kayombo ⁽¹⁸⁾, recommended that the loading should be between 100 – 400 $g/m^3.day$, in order to maintain anaerobic conditions. The hydraulic retention time is then calculated, using equation 2.32 as follows:

$$t_{an} = V_a / Q \quad \dots(2.32)$$

A retention time less than one day should not be used for anaerobic ponds; if it occurs, however, a retention time of one day should be used, and the volume of the pond should be recalculated. Table (2.6) illustrates the permissible loadings to the anaerobic ponds ⁽¹⁸⁾.

Table 2.6 Design value of permissible volumetric BOD loadings on, and percentage BOD removal in, anaerobic ponds at various temperatures (18).

Temperature ($^{\circ}C$)	Volumetric loading ($g/m^3.day$)	BOD removal (%)
<10	100	40
10-20	20T-100	2T+20
20-25	10T+100	2T+20
>25	350	70

B- Facultative ponds

Facultative ponds (1-2 m deep) are of two types: primary facultative ponds that receive raw wastewater, and secondary facultative ponds that receive particle-free wastewater (usually from anaerobic ponds, septic tanks, primary facultative ponds, and shallow sewerage systems). The process of oxidation of organic matter by aerobic bacteria is usually dominant in primary facultative ponds or secondary facultative ponds.

Several design formulas with operational data were presented in the design manual ⁽¹⁹⁾. Calculations of the size of a facultative pond are also illustrated for the areal loading rate procedure, Gloyna equation, Marais-Shaw equation, plug-flow model, and Wehner-Wilhelm equation. In this section, design method of the Wehner and Wilhelm model are discussed.

Wehner and Wilhelm ⁽²⁰⁾ used the first-order substrate removal rate equation for a reactor with an arbitrary flow-through pattern, which is between a plug-flow pattern and a complete-mix pattern. Their proposed equation is:

$$\frac{C}{C_o} = \frac{4a \exp(1/2D)}{(1+a)^2 \exp(a/2D) - (1-a)^2 \exp(-a/2D)} \quad \dots(2.33)$$

Where:

C = effluent substrate concentration, mg/l.

C_o = influent substrate concentration, mg/l.

$$a = \sqrt{1 + 4ktD}$$

k = first-order reaction constant, 1/h.

t = detention time, h.

D = dispersion factor = H/uL.

H = axial dispersion coefficient, m²/h.

u = fluid velocity, m/h.

L = length of travel path of a typical particle, m.

The Wehner-Wilhelm equation for arbitrary flow was proposed by Thirumurthi ⁽²¹⁾ as a method of designing facultative pond systems. Thirumurthi developed a graph, shown in Fig. (2.6), to facilitate the use of the equation. In Fig. (2.6), the term kt is plotted against the percent BOD₅ remaining $\frac{C}{C_o}$ in the effluent for dispersion factors varying from zero for an ideal plug-flow reactor to infinity for a complete-mix reactor.

Dispersion factors for stabilization ponds range from 0.1 to 2.0, with most values not exceeding 1.0 due to the mixing requirement. Typical values for the overall first-order BOD₅ removal rate constant k vary from 0.05 to 1.0 per day, depending on the operating and hydraulic characteristics of the pond. The use of the arbitrary flow equation is complicated in selecting k and D values ⁽²²⁾.

Temperature adjustment for k can be determined by

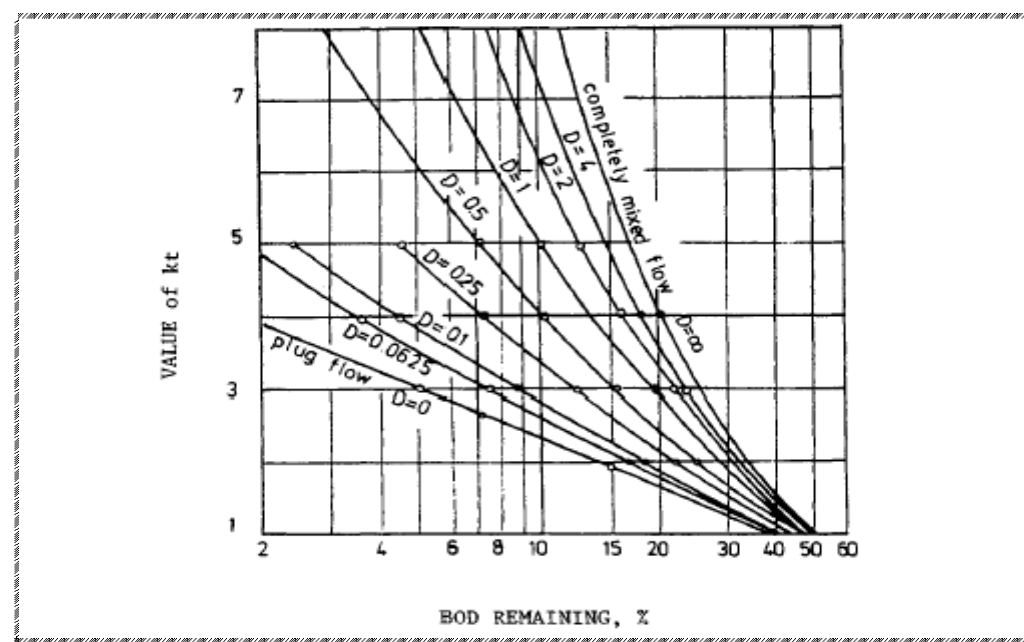
$$k_t = k_{20}(1.09)^{T-20} \quad \dots(2.34)$$

where:

k_t = reaction rate at minimum operating water temperature t , per day.

k_{20} = reaction rate at 20°C, per day.

T = minimum operating temperature, °C.



Fig(2.6) Relationship between kt values and percent BOD remaining for various dispersions factors, by the Wehner-Wilhelm equation(21).

C-Aerobic Ponds

Aerobic ponds also referred to as high-rate aerobic ponds, are relatively shallow with usual depths ranging from 0.3 to 0.6 m allowing light to penetrate the full depth. The ponds maintain DO throughout their entire depth. Mixing is often provided to expose all algae to sunlight and to prevent deposition and subsequent anaerobic conditions. DO is supplied by algal photosynthesis and surface reaeration. Aerobic bacteria utilize and stabilize the organic waste. The HRT in the ponds is short 3 to 5 days.

The use of aerobic ponds is limited to warm and sunny climates, especially where a high degree of BOD removal is required but the land area is not limited. However, very little coliform die-off occurs ⁽¹⁰⁾.

Because of the complex nature of the treatment system, no truly rational design procedure has been developed for aerobic ponds, but several empirical procedures have been proposed. Of the different empirical methods, the one developed by Oswald and Gotass ⁽²³⁾ seems to have the most rational basis. Assuming that the content of the pond are completely mixed and that no settling occurs ⁽²⁴⁾.

From the relationship proposed by Oswald and Gotass , Rich ⁽²⁵⁾ reports that an energy balance between algae mass produced and energy utilized may be expressed as

$$hW_a = ES_R A \quad \dots(2.35)$$

Where:

h =unit heat of combustion of algae cells, cal/g.

W_a =algae biomass produced, g/d.

E =efficiency of energy conversion, %.

S_R = solar radiation incident on the pond surface, cal/cm²-day.

A =surface area of pond, cm².

Oxygen production and algae growth are related by the expression

$$W_{O_2} = pW_a \quad \dots(2.36)$$

Where:

W_{O_2} = Oxygen production as a result of photosynthesis, g/day.

P = oxygenation factor representing the oxygen produced per day per unit of biomass synthesized.

An expression for the surface area of the pond can be obtained by substituting from equation (2.36) for the W_a term in equation (2.35) and solve for A .

$$A = \frac{hW_{O_2}}{pES_R} \quad \dots(2.37)$$

$$h = \left[\frac{R}{7.89} + 0.4 \right] 1000 \quad \dots(2.38)$$

R : represents the degree of reduction of the cellular material formed as a result of the synthesis function and may be estimated the expression

$$R = (100) \frac{(\% \text{carbon})(2.66) + (\% \text{hydrogen})(7.94) - (\% \text{oxygen})}{398.9} \quad \dots(2.39)$$

Fig.(2.7) shows a schematic chart of stabilization ponds

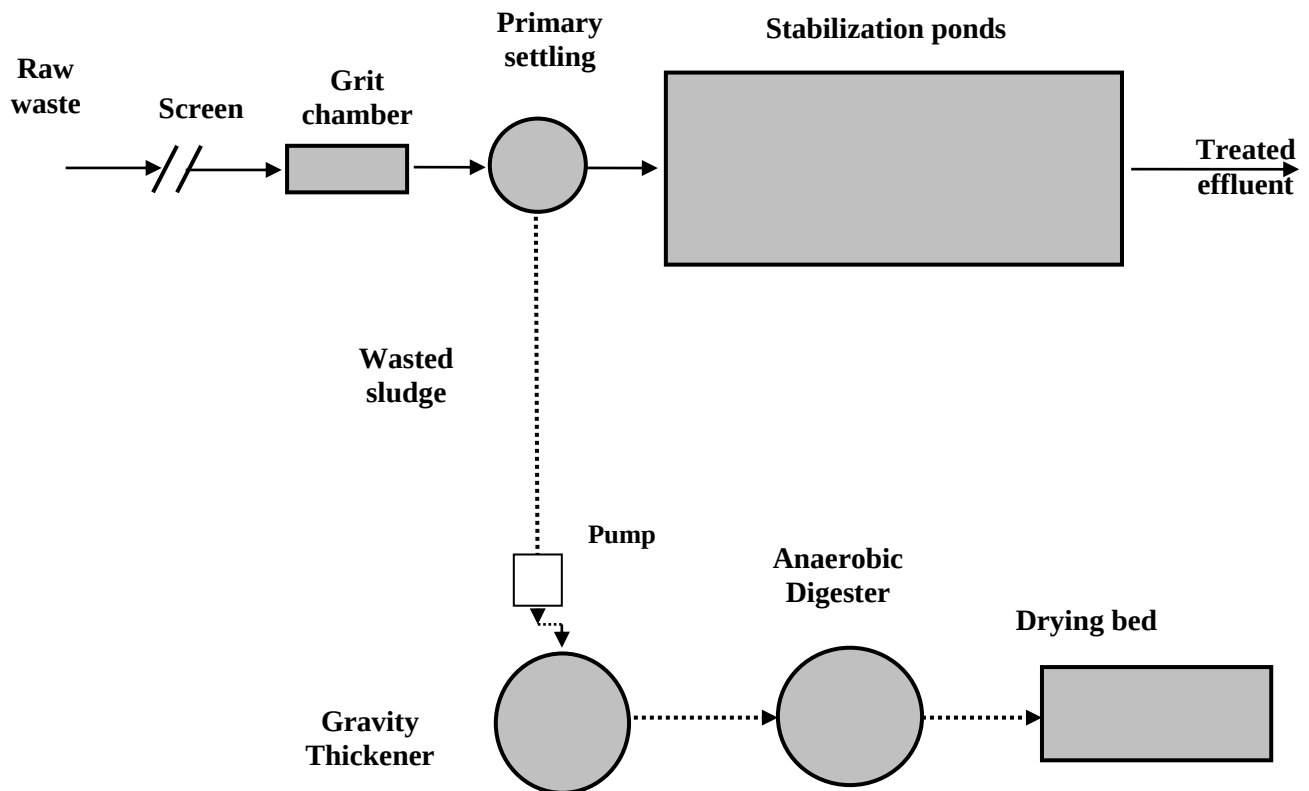


Fig. (2.7): A Stabilizations Ponds Wastewater Treatment Plant in the Present Study.

B- Attached – growth process:

1- Trickling filter

It consists of a shallow bed filled with crushed stones or synthetic media. Wastewater is applied on the surface by means of a self propelled rotary distribution system. The organics are removed by the attached layer of microorganism (slime layer) that develops over media. The underdrain system collects the trickled liquid that also contains the biological solids detached from media. The air circulates through the pores due to natural draft caused by thermal gradient. The trickled liquid and detached biological solids are settled in a clarifier. A portion of the flow is recycled to maintain a uniform hydraulic loading and to dilute the influent.

Based on the organic and hydraulic loading, the trickling filters are classified into low-rate, intermediate-rate, high-rate, and super-rate (roughing filters). Often two-stage trickling filters are used for treating high-strength wastes ⁽¹¹⁾.

Many design equations and procedures for design trickling filters have been proposed over the years. Some of these equations were developed by the National Research Council (NRC), and Eckenfelder.

a-NRC formula ⁽²⁶⁾

$$E_1 = \frac{100}{1 + 0.0561 \sqrt{W/VF}} \quad \dots(2.40)$$

Where:

E_1 =efficiency of BOD removal for first stage including recirculation and sedimentation, %

W =BOD loading to filter, kg/d.

V =volume of filter media, m³.

F =recirculation factor.

The recirculation factor is calculated by

$$F = \frac{1+r}{(1+0.1r)^2} \quad \dots(2.41)$$

Where:

r = recirculation ratio, Q_r/Q .

Q_r =recirculation flow, m³/d.

Q =wastewater flow, m³/d.

For the second-stage, the formula becomes

$$E_2 = \frac{100}{1 + \frac{0.0561}{1-E_1} \sqrt{W'/V'F'}} \quad \dots(2.42)$$

Where:

E_2 = efficiency of BOD₅ removal for second-stage filter, %.

W' = BOD loading applied to second-stage filter, kg/d.

b- Eckenfelder formula ⁽²⁷⁾

$$S_e/S_i = \exp[-KA_s^{1+m}d/q^n] \quad \dots(2.43)$$

Where:

S_e = effluent soluble BOD5, mg/l.

S_i = influent soluble BOD5, mg/l.

K = observed reaction rate constant, m/d.

A_s = specific surface area.

= surface area/volume, m^2/m^3 .

d = depth of media, m.

q = influent volumetric flow rate.

= Q/A

Q = influent flow, m^3/d .

A = cross-sectional area of filter, m^2 .

m, n = empirical constants based on filter media

Fig.(2.8) shows a schematic chart of trickling filter

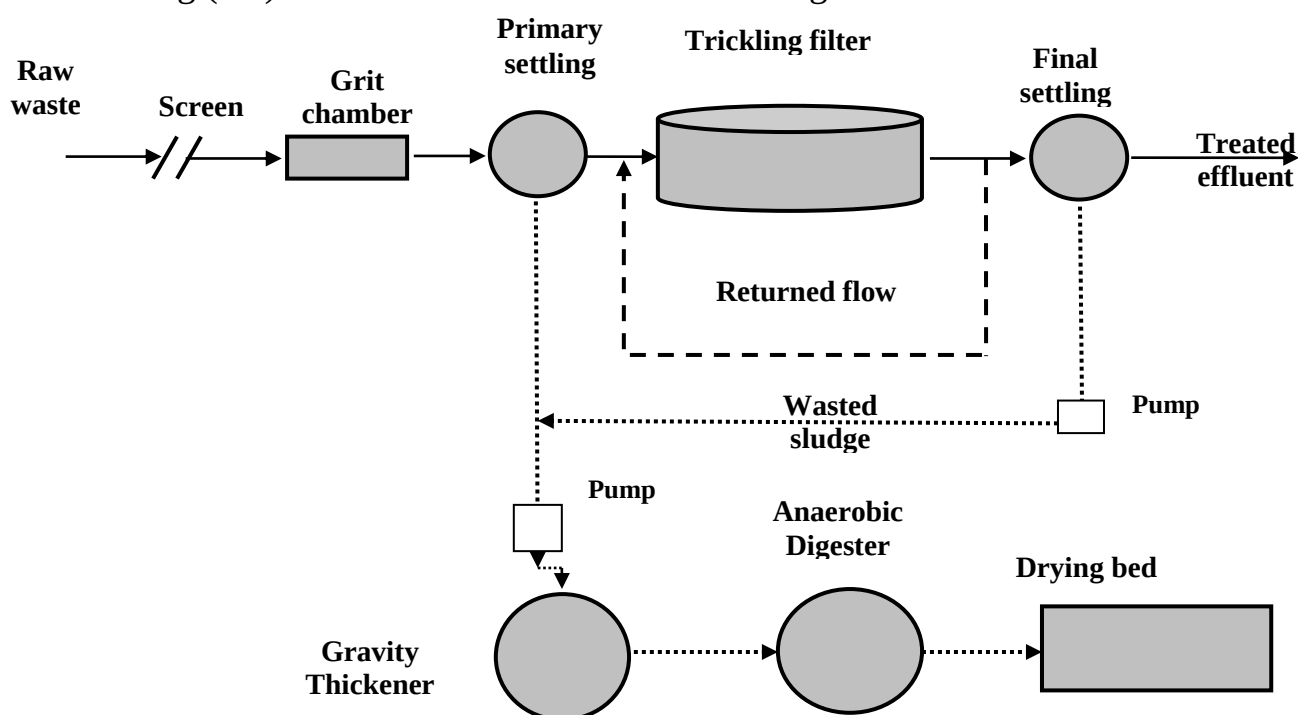


Fig. (2.8): A Trickling Filter Wastewater Treatment Plant in the Present Study.

2- Rotating Biological Contactors (RBC)

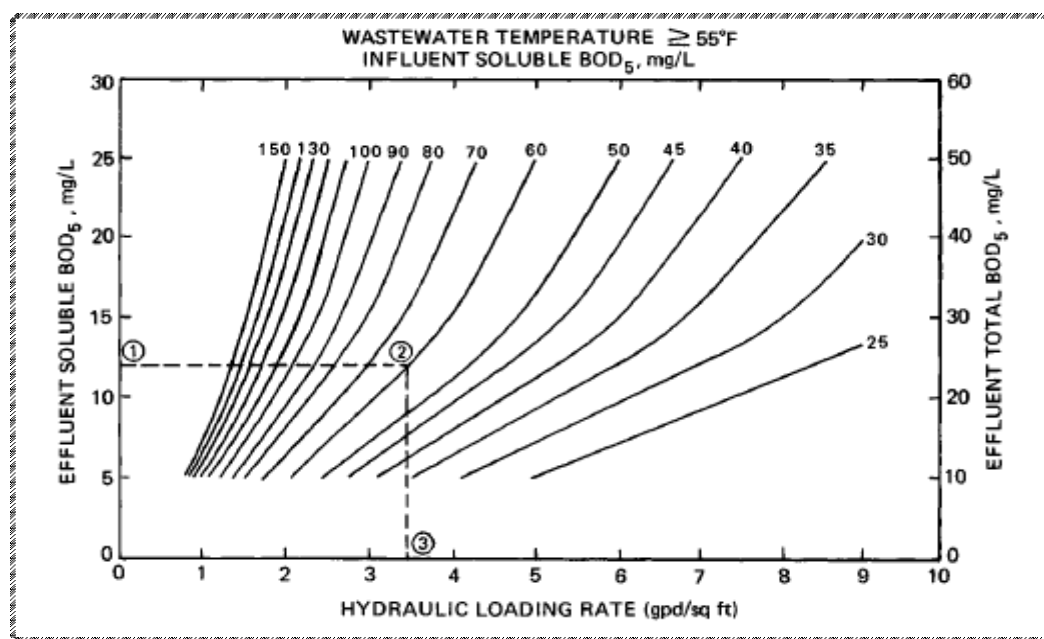
It consists of a series of circular plates (discs) mounted over shaft that rotates slowly. These discs remain approximately 40 percent immersed in a contoured tank. The discs are spaced so that wastewater and air can enter the space. The biological growth develops over the discs that receives alternating exposures to organics and air. The excess growth of microorganisms becomes detached and therefore the effluent requires clarification. The rotating biological contactor has low power demand, greater process stability, higher organic loading, and smaller quantity of waste sludge than the activated sludge process ⁽¹¹⁾.

The RBC are used to treat wastewater for small communities. Effluent suspended solids is much higher than that of activated sludge process, and the BOD removal rate per unit floor area is rather small. The RBC not well suited for use in large scale wastewater treatment plants ⁽²⁸⁾.

Factors affecting the RBC performance include wastewater temperature, influent substrate concentration, hydraulic retention time, tank volume to media surface area ratio, media rotational speed, and dissolved oxygen ⁽²⁹⁾.

An RBC system can be designed on the basis of information on total required media surface area, temperature correction, staging, and organic loading limits.

Design of an RBC unit is based on hydraulic loading rates. Graphs showing relationships between loading rates and efficiency similar to the graph shown in Fig(2.9) can be obtained from manufacturers for specific media and various wastewater strengths. Required surface area is then translated into the number and size of the modules necessary ⁽³⁰⁾.



Fig(2.9) RBC (mechanical drive) process design curves for total and soluble BOD removal ($T > 13^{\circ}\text{C}$, 55°F) (30).

Fig.(2.10) shows a schematic chart of rotating biological contactors.

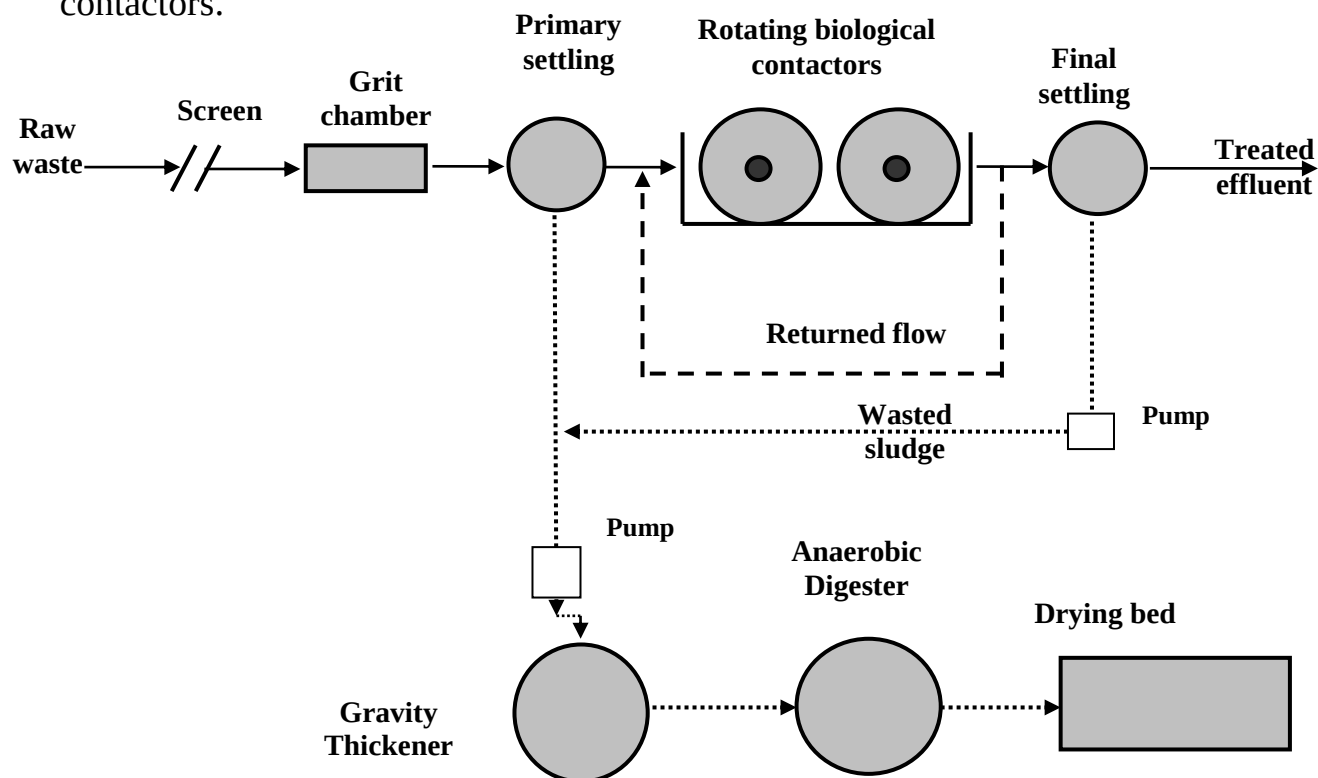


Fig. (2.10): A Rotating Biological Contactors Wastewater Treatment Plant in the Present Study.

3- Bio-towers

Bio-towers are essentially deep trickling filter. Light weight, modular media formed by welding corrugated and flat polyvinyl chloride sheets together in alternating patterns provide structural rigidity for vertical stacking without the excessive weight that would result from stone or slag media ⁽³⁰⁾.

In tower trickling filter, air and wastewater get thoroughly mixed while the wastewater trickles downward. The contact among the wastewater, microorganisms, and air is improved due to the increase in the length of travel. Moreover, the velocity of air and wastewater through the tower trickling filter is increased, causing higher oxygen transfer to the flocs.

The ratio between the diameter and the height of filter should be (1.6). In practice tower trickling filters up to a diameter of 3.5m have been constructed. The height of the filter should not be more than 21m ⁽⁶⁾.

Design of bio-tower is usually based on formulas developed for trickling filter, with allowances being made for medium characteristics. The most commonly used formula was proposed by Eckenfelder and is of the form

$$\frac{S_e}{S_o} = e^{-KD/Q^n} \quad \dots(2.44)$$

Where: S_e = effluent substrate concentration, BOD₅, mg/l.

S_o = influent substrate concentration, BOD₅, mg/l.

D=depth of the medium, m.

K= treatability constant relating to the wastewater and the medium characteristics, min⁻¹.

n= coefficient relating to the medium characteristics.

The above formula does not account for recirculation of wastewater. Because bio-towers almost universally employ recirculation, Eq (2.44) must be modified as follows:

$$\frac{S_e}{S_a} = \frac{e^{-K.D/Q^n}}{(1+r) - re^{-K.d/Q^n}} \quad \dots(2.45)$$

Where S_a is the mixture of the raw and recycled mixture applied to the medium.

$$S_a = \frac{S_o + rS_e}{1+r} \quad \dots(2.46)$$

And r is ratio of recycled flow to the influent flow ⁽³⁰⁾.

Fig.(2.11) shows a schematic chart of bio-tower method.

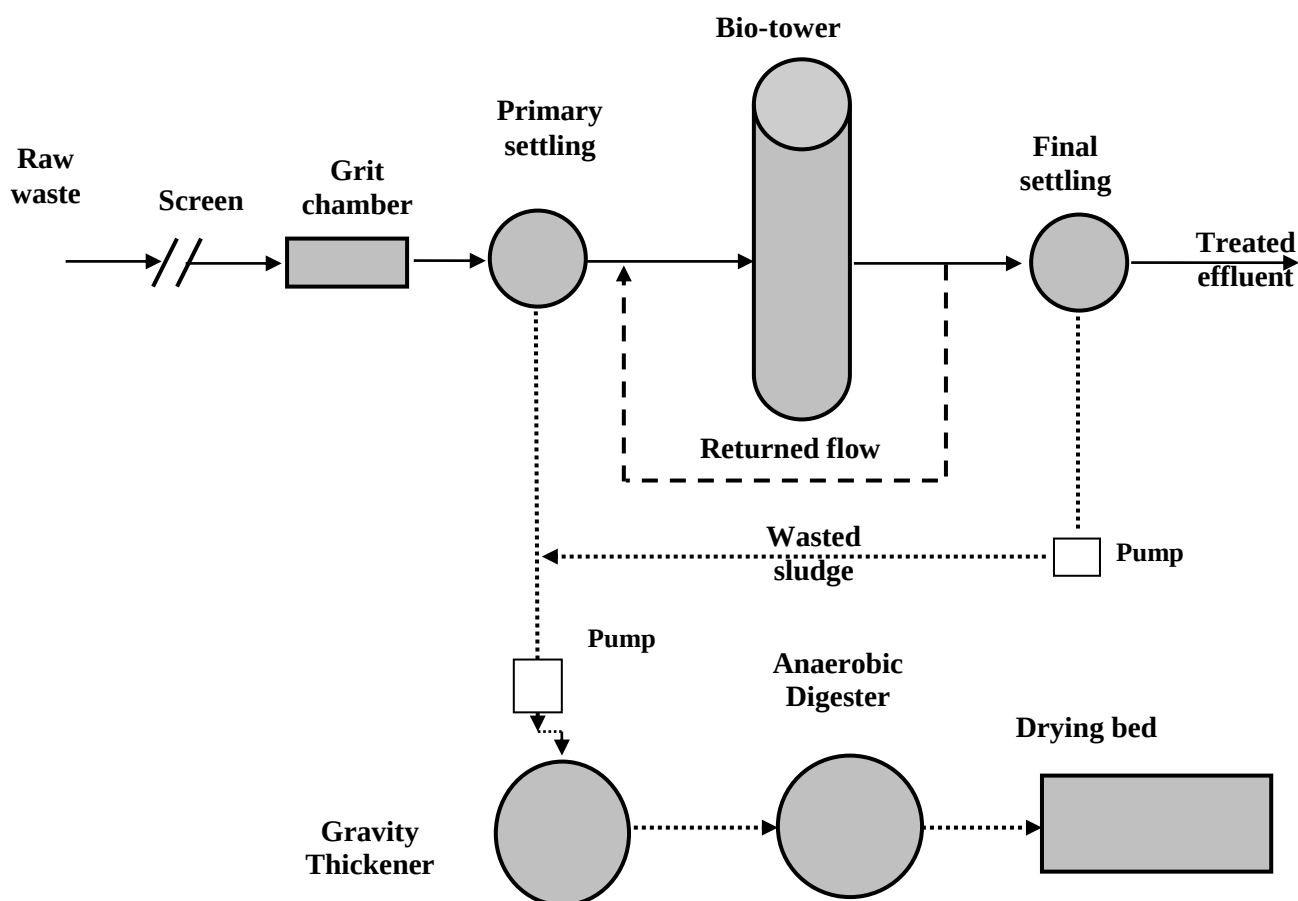


Fig. (2.11): A Bio-Tower Wastewater Treatment Plant in the Present Study.

2.4.4 Secondary Clarifier

The secondary settling tank is an integral part of both suspended- and attached-growth biological treatment processes. It is vital to the operation and performance of the activated sludge process. The clarifier separates MLSS from the treated wastewater prior to discharge. It thickens the MLSS before their turn in the aeration tank or their wasting ⁽⁴⁾.

The secondary settling tank for attached-growth system :

Design of secondary clarifiers for attached-growth system is similar to that for primary clarifiers. Overflow rates from $(25-22\text{m}^3/\text{m}^2.\text{d})$ are commonly used, with a maximum of $50\text{m}^3/\text{m}^2.\text{d}$. no sludge is recycled to the reactor, so the underflow is negligible. Solid production can be expected to range from 0.2 to 0.5 kg/kg BOD₅ removed from the liquid. Well-settled sludge range from 10 to 20 percent solids ⁽³⁰⁾. The minimum side water depth is 3 m with greater depths for larger diameter basins. The retention time in the secondary settling tank is in the range of 2-3 h. The maximum recommended weir loading rate is $[250 - 375] \text{ m}^3/\text{m.d}$ ⁽⁴⁾.

The secondary settling tank for suspended-growth system:

In order to produce the proper concentration of return sludge, activated-sludge settling tanks must be designed to meet thickening as well as separation requirements.

Solids loading rate is of primary importance to insure adequate function in the secondary settling tanks following aeration basins. In practice, most domestic wastewater plants have values of solids volume index in the range of 100-250 mg/L ⁽³¹⁾. Most design engineers prefer to keep the maximum solids loading rates in the range of 4-6 kg/(m².h). Solids loadings rates of 10 kg/(m².h) or more have been found in some well-operating plants. The depth of secondary clarifiers is commonly

designed as 4-5 m. The shapes of settling tanks include rectangular, circular, and square.

The maximum allowable hydraulic loading rate HLR as a function of the initial settling velocity ISV at the design MLSS concentration was proposed by Wilson and Lee⁽³²⁾. The equation is expressed as follows:

$$\text{HRT} = Q/A = 24 \cdot \text{ISV} / \text{CSF} \quad \text{.....(2.47)}$$

Where:

HRT = hydraulic retention time, h

Q = limiting hydraulic capacity, m³/d

A = area of the clarifier, m²

24 = unit conversion factor, 24 h/d

ISV = initial settling velocity at the design MLSS concentration, m/h

CSF = clarifier safety factor, 1.5 to 3, typically 2.

2.4.5 Disinfection

Wastewater contains many types of human enteric organism that are associated with various wastewater

Disinfection is a chemical treatment method carried out by adding the selected disinfectant to an effluent to destroy or inactivate the disease-causing organisms.

The purposes of effluent disinfection are to protect public health by killing or inactivating pathogenic organisms such as enteric bacteria, viruses, and protozoans, and to improve the effluent discharge standards. The disinfection agents (chemicals) include chlorine, ozone, ultraviolet (UV) radiation, chlorine dioxide, and bromine. Design of UV irradiation can be referred to the manufacturer or elsewhere ⁽³³⁾.

Chlorine is widely used, effective at low concentration, inexpensive, forms residual, drawn back, and applied as chlorine gas or

hypochlorite. It acts by oxidizing enzymes of cells, preventing essential metabolic process.

Chlorine is added to a secondary effluent for a certain contact time (20-45 min for average flow and 15 min at peak flow),

The disinfection efficiency of chlorine depends on contact time, chlorine dosage, temperature, pH, nature of liquid and suspended matter, and type and number of organisms.

For disinfection of wastewater a chlorine residual of 0.5 mg/l after 20-30 min of contact period is required. The effluent quality to be disinfected must be evaluated for chemical dosage and contact period.

2.4.6 Sludge Treatment Processes

Solids separated from wastewater by physical and biological processes (primary and secondary treatment) must be treated and suitably discarded. These solids consist of mineral as well as organic matter and are called sludge. A variety of treatment processes and overall sludge management options can be established, depending on the type and quantity of sludge generated. The basic processes for sludge treatment include thickening, stabilization, conditioning, dewatering, and volume reduction.

A- Sludge thickening

The purposes of sludge thickening are to reduce the sludge volume to be handled in the subsequent sludge processing units and to reduce the construction and operating costs of subsequent processes.

Sludge thickening mainly involves physical processes such as gravity settling, flotation, centrifugation, and gravity belts.

In the present study plants that need sludge thickening (activated sludge, trickling filter, bio-tower, rotating biological contactors, and oxidation ponds) deals with gravity thickening, a gravity thickener will be designed on the basis of hydraulic surface loading and solids loading. The

design principles are to be the same as those for sedimentation tanks. Tanks usually range from 10 to 24 m in diameter. Side-water depths vary from 3 to 4 m and floor slopes vary from 1 : 4 to 1 : 6, both depending on the period of time required to thicken the sludge to the concentration and storage volume required to compensate for fluctuations in the solids loading rate period.

Fig.(2.12) shows a schematic of gravity thickener.

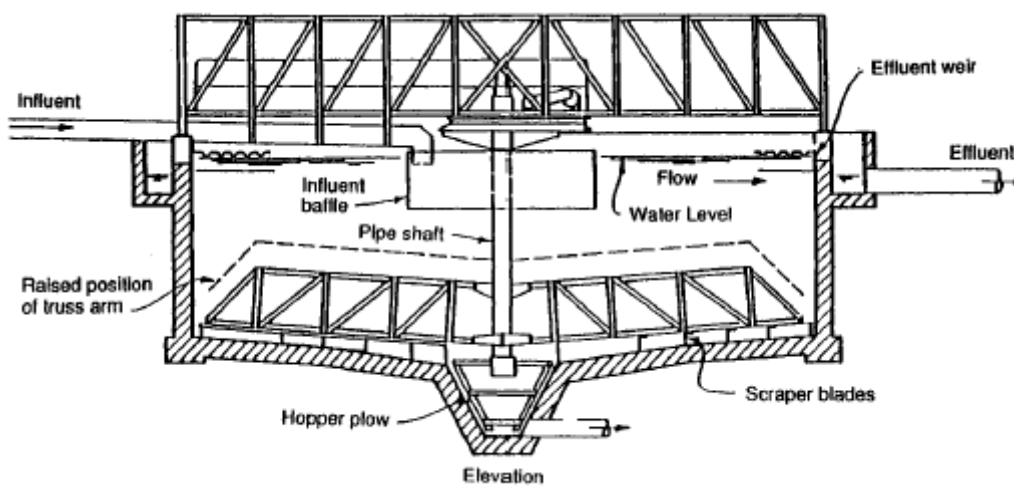


Fig. (2.12): Schematic of gravity thickener(34)

B-Sludge stabilization

After sludge has thickened, it requires stabilization to convert the organic solids to a more refractory or inert form. Thus the sludge can be handled or used as a soil conditioner without causing a nuisance or health hazard. The purposes of sludge stabilization are to reduce pathogens, eliminate odor-causing materials, and to inhibit, reduce, and eliminate the potential for putrefaction. Treatment processes commonly used for stabilization of wastewater sludge's include anaerobic digestion, aerobic digestion, and chemical (lime, disinfectants) stabilization and composting (4).

The present plants that need to sludge digestion are: (activated sludge, trickling filter, bio-tower, rotating biological contactors, and oxidation ponds) deals with anaerobic digestion.

Anaerobic digestion is the predominant method and also usually effective and energy efficient. The process involves gasification, liquefaction, stabilization, colloidal structure breakdown, and water reduction .

The most important factors affecting the performance of anaerobic digesters are solids residence time, hydraulic residence time, temperature, pH, and toxic materials.

The thickened sludge is fed into digester tanks where it stays for up to 14 days. During this time it is heated to a constant 35°C, allowing bacteria within the waste to break down organic matter in the sludge to water, carbon dioxide and methane gas.

The design of anaerobic digester. The most common tank design is a low vertical cylinder 6 to 38 m in diameter with a sidewall depth of 6 to 14 m. Tanks are usually made of concrete, although steel tank designs are not uncommon in smaller. Tank floors are usually conical with slopes varying between 1: 3 and 1: 6. Digested sludge is withdrawn from the low point in the centre of the tank.

There is a great interest in the utilization of sludge gas as an energy source. The digester gas contains approximately 60-70 percent methane and 25-30 percent carbon dioxide. And small amount of hydrogen, nitrogen, hydrogen sulfide, and other gases⁽¹¹⁾ . The quantity of methane gas produced can be computed by the following equation ⁽³⁵⁾:

$$V_{\text{gas}} = 350[Q(S_0 - S)/(1000) - 1.42P_x] \quad \dots (2.48)$$

Where;

V_{gas} = volume of methane produced at standard conditions (0 °C, and 1 atm), L/d.

C- Sludge dewatering

An advantage of dewatering is that it makes the sludge odorless and non-putrescible. the drying beds are the best and most suitable type of dewatering of sludge.

The simplest method of drying the sludge is to apply it on open drying beds, but the sludge must have been digested. It is difficult to dry fresh sludge without causing an odor nuisance.

In a sludge-drying bed, part of the sludge water is removed by seepage and part is evaporated by sun's heat. When the digested sludge is applied on the drying beds, the gases adhering to the sludge particles are released to atmosphere. As a result, the sludge particles rise to the surface and float. The sludge water settles down and seeps out of the bed.

The drying area is partitioned into individual beds, 6 m wide by 6-30 m long, or of a convenient size so that one or two beds will be filled in a normal cycle. Design area requirements for open sludge drying beds are based on sludge loading rate, which is calculated on a per capita basis or on a unit load of mass of dry solids per unit area per year. On a per capita basis, surface areas range from 0.09 to 0.23 m²/ person, depending on sludge type ⁽¹⁴⁾.

D- Sludge holding tank

It means any structure to hold the wastewater without any leak, seepage or overflow into the surrounding environment. The holding tank shall normally be rectangular. In special circumstances alternative shapes may be acceptable at the discretion of the concerned municipality. The nominal water depth of the tank must not be less than 1.5 meters and not more than 2 meters ⁽³⁶⁾.

2.5 Literatures Review of The Studied Factors

Wastewater treatment plant containing various unit processes and equipment should be designed and sized for the most critical

environmental conditions to which will be exposed at the proposed plant site. Design of wastewater treatment plant may be affected by several factors such as population, temperature fluctuation, flow and composition of the wastewater to be treated, substrate concentration, availability of nutrients, dissolved oxygen, pH value, winds, topography, and others ⁽¹⁾.

2.5.1 Population

The wastewater generated depends upon the population and per capita contribution of wastewater. Predicting the dynamic of human population is important in design treatment works because it is the bases for the determination of design capacity for municipal water and wastewater treatment systems and for waste reservoirs ⁽³⁷⁾.

2.5.2 Average and Maximum per Capita Sewage Contribution

Unless satisfactory justification can be given for using a lower or higher per capita flow, new wastewater systems should be designed on the basis of an average daily per capita (lpcd) flow of wastewater of not less than (270 liters) nor greater than (350 liters) ⁽¹⁰⁾.

In Iraq wastewater systems are designed on the basis of an average daily per capita flow of wastewater of (270 liters).

2.5.3 Organic Loadings and Total Solids Concentrations

The strength (organic content) of a wastewater is usually measured as 5-days biochemical oxygen demand (BOD₅), and total suspended solids⁽³³⁾.

In middle Euphrates regions wastewater systems designed on the basis of an average daily per capita for BOD and TSS production of wastewater of (70 liter), (90 liters) respectively.

2.5.4 Variation in Temperature

The temperature of the sewage is very important in assessing the overall efficiency of a biological treatment process, the fermentation in the sludge layer in oxidation ponds depends very much on temperature. The activity of bacteria increases seven time for every 5°C rise in temperature ⁽¹⁾.

Eckenfelder ⁽³⁹⁾ described that performance of the organisms responsible for degradation the organic pollutants in the waste in biological treatment are significantly impacted by change in wastewater temperature. He has described the kinetics of oxidation as:

$$\frac{S_o - S}{XTK_d} = \frac{S}{S_o} \quad \dots (2.49)$$

Where;

T = temperature, °C.

K_d = reaction rate, day⁻¹.

As shown in equation above, one of the factors, which relates to biodegradation is the MLVSS concentration (X). To overcome a reduction in the reaction rate (K_d) in winter, it is possible to increase the number of microorganisms. This will help maintain the overall removal rate and thus maintain treatment plant performance. In addition, he found that an increase in temperature above 20 °C would significantly reduce in sludge production rate and relation between temperature and reaction rate expressed be following:

$$k_t = k_{20} \theta_f^{T-20} \quad \dots (2.53)$$

Where;

k_t = reaction rate at any temperature, d⁻¹.

k₂₀ = reaction rate at 20°C, d⁻¹.

T = temperature, °C.

θ_f = temperature correction factor (1.04).

The effect of temperature on biodegradation rates has long been investigated and is generally well documented. Temperature decreasing may result in a significant decreasing in the soluble (BOD) removing rate⁽⁴⁰⁾. Fig. (2.12) illustrates the effect of temperature on wastewater treatment ability.

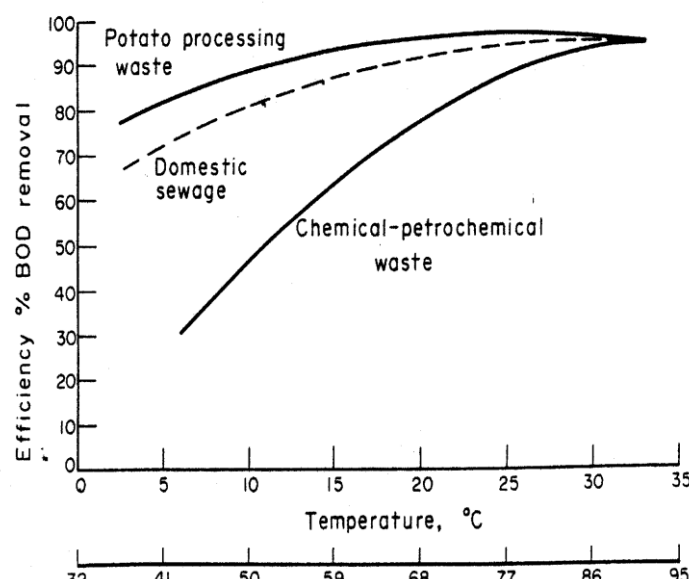


Fig.(2.13): Effect of Temperature on Wastewater Treatment (40).

2.5.5 Wind direction

Wind plays a role in mixing and movement of seawater, freshwater and wastewater. A distinctive element of the wind environment is a pronounced diurnal variation in wind speed, especially in spring and summer seasons.

2.5.6 Toxicity

Toxic chemicals in the wastewater can enter the bacteria and the rate of growth is reduced. Toxic shocks can be a severe problem in activated sludge operation.

P. Davies and Murdoch ⁽⁴¹⁾ referred in their study, that storm waste flows can introduce toxins from leachates and other urban runoffs, which will cause reduction in treatment plant efficiency resulting from the effects of toxicity on the activated sludge bacteria therefore result in unacceptable levels of effluent toxicity.

2.5.7 Nutrients Requirements

Growth requirement also dependent on the intake of nitrogen and phosphorus. The optimum ratio of C: N: P in the mixed liquor is generally thought to be 100: 5: 1. The ratio of these nutrients in settled domestic sewage is variously reported as 100: 17: 5 or as 100:19: 6. This indicates that nitrogen and phosphorus will not be limiting for growth. Nutrients are essential for obtaining maximum bacterial growth and to optimize carbonaceous treatment. From an operational point of view, lack or an insufficiency of a critical nutrient may result in incomplete treatment, because the bacteria are unable to grow optimally ⁽¹⁷⁾.

2.5.8 Site Evaluation

The major factors in the selection of suitable sites for treatment facilities include the following: topography; availability of a suitable discharge point; and maintenance of a reasonable distance from living quarters, working areas and land use of the proposed facilities, as reflected by the master plans. Also, sufficient space must be allocated for future expansion which includes the provision of increased capacity for existing processes and the addition of new types of units known to be required for upgrading redesigned systems to the future requirements of more stringent stream and effluent standards ⁽⁹⁾.

2.5.9 Infiltration / Inflow (I / I)

Infiltration means water other than wastewater that enters the sewer system from the ground through defective pipe, pipe joints, and

manholes. Inflow means wastewater that enters a sewer system from sources such as roof leaders, foundation drains, manhole covers, sanitary sewers, storm water runoff and other drainage structures. I/I is a part of every collection system and must be taken into account in the determination of an appropriate design flow ⁽⁴²⁾.

If infiltration is known to be negligible at manholes, then an infiltration allowance may be calculated based upon area served by network as shown in Fig. (2.14). Curve A should be used for worst conditions when pipes are old and joints are composed of cement. Curve B applies to old pipes with hot or cold asphalt joints or for new pipes known to have poor joints. Curve C is used for new sewers where groundwater does not cover inverts and when joints and manholes are modern and quite tight ⁽⁵⁾.

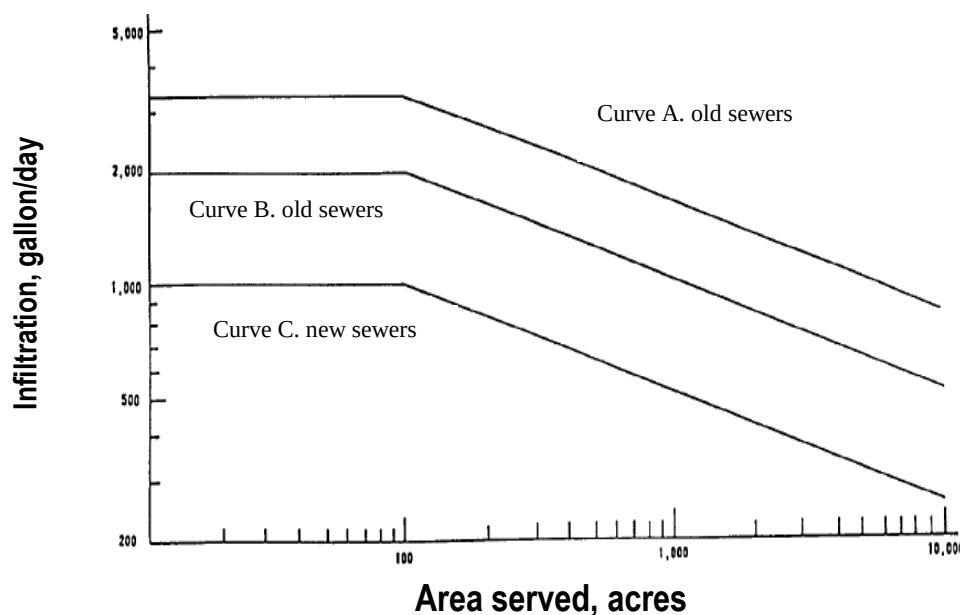


Fig.(2.14): Infiltration allowances (5).

Also, infiltration flow is calculated by as a percent of average design flow rate as ⁽⁴³⁾:

$$Q_i = (5-10)\%Q_{avg} \quad \dots (2.54)$$

where;

Q_i = infiltration flow rate, m^3/d

For there is no information available, infiltration flow can be estimated as:

- 1- (24 - 95) m³/day. Km pipe length.
- 2- (0.46) m³/day. cm pipe diameter.
- 3-Infiltration rate =(0.020-0.28) l/s.ha (some engineering designers assumed typically = 0.1 l/s.ha), so infiltration flow can be calculated as infiltration rate × area served by network ⁽¹⁾.

2.5.10 Variation in Raw Waste Load (RWL)

Robert and Stanley ⁽⁴⁴⁾ studied the response of activated sludge process to shock organic load. They found a quick response, quick increase in effluent (BOD₅) and (TSS) concentrations, of activated sludge process to input organic concentration changes. Shock loads caused a distribution in steady - state conditions in reactors. These shock loads may be either qualitative or quantitative in nature and affect treatment performance during the transient state.

S. Davies ⁽⁴⁵⁾ stated that the increasing in the concentration of substrate, the growth rate increases exponentially and then levels off. Therefore, with further increase in concentration of substrate in the medium, there is no further increase in growth. The bacteria are at their maximum growth rate.

He concluded that the curve does not pass through the origin, because at very low concentrations, substrate is being used for respiration simply to keep the bacteria alive. At concentrations below A as shown in Fig (3-15), the bacteria remain alive, but are not growing. The slope of the growth versus substrate concentration curve can be important ⁽⁴⁵⁾.

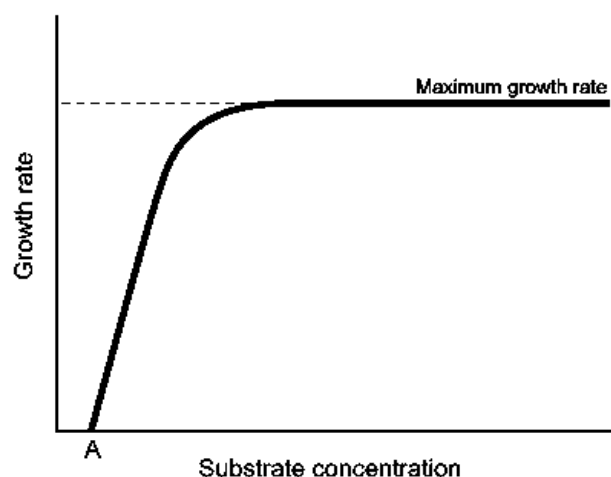


Fig. (2.15): The growth rate increases exponentially with increase in substrate concentration, to a maximum (45).

2.5.11 pH value

The pH of wastewater needs to remain between (6.0 - 9.0) to protect organisms. Acids and other substances that alter pH can inactivate treatment processes when they enter wastewater from industrial or commercial sources. A large problem in some plants is a low pH (to as low as pH = 6.0) caused by extensive nitrification and low wastewater alkalinity. This often causes pin floc and high effluent turbidity. Most microorganisms cannot survive below pH=4.0 as the rises over 9.5, the hydroxyl ion begins to exert toxic effect ⁽⁴⁶⁾.

2-5-12 Design Period

Wastewater treatment plants are nowadays planned and designed based on forecasts encompassing a period of 25 to 40 years. However, the environment of a WWTP usually changes drastically and unpredictably during this extended life span. Unforeseen changes like new regulations, shifting economic conditions or improving technologies are not uncommon and may require costly adaptations in the operation of a plant. This future uncertainty can not be adequately characterized through the forecast based approaches used nowadays in the field of wastewater treatment.

Qasim Syed ⁽¹¹⁾ declared that the selection of design period is made large enough to satisfy community needs for several years in the future. This selection depends on useful life of treatment units, future growth in population, service area, water demand and wastewater characteristics and performance of treatment facility during the initial year when it's oversized this choice lies between (10-25) years.

Arceivela ⁽¹⁷⁾ showed that the very long periods are advisable and often their sizing is done so as to sufficient at full development, in the future. If the treatment units are designed for unduly longer periods, they may be quite oversized in their early life, and can sometimes be a disaster, especially in a warm climate, where septic conditions can be readily induced in settling tanks and other an aerated unit.

2-5-13 Operating and Maintenance Costs

It includes labor cost, electricity cost, chemical cost, and power consumption cost.

Treatment cost of unit process vary with: The volume of waste flow to be treated (increasing volume reduce unit cost), the variability of the rate of incoming waste flow (cost per volume of water treated increases with the peak-to-volume ratio), the concentration of the waste (the more concentration the waste, the higher the cost will be), the character of the waste and the required degree of waste removal ⁽⁴⁷⁾.

2-5-14 Change in Land Use

Traditional design and upgrade concepts for wastewater treatment plants are based on the forecasting of load parameters over a period of 25–40 years. This approach is adequate as long as the environment of a wastewater treatment plant is stable or predictable over this long time period. However, these conditions are usually not met, as the catchment area, discharge requirements, available technology and institutional

conditions of a wastewater treatment plant may change drastically over time.

The characteristics of the catchment area can change substantially during the long time span between the initial planning and the beginning of operation of a wastewater treatment plant. Changes continue occurring during the operational life of a plant ⁽⁴⁸⁾.

2-5-15 Other and General Factors

The main criteria for the selection of a suitable treatment method are: the cost, the hygienic impacts, the requirements for operation, the final disposal possibilities, subsequent utilization of the receiving water body, the BOD reduction, and the land demand ⁽⁴⁹⁾.

The exact flow sheet configuration selected will also depend on factors such as: the designer's past experience, company and regulatory agency policies on the application of specific treatment methods, the availability of suppliers of equipment for specific treatment methods, the maximum use that can be made of existing facilities, initial construction costs, future operation, and maintenance costs ⁽⁵⁾.

2.6 Literature Review of the Designed Computer Programs for Design Wastewater Treatment Plant

AL- Turaihy, ⁽¹⁾ developed a computer program for designing five types of wastewater treatment plant which are conventional activated sludge, extended aeration, trickling filters, oxidation ponds, and rotating biological contactors.

Tomei, ⁽⁵⁰⁾ developed Activated Sludge Computer Aided Modeling (ASCAM). The ASCAM software can be utilized in design and operation of activated sludge plants for carbonaceous substrate and nutrient removal. The following process layouts are considered: carbonaceous

substrate (COD) removal in one aerobic stage; COD and nitrification in one aerobic stage; COD and nitrogen removal in a two stage anoxic aerobic single sludge system. In the previous plant lay -outs an anaerobic stage as first reactor of the plant can be included in order to perform the biological phosphorus removal.

Tongchai, ⁽⁵¹⁾ developed The Integrated Film Activated Sludge Process (IFAS) to reduce the cost of additional facilities required to complete year round nitrification in the design of new or retrofit wastewater treatment plants. The purpose of his project was to develop a computer-based mechanistic model, called IFAS (Integrated Film Activated Sludge Process), which can be used as a tool by scientists and engineers to optimize their designs and to troubleshoot a full-scale treatment plant. The program also can be employed to assist researchers conducting their studies of IFAS wastewater treatment processes.

Raeed, ⁽⁵²⁾ developed a computer program designed in programming language (Visual Basic 6) for design conventional activated sludge plant. The capability of the software is to design wastewater treatment plant with a capacity ranging from (15000 to 945000) m³/ day with period of expansion for 60 years and to assess the effect of flow rate variations on the design. Using two systems for wasted sludge the first system (1) is to treat wasted sludge from the aeration basin, while system (2) is to treat wasted sludge from the secondary clarifier.

Chapter Three

Application of Computer Program

3.1 Introduction

The description of the computer program that made in the present study for designing deferent types of wastewater treatment plant and it's verification were included in this chapter. The environmental factors affect designing and sizing of various unit processes of wastewater treatment plants are population, specific sewage production, temperature, organic load and solids loading, infiltration rate, design period, dispersion factor, recirculation factor, and area served by network.

3.2 Description of Computer Program

A computer program is written by using Visual Basic 6.0 software. This program includes the required procedures of design of deferent types of wastewater treatment plants. Fig. (3-1) shows the flow chart of the program used. The procedures of the program are as follows:

1. Choose the type of wastewater treatment plant (Conventional Activated Sludge ASP, Extended Aeration EA, Oxidation Ditch OD, Aerated Lagoon AL, Aerobic Pond AP, Facultative Pond FP, Anaerobic Pond ANP, Trickling Filter TF, Rotating Biological Contactors RBC and Bio-Tower BT).
2. The run of the computer program required the inputs data, these data are found in every type of treatment and assumed as follows:
 - ❖ Initial Population, (P_o) : 100000 capita
 - ❖ Specific sewage production, (μ_o): 270 l/c. day
 - ❖ Design period, (t_f) : 25 year
 - ❖ Growth rate, (r_g) : 3.8 %
 - ❖ The specific domestic BOD₅ in raw sewage flow, (B_i): 70g/c.day

- ❖ The specific domestic TSS in raw sewage flow, (T_{si}) : 90 g/c.day
 - ❖ The temperature, (T) : 20 °C
 - ❖ The area served by network, (A_n) : 400 hectare, and
 - ❖ The infiltration rate, (i) : 0.1 l/s.ha
3. The effluent standards were kept constants values $B_e = 20$ mg/l, $T_{s_e} = 30$ mg/l.
 4. Determining of future population (P_f) , peaking factor (PF), total average flow rate (Q_{avg}), peak flow rate (Q_p), minimum design flow rate (Q_m), organic load and solids concentrations (BOD and TSS), then design (Preliminary treatments).
 5. Design Preliminary treatments
 - ❖ Design screening chamber, by using design procedure shown in sec. (2.4.1.A). Input data include maximum velocity through bar rack, bar spacing, bar width, and bar shape factor.
 - ❖ Design grit chamber by using the design procedure shown in sec. (2.4.1.B) to determine volume of grit chambers, dimensions, and quantity of theoretical air supply.
- Note: the steps from(1 to 5) are found in every types of wastewater treatment.
6. Design primary sedimentation tanks (rectangular and circular basins) by assuming values of surface overflow rate SOR, detention time D_t , weir loading rate for determining volume of the basins, dimensions, actual SOR, primary sludge characteristics.
- Note: this step is found only for the type of treatment that need this treatment like (conventional activated sludge, trickling filter, rotating biological contractors, and oxidation ponds).
7. Design a biological treatment according to it's type as shown:
 - ❖ Design an aerobic reactors by using equations shown in sec. (2.4.3.A.1), assuming that (K_d , Y , θ_c , MLSS, X_r).

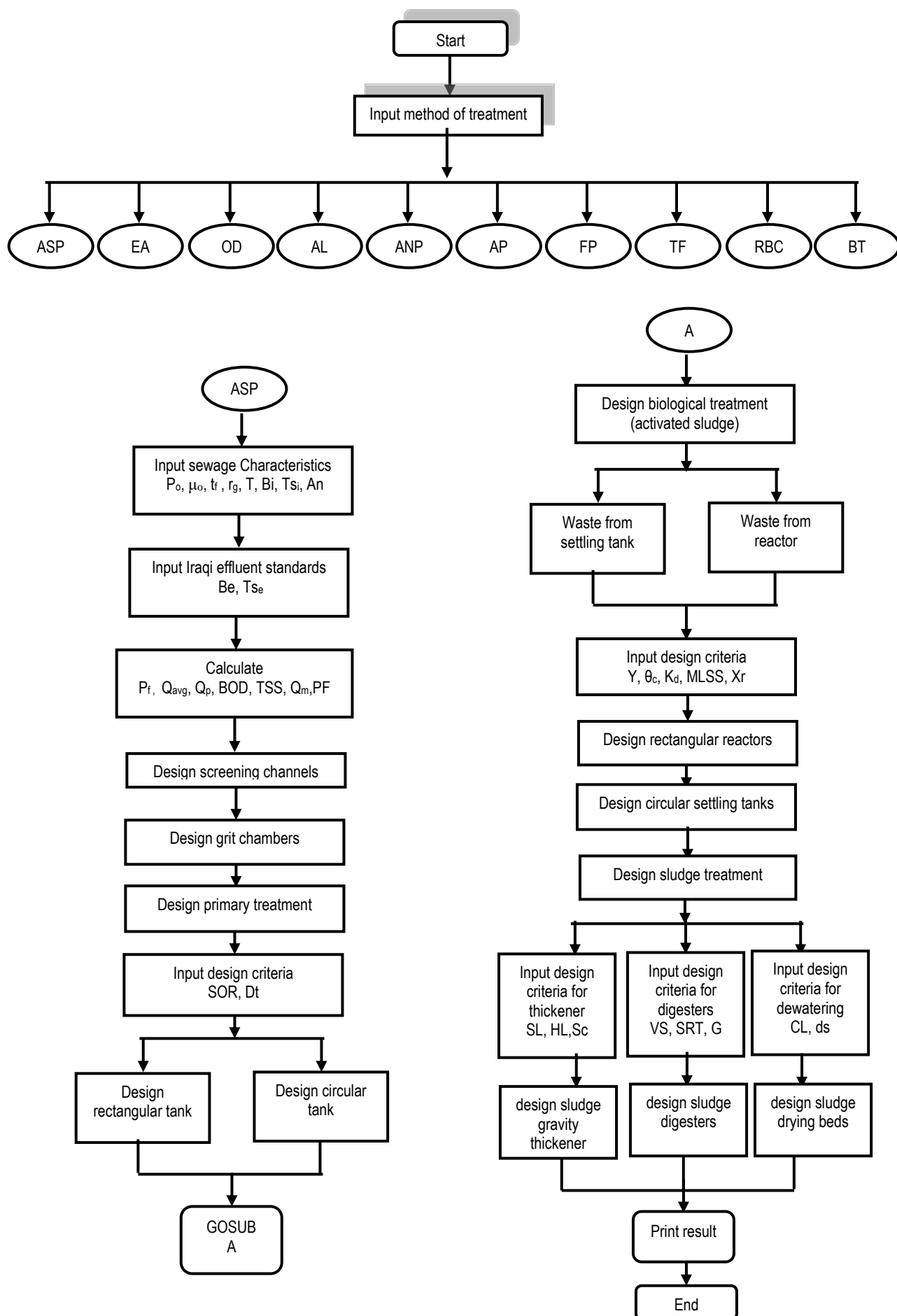
- ❖ Design an extended aeration by using equations shown in sec.(2.4.3.A.2), assuming that (K_d , Y , θ_c , MLSS, X_r).
 - ❖ Design an oxidation ditch by using equations shown in sec. (2.4.3.A.3), assuming that (K_d , Y , θ_c , MLSS, X_r).
 - ❖ Design an aerated lagoon by using equations shown in sec. (2.4.3.A.4), assuming that (K_d , Y , θ_c , X_r).
 - ❖ Design an stabilization ponds by using equations shown in sec. (2.4.3.A.5.A) (anaerobic pond) by assuming that (temperature T and hydraulic retention time HRT), (2.4.3.A.5.B) (facultative pond) by assuming that (temperature T and dispersion factor D), and (2.4.3.A.5.C) (aerobic pond) by assuming that (elevation e and energy utilization efficiency E).
 - ❖ Design a trickling filter by using equations shown in sec. (2.4.3.B.1), assuming that (recirculation ratio r , depth d).
 - ❖ Design a rotating biological contactors by using equations shown in sec. (2.4.3.B.2), assuming that hydraulic loading HL .
 - ❖ Design a bio-tower by using equations shown in sec. (2.4.3.B.3), assuming that (recirculation ratio r , reaction coefficient k_{20}).
8. Design secondary sedimentation tanks (circular basins):
- ❖ For suspended growth system by assuming values of solid flux index, and peak solid loading rate as showing in sec (2.4.4).
 - ❖ For attached-growth system is similar to that for primary sedimentation tanks as showing in sec (2.4.4).
9. Design sludge treatment process
- ❖ For treating wasted sludge that wasting from settling tanks only, these processes include holding tank with design criteria (Temperature T , Mean Cell residence times θ_c) and drying beds with design criteria (per capita loading CL , depth of applied sludge ds) for oxidation ditch, extend aeration, and aerated lagoon.

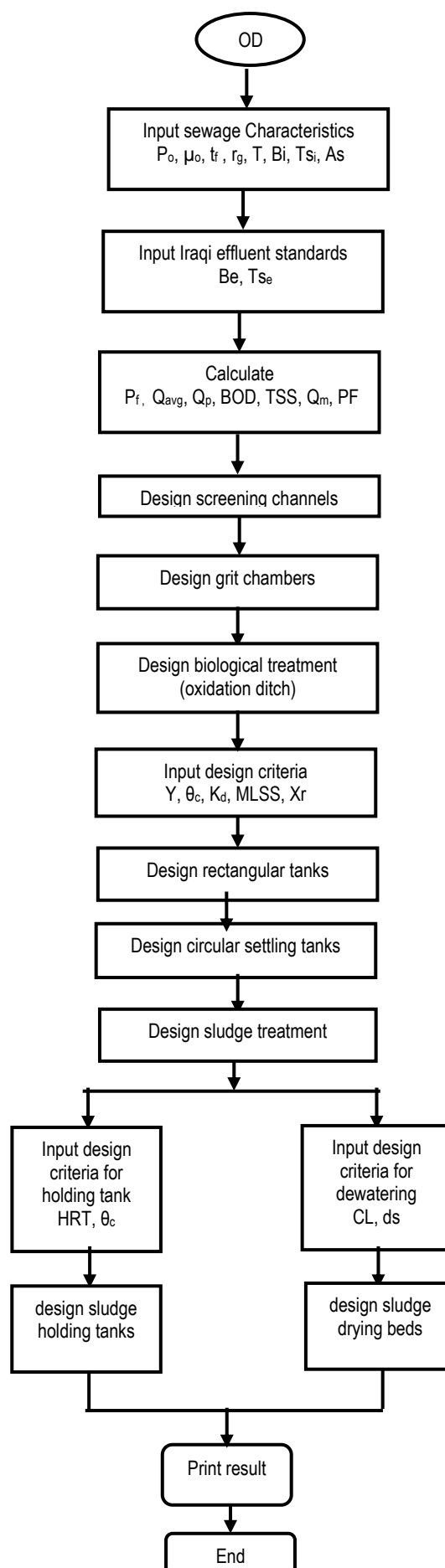
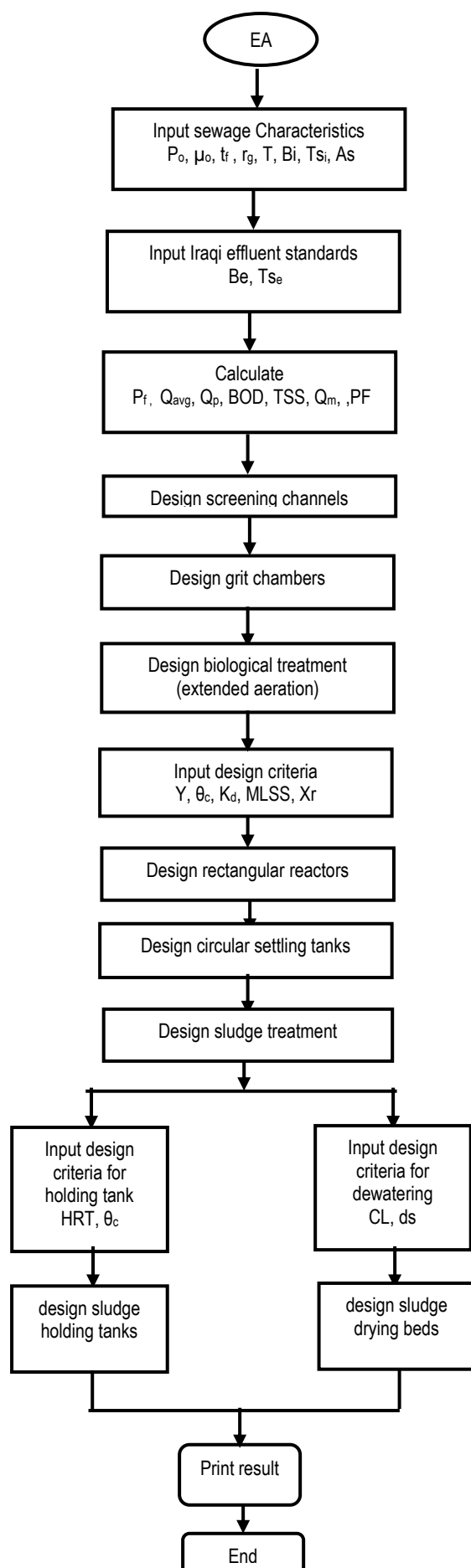
- ❖ Design sludge treatment process for treating (primary sludge and wasted sludge that wasting from settling tanks) these processes include sludge thickening with design criteria (solid loading SL, hydraulic loading HL, Solid concentration S_c), anaerobic digestion with design criteria (volatile solid loading VS, residence time SRT, gas density G), and drying beds with design criteria (per capita loading CL, depth of applied sludge d_s) for the conventional activated sludge, trickling filter, rotating biological contractors, and stabilization ponds).

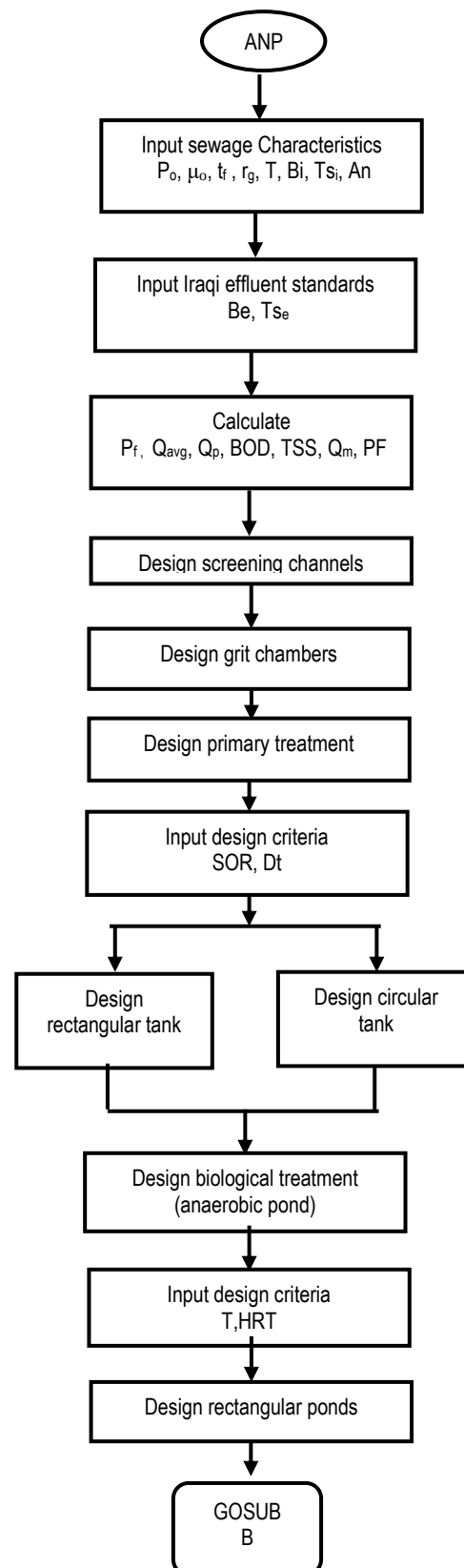
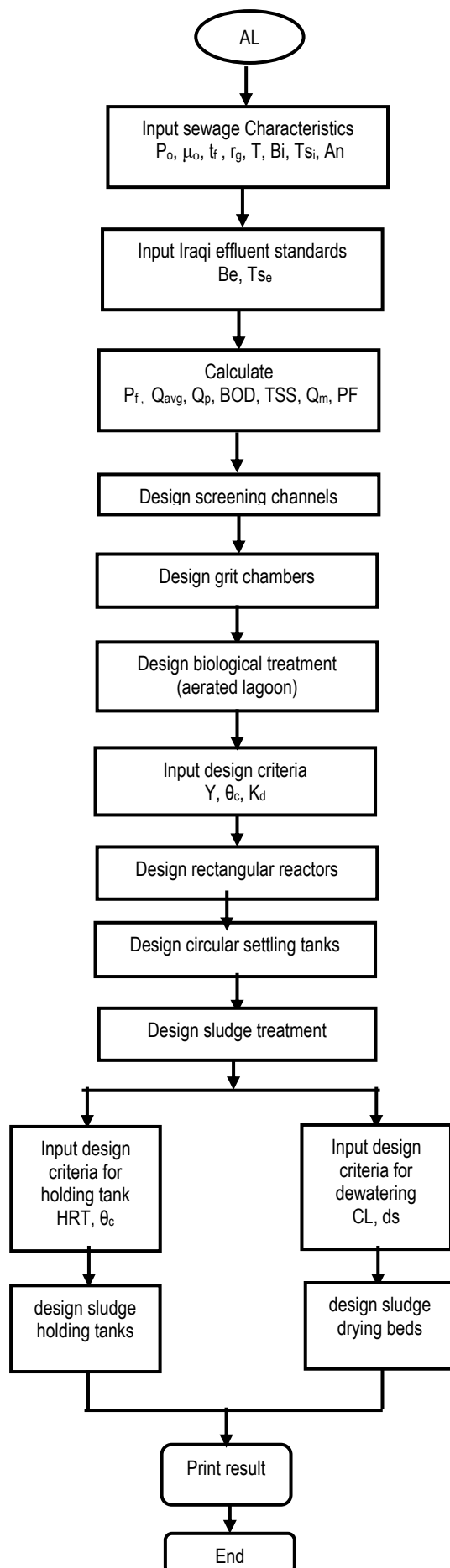
10. The design procedures are repeated from step No. 1 to step No.9 for each different factors.

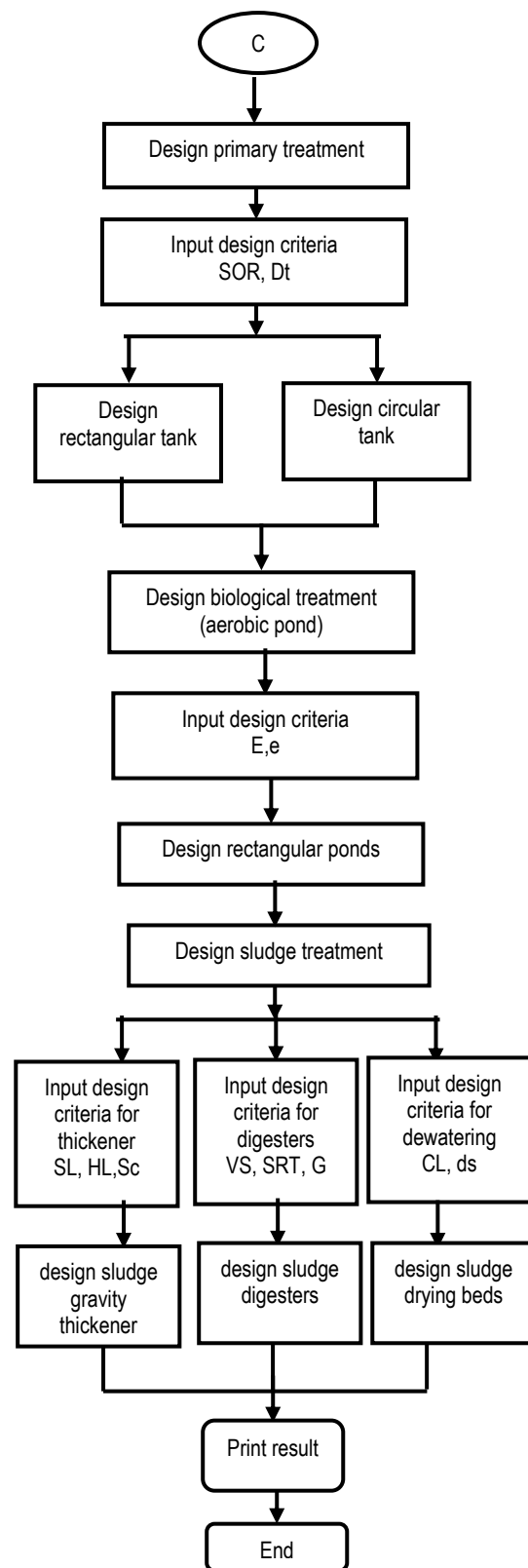
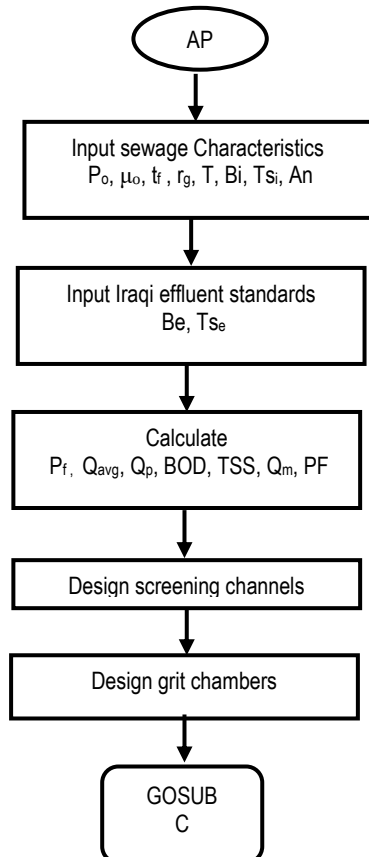
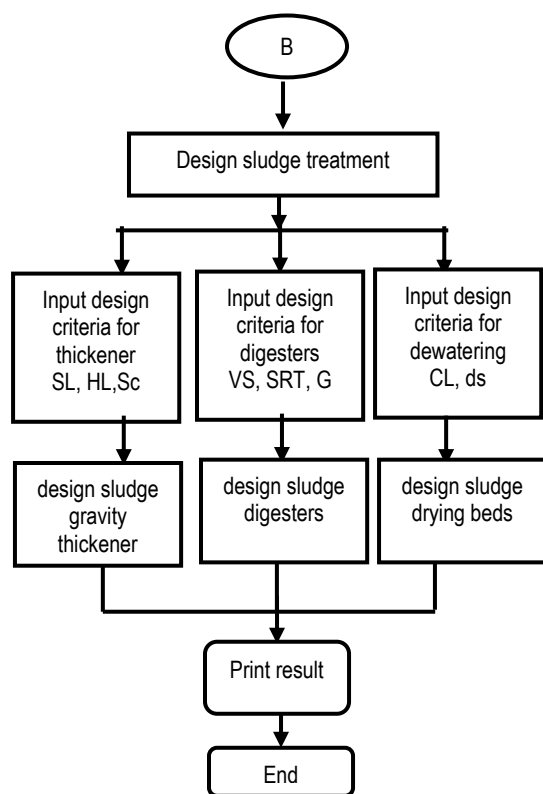
Table (3.1): Values of Characteristics of Sewage as Used in Program of Present Study

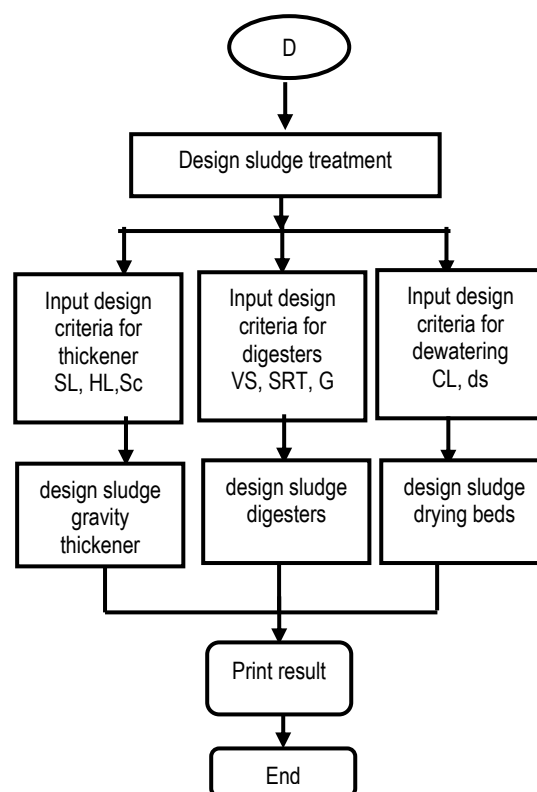
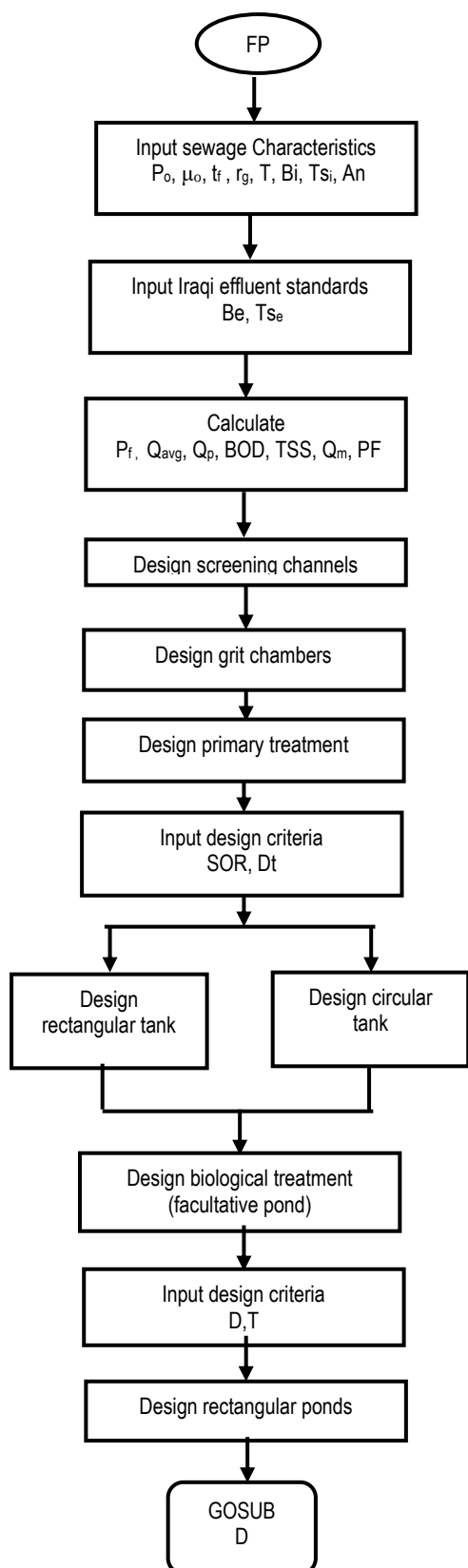
Characteristics	Units	Limits		
		Lower	Upper	Increment
Population	Capita	100000	200000	10000
Specific sewage production	L /c. day	150	350	20
Area served by network	Hectare	100	1000	100
Specified BOD production	g/c. day	30	90	10
Specific domestic S.S	g/c. day	50	130	10
Temperature	$^{\circ}\text{C}$	10	50	10
Infiltration rate	l /s. ha	0.02	0.28	0.1
Design period	year	10	50	25
Dispersion factor	-	0.1	2	0.5

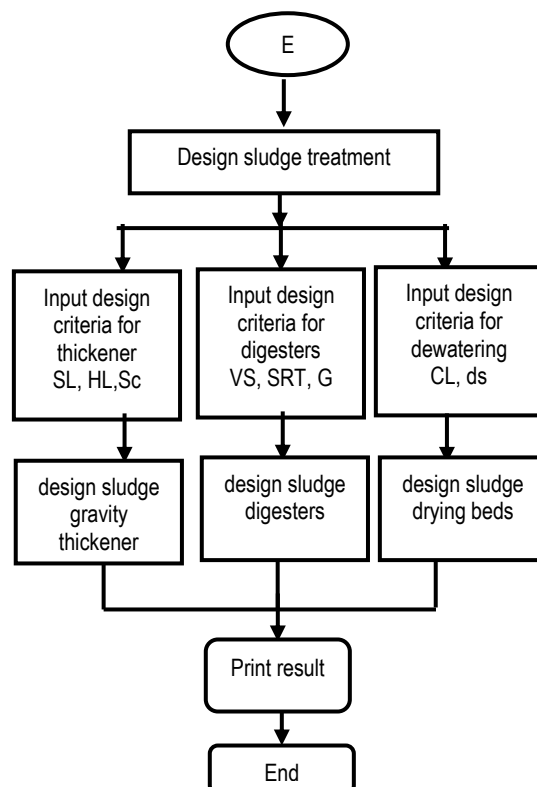
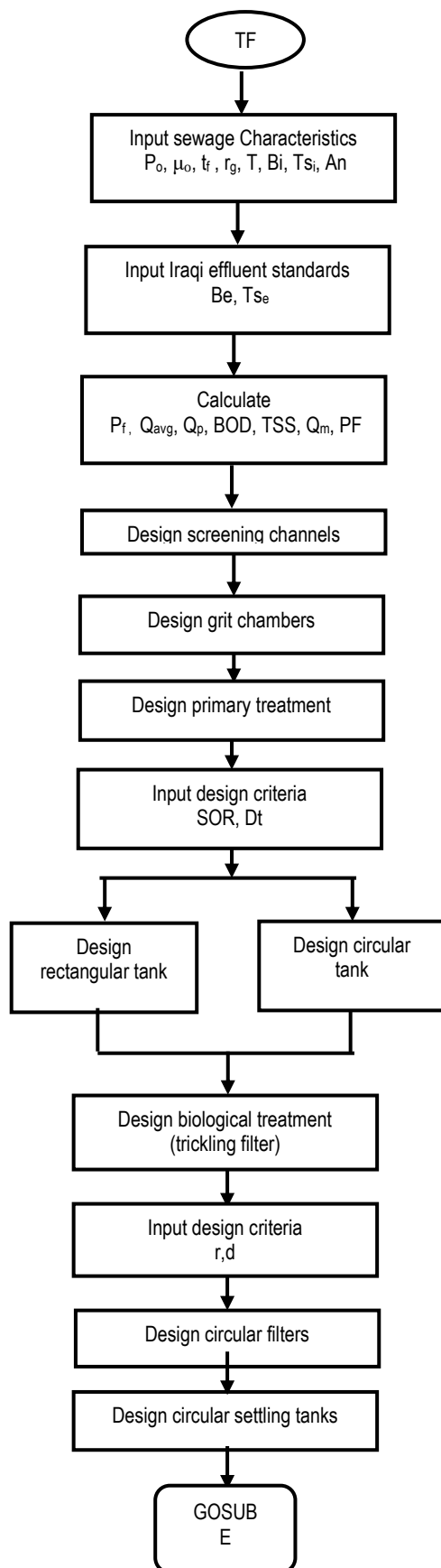


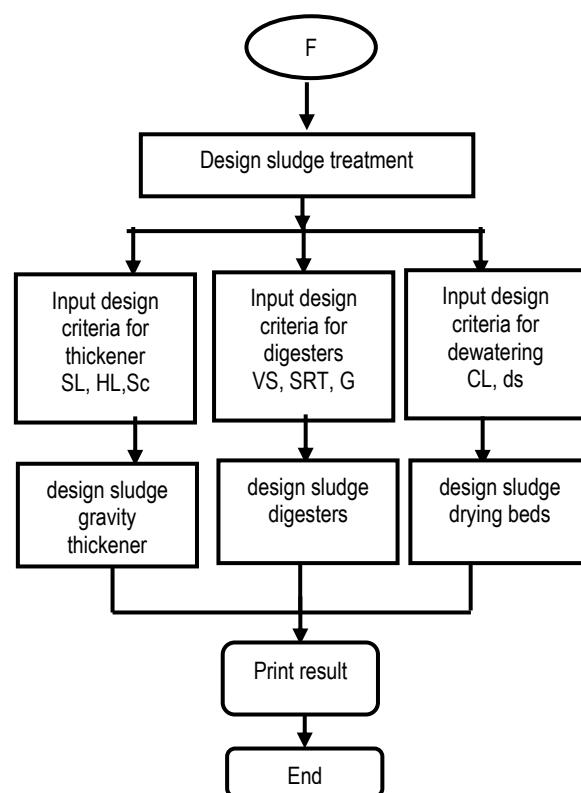
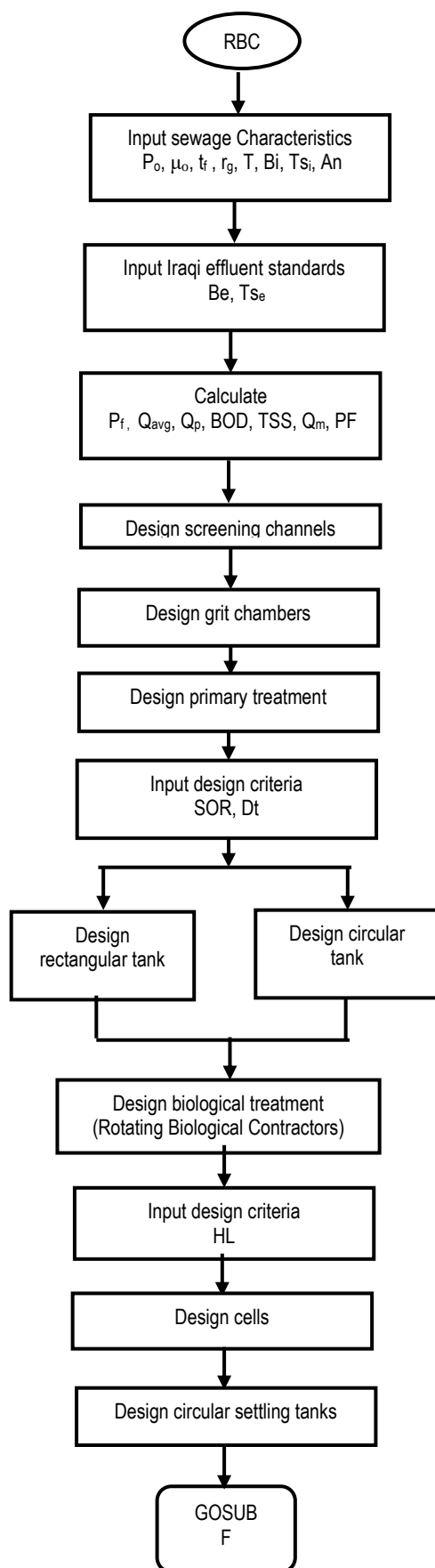












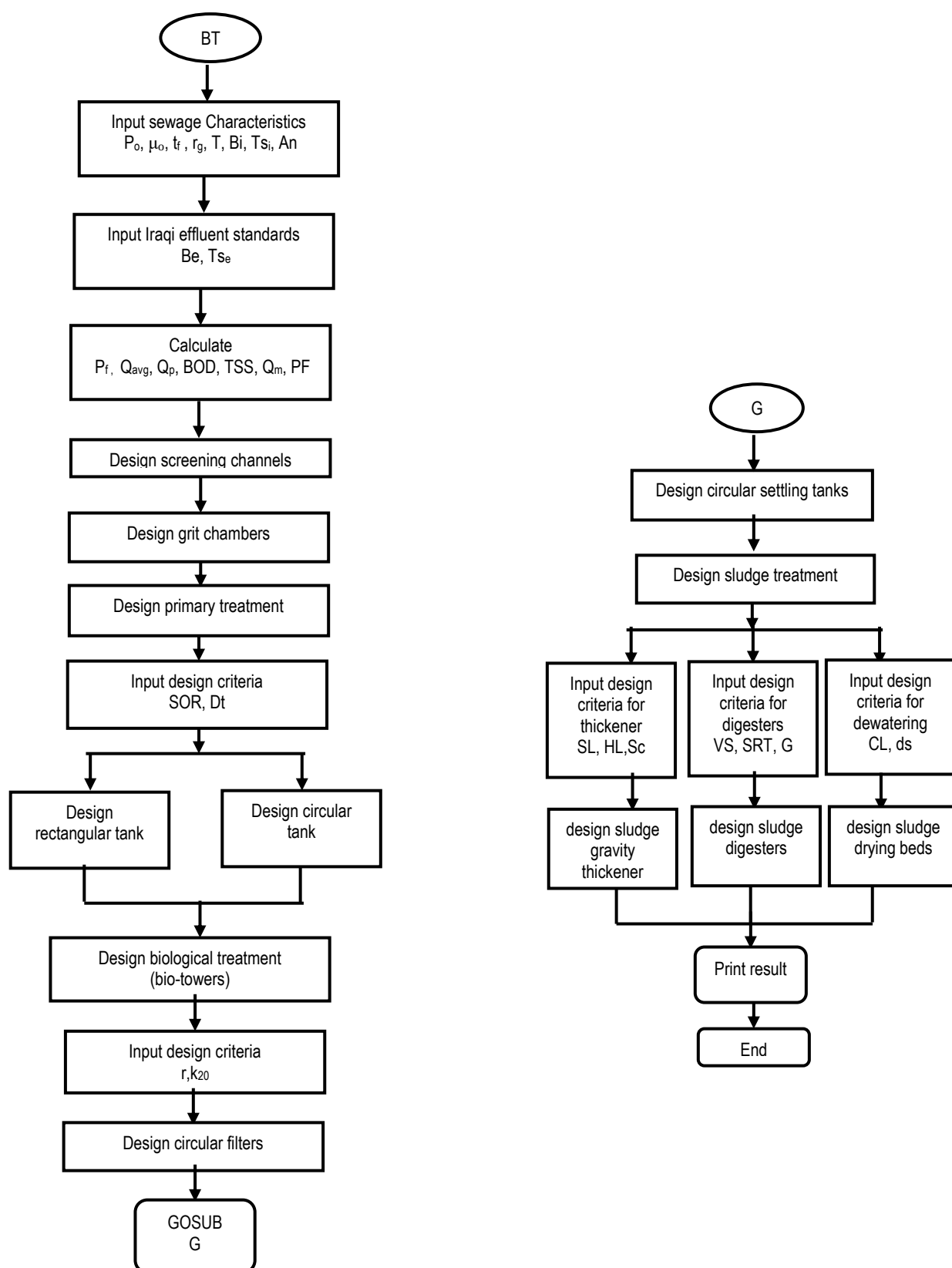


Fig.(3.1): Flowchart for Computer Program of Designing the Types of Wastewater Treatment Plants in The Present Study

3.3 Application of Computer Program for Study Treatment Plant

The computer program consists three main parts, which are:

1. The choice of biological treatment type as shown in Fig (3.2).
2. The information base which includes design period, initial population, sewage characteristics as shown in Fig. (3.3).
3. The design calculation modules which contain design requirement as shown in Figs.(3.4) to (3.16).

The effluent targeted in this design was to have 40 mg/l of BOD₅ or less and 60 mg/l of TSS or less.

Once the program is opened, the first page automatically appeared. At the first page, choose type of treatment as shown in Fig.(3.2). Then, user automatically go to the next page, this page is consisting of sewage characteristics as shown in Fig.(3.3) in this page all design data are entered, user can click the “DESIGN” button, and can go to the next page by clicking the “Next” button. In this page the outputs are consisting of average and peak flow rate as shown in Fig. (3.4). For other pages when all design data are entered, user can click the “DESIGN” button and the result automatically appeared. The steps should be followed for other pages similar with the first page until final design.

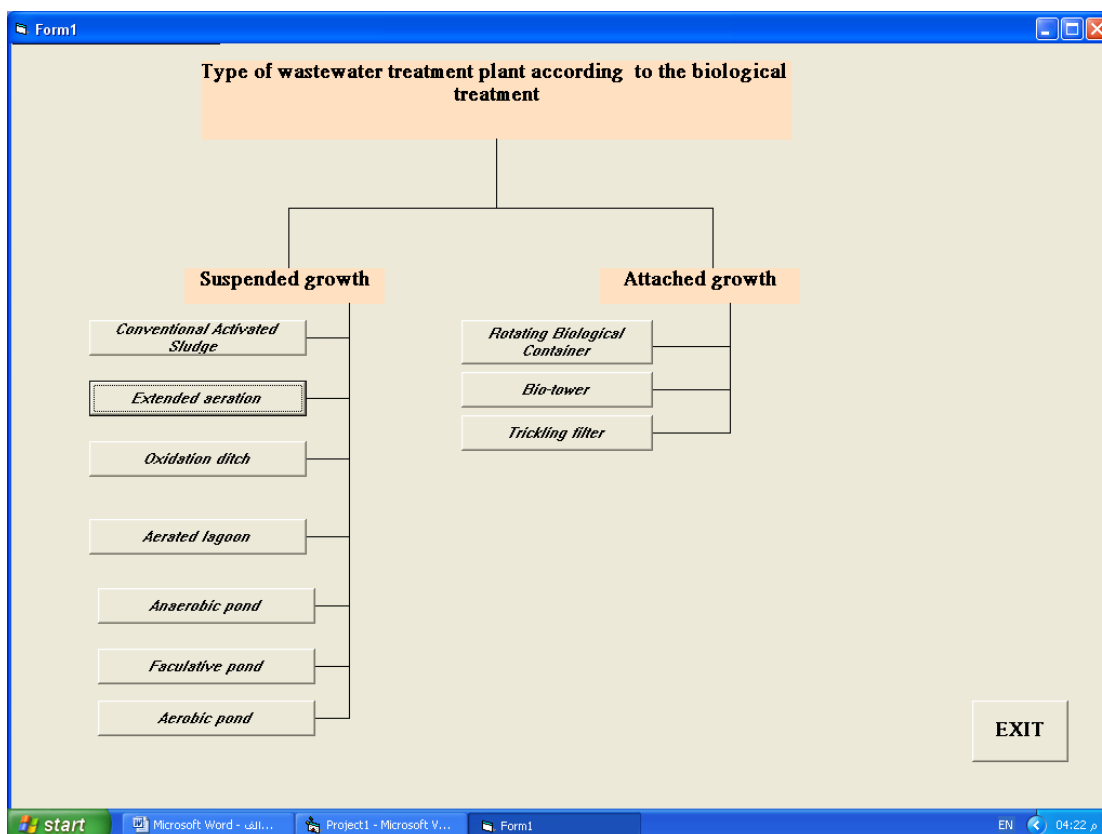


Fig.(3.2): Different types of Wastewater Treatment Plant of Present Study

General Information

Domestic Wastewater

Sewage production	270	l/c/day
Design Period	25	year
Initial population	100000	Capita
Growth rate	3.8	%
BOD5 production	70	g/c.day
TSS production	90	g/c.day
Infiltration Rate	0.1	l/s.ha
Area Served	400	ha

Temperature

Summer Temperature	40	C
Winter Temperature	18	C
Wastewater temperature	20	C

Iraqi Effluent Standard

BOD5	20	mg/l
TSS	30	mg/l

Other Characteristics

COD	125	g/c.day
Total Nitrogen	8	g/c.day
Total Phosphours	2	g/c.day
Settable Solids	20	mL/l
pH	7.2	-
Volatile Solids	75	%

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Fig.(3.3): General Information for Wastewater Treatment Plant

Influent Wastewater

Domestic Wastewater			Wastewater Characteristics		
Parameter	Value	Unit			
Total Design Population	109772	Capita	Domestic BOD concentration	259	mg/L
Average Design Flow	29638.44	m ³ /day	Domestic BOD production	7684	Kg/day
Peaking factor	2.19	-	Domestic TSS concentration	333	mg/L
Minimum factor	0.43	-	Domestic TSS production	9879	Kg/day
Peak Design Flow	65015.16	m ³ /day	COD	463	mg/L
Min.Design Flow	12991.14	m ³ /day	Total Nitrogen	30	mg/l
Infiltration flow rate	3456	m ³ /day	Total Phosphour	7	mg/l
Total Average Design Flow	33094.44	m ³ /day			
Total Peak Design Flow	68471.16	m ³ /day			

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Fig. (3.4): Wastewater Characteristics

Primary Treatment

General Information			Design Calculation		
Parameter	value	unit			
Specific Gravity	1.03	-	Number of units	2	units
Solids Concentration	4.5	z	Volume of Basin	2068.403	m ³
Wier Loading	186	m ³ /m.day	Width	10.87	m
L:W ratio	4	-	Length	43.48	m
Depth	3.5	m	Depth	4.1	m
Overflow rate	35	m/day	Diameter		m
Detention time	1.5	hour	Actual overflow Rate	35	m/day
TSS Removal	64	z	Actual Detention time	2.4	hour
BOD5 Removal	33	z	Wier length	88.96	m
COD Removal	35	z	Wier loading	152.2	m ³ /m.day
Settleable solids Removal	99	z	Head Over V- notch at Average flow	3.960112	cm
TN Removal	15	z	Head Over V- notch at peak flow	5.296739	cm
TP Removal	15	z	Average Quantity of Scum	0.27	m ³ /day
Spacing between Noches	20	cm	Number of V- notches	445	-
Quantity of Scum	8				

	Influent	Effluent	Sludge	Unit
Flow Rate	33094.4	32942.2	152.170	m ³ /day
BOD5	259	173.53	85.4699	mg/L
TSS	333	119.88	213.120	mg/L
COD	463	300.95	162.05	mg/L
Settleable solids	20	0.19999	19.8	ml/L
Total Phosphours	7	5.95	1.05	mg/L
Total Nitrogen	30	25.5	4.5	mg/L

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Fig. (3.5): Module for Design Rectangular Primary Sedimentation Tanks

Secondary Treatment ((Biological treatment --Compeletly mixed Activated sludge)).....

Parameter	value	unit
Mean Cell residence times	10	day
Depth	4.5	m
Length to Width Ratio	2	--
MLSS	3000	mg/L
Ratio of MLVSS/MLSS	0.8	z
MLVSS	2400	mg/l
Return Activated Sludge	10000	mg/L
Yeild coeeficient, Y	0.5	--
Decay Rate constant, kd	0.06	1/day
Fraction That Biodegrade	65	--
BOD5/BODu	0.68	--
Dissolved oxygen , DO	1.5	mg/L
Oxygen Transfer effieincy	8	z
Solubility of oxygenin tap water at 20 C	9.15	mg/L
Solubility of water at tap at field temperature	8.5	mg/l
Oxygen solubility correction factor	0.95	--

Design Calculation	value	unit
Number of Reactors	2	--
Volume of Reactors	9582.03	m3
Surface Area	1064.67	m2
Length	46.14	m
Width	23.07	m
Total Depth	4.5	m
Aeration Period	6.54	hour
F/M ratio	0.32	KgBOD/day
Organig Loading	0.81019	1/day
Soluble BOD5	11.5136	mg/L
Biological Treatment Efficiency	94.79	z
Over AllTreatment Efficiency	92.27	z
Net solids produced, PXss	2874.6118	kg/day
Oxygen transfer capability	12619.17	Kg O2/hp.hr
Air requirments	589.71	m3/min
Air /BOD	115.39	m3/Kg

	Influent	Effluent	Units
Design Flow Rate	33440.05	33440.05	m3/day
BOD5	210.5713	22.72	mg/L

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Fig. (3.6): Module for Design Rectangular Aerobic Reactor Basin

Secondary Treatment ((Biological treatment..Extended aetation))

Parameter	value	unit
Mean Cell residence times	20	day
Depth	3	m
Length to Width Ratio	2	--
MLSS	4000	mg/L
Ratio of MLVSS/MLSS	0.8	z
MLVSS	3200	mg/l
Return Activated Sludge	10000	mg/L
Yeild coeeficient, Y	0.7	--
Decay Rate constant, kd	0.07	1/day
Fraction That Biodegrade	63	--
BOD5/BODu	0.68	--
TKN	480	mg/L
Dissolved oxygen , DO	1.5	mg/L
Oxygen Transfer effieincy	8	z
Solubility of oxygenin tap water at 20 C	9.15	mg/L
Solubility of water at tap at field temperature	8.5	mg/L
Oxygen solubility correction factor	0.95	--

Design Calculation	value	unit
Number of Reactors	7	--
Volume of Reactors	26867.57	m3
Surface Area	1279.4	m2
Length	50.58	m
Width	25.29	m
Total Depth	3	m
Aeration Period	11.13	hour
F/M ratio	0.171428	KgBOD/day
Organig Loading	0.558295	1/day
Soluble BOD5	4.51	mg/L
Biological Treatment Efficiency	98.25	z
Over AllTreatment Efficiency	92.27	z
Net solids produced, PXss	5373.51	kg/day
Oxygen transfer capability	17422.62	kg/day
Air requirments	1359.66	kg/day

	Influent	Effluent	sludge	Units
Design Flow Rate	33094.44	32664.56	429.881	m3/day
BOD5	259	19.63	239.368	mg/L
TSS	333	92.78233	240.21	mg/L

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Fig. (3.7): Module for Design Rectangular Extended Aeration

Secondary Treatment ((Biological treatment..Oxidation ditch))

Parameter	value	unit
Mean Cell residence times	20	day
Depth	2.5	m
Length to Width Ratio	3	
MLSS	4000	mg/L
Ratio of MLVSS/MLSS	0.8	--
MLVSS	3200	mg/l
Oxygen Transfer efficiency	8	%
Yield coefficient, Y	0.5	--
Decay Rate constant, kd	0.06	1/day
Fraction That Biodegrade	0.68	--
BOD5/BODu	1.42	--
sludge loading, Lds	0.1	kg BOD/kg sludge per day
sludge volume index	100	mg/l
O2 supply to BOD load	2	--
O2 supply per KW.H	1.5	--
SRT	20	day
Solubility of oxygen in tap water at 20 C	9.15	mg/l
Dissolved oxygen, DO	1.5	mg/l
Solubility of water at tap at field temperature	8.5	mg/l
Oxygen solubility correction factor	0.95	--

Design Calculation			
Number of Ditches	8	--	
Volume of Ditches	37500.13	m3	
Surface Area	1875	m2	
Length	75	m	
Width	25	m	
HRT	15.53	hr	
Return Activated Sludge	10000	mg/L	
oxygen supply	30000.1	kg/d	
Biological Treatment Efficiency	97.05	%	
Over All Treatment Efficiency	92.27	%	
Energy required	20000.07	Kw.d	
Air requirements	2341.21	m3/d	

	Influent	Effluent	Sludge	Units
Design Flow	33094.4	32344.43	750	m3/day
BOD5	259	12.04	246.95	mg/L
TSS	333	71.4	261.59	mg/L

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Fig. (3.8): Module for Design Rectangular Oxidation Ditch

Secondary Treatment ((Biological treatment..Aerated lagoon))

Parameter	value	unit
Mean Cell residence times	5	day
Depth	4.5	m
Length to Width Ratio	3	--
Yield coefficient, Y	0.7	--
Decay Rate constant, kd	0.07	1/day
Proportionality factor, f	0.5	--
Overall first-order BOD5 removal-rate constant, K	6	1/day
substrate concentration @one-half the maximum growth rate, Ks	100	mg/L
first-order soluble BOD5 removal-rate constant, K20	2.5	1/day
Dissolved oxygen, DO	1.5	mg/L
Oxygen Transfer efficiency	8	%
Solubility of oxygen in tap water at 20 C	9.15	mg/L
Solubility of water at tap at field temperature	8.5	mg/L
Oxygen solubility correction factor	0.95	--

Design Calculation			
Number of Lagoons	34	--	
Volume of Lagoons	289576.3	m3	
Surface Area	64350.29	m2	
Length	75.35	m	
Width	25.11	m	
Total Depth	5.1	m	
Soluble BOD5	6.87	mg/L	
Biological Treatment Efficiency	97.34	%	
Over All Treatment Efficiency	92.27	%	
Net solids produced, PXss	7571.49	kg/day	
Oxygen requirements	18452.8	kg/day	
Air requirements	1440.05	m3/min	

	Influent	Effluent	Units
Design Flow Rate	33094.44	33094.44	m3/day
BOD5	259	12.98	mg/L

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Fig. (3.9): Module for Design Rectangular Aerated Lagoon

Secondary Treatment ((Biological treatment --Anaerobic pond)).....

Parameter	value	unit
Depth	2.5	m
Retention time	1.5	d
Legth/Width ratio	3	-
wastewater tempreture	20	c

parameter	value	unit
Number of ponds	4	-
Volume of Ponds	74386.5	m ³
Area of the Ponds	29754.6	m ²
Width	49.79	m
Length	149.38	m
Actual Retention time	1.5	d
Volumatric loading	124.02	g/m ³ .d

	influent	effluent	units
Flow rate	33094.4	33094.4	m ³ /day
BOD5	186.03	61.96	mg/L

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Fig. (3.10): Module for Design Rectangular Anaerobic Pond

Secondary Treatment ((Biological treatment --Faculative pond)).....

Parameter	value	unit
Depth	1.5	m
Reaction coefficient, K ₂₀	0.23	1/day
Length/Width ratio	3	-
Dispersion factor	0.5	-
Oxygen/BOD5	2	Kg/Kg
Mean Cell residence times	18.24	d

parameter	value	unit
Number of ponds	36	-
Volume of ponds	904617.3	m ³
Area of the ponds	603078.2	m ²
Width	74.72	m
Length	224.17	m
Oxygen supply	16467.52	g/d
Aerator transfer	686.14	g/m ³ .d
Over all effesincy	89.24	%

	influent	effluent	units
Flow rate	33082.2	33082.2	m ³ /day
BOD5	186.03	20	mg/L

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Fig. (3.11): Module for Design Rectangular Facultative Pond

Secondary Treatment ((Biological treatment --Aerobic pond)).....

Parameter	value	unit
Energy utilization efficiency	0.04	-
Locatid @	30	
Length/Widt ratio	3	-
Elevation above	50	ft
Mean percent of total time sun above horizontal	0.44	-
r	0.5	-
Month	Nov	
depth	1	

parameter	value	unit
Number of ponds	6	-
Volume of ponds	617098.4	m3
Area of the ponds	617098.4	m2
Width	453.54	m
Length	1360.62	m
Actual depth	0.24	m
Retention time	12.44	day
Biological Efficiency	89.24	%

	influent	effluent	units
Flow rate	33060.6	33060.6	m3/day
BOD5	186.033	20	mg/L

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Fig. (3.12): Module for Design Rectangular Aerobic Pond

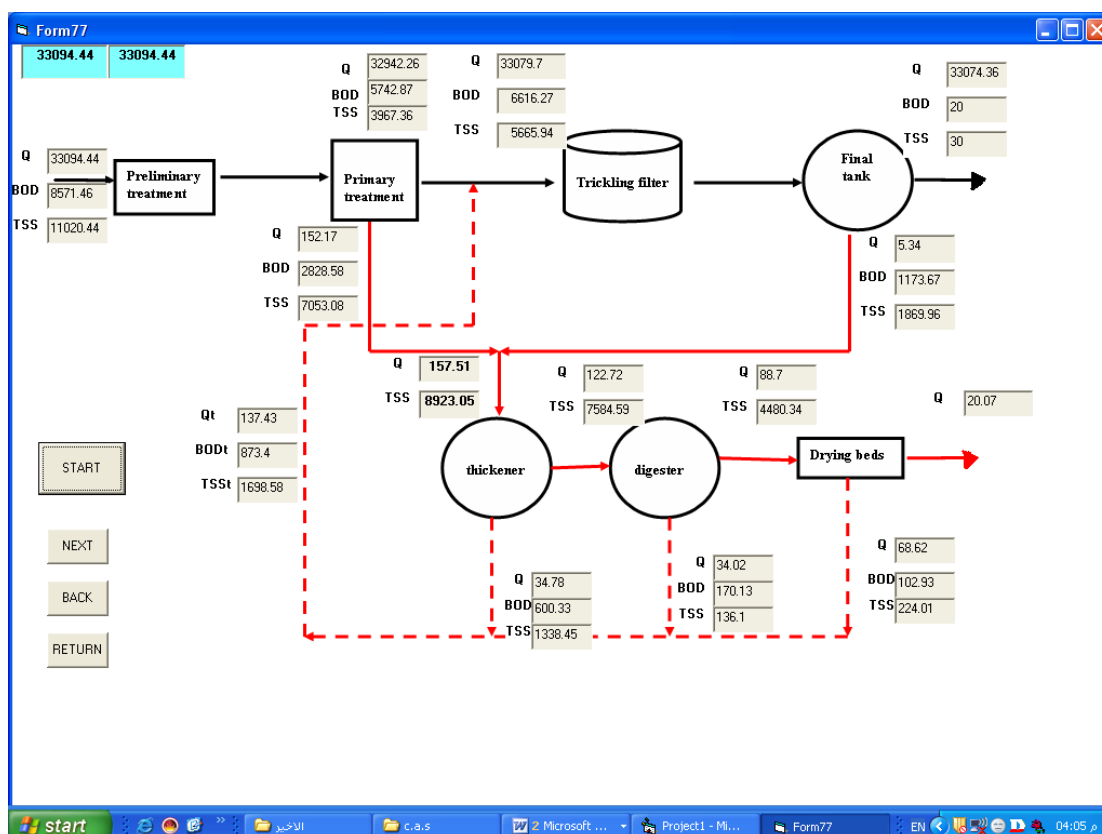


Fig. (3.13): Module for Mass Balance For Trickling Filter

Secondary Treatment ((Biological treatment..Trickling filter))

Design Information

Recirculation Ratio (r) -

Recirculation factor (F) -

Space between nodels m

Velocity through each nodel m/sec

Depth m

Design Calculation

parameter	value	unit
Number of unit	4	-
Total area	5602.4	m ²
Total Volume	16807.22	m ³
Diameter	42.22	m
Total efficiency	90	-
Efficiency of first stage	75	-
Efficiency of second stage	59.99	-
BOD loading rate	295.37	g/m ³ .d
Hydraulic loading rate	17.72	m ³ /m ² .d
Organic loading rate	0.39	KgBOD/m ³ .d
Total number of nodels per unit	161	-
Flow through each nodel	154.09	m ³ /day
Diameter of nodel	0.06	m

Influent Effluent Unit

Flow Rate	33079.7	33079.7	m ³ /day
BOD5	200.009	20	mg/L

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Fig. (3.14): Module for Design Trickling Filter

Secondary Treatment ((Biological treatment --Bio-disc)).....

Parameter value unit

Depth m

space of disc m

Ka -

Hydraulic loading m³/m².d

number of cells per basin -

Diameter of disc m

Sludge/BODremoved -

Design Calculation

parameter	value	unit
Number of tank	20	-
Number of disc	1200	-
Area of tank	187.5	m ²
volume of tank	11250	m ³
Length of tank	15	m
Width of tank	12.5	m
Surface loading of tank	15.43	m ³ /m ² .d
Hydraulic loading of disc	0.17	m ³ /m ² .d
Net solids produced	6947.08	Kg/d
Biological Treatment Efficiency	59.47	%
Over All Treatment Efficiency	90	%

Influent Effluent Units

Design Flow Rate	33079.70	33079.70	m ³ /day
BOD5	200.01	81.05	mg/L

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Fig. (3.15): Module for Design Rotating Biological Contactor.

Secondary Treatment ((Biological treatment - Bio-tower)).....

Design Information			Design Calculation		
parameter	value	unit	parameter	value	unit
Recirculation Ratio (r)	2	-	Number of unit	9	-
Treatability constant, k20	0.055	1/min	Total area	86.36	m ²
Space between nodels	0.15	m	Total Volume	561.36	m ³
Velocity through each nodel	0.6	m/sec	Diameter	3.49	m
Depth	6.5	m	Total efficiency	90	%
			Hydraulic loading rate	0.26	m ³ /m ² .d
			Total number of nodels per unit	40	-
			Flow through each nodel	91.888	m ³ /day
			Diameter of nodel	4.7518	cm

	Influent	Effluent	Unit
Flow Rate	33079.7	33079.7	m ³ /day
BOD5	200.009	20	mg/L

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Fig. (3.16): Module for Design Bio-Tower.

3.4 Verification of Computer Program Results

In the present work, a comparison was conducted between the running of program results with these results of hand calculations in order to get verification of the present models because the field data are not available. More than one factor could be taken for comparison but the population had been used because it is the most important factor. Figs. (3.17 to 3.48) show there are a difference in results with calibration equation which is indicates that there is a very good agreement Characteristics of sewage:

- Population = (60000-100000) capita.
- Specific sewage contribution = 270 l/c.d.
- Specific BOD production = 70 gpcd.
- Specific TSS production = 90 gpcd.

- Area served by network or = 400 hectare.
 - Infiltration rate = 0.1 l/s.ha.
 - Sewage temperature = 20 °C.
 - Design period = 25 year.
 - Dispersion factor = 0.5
1. The standard of effluent BOD and TSS in Iraq are 40 mg/l and 60 mg/l respectively.

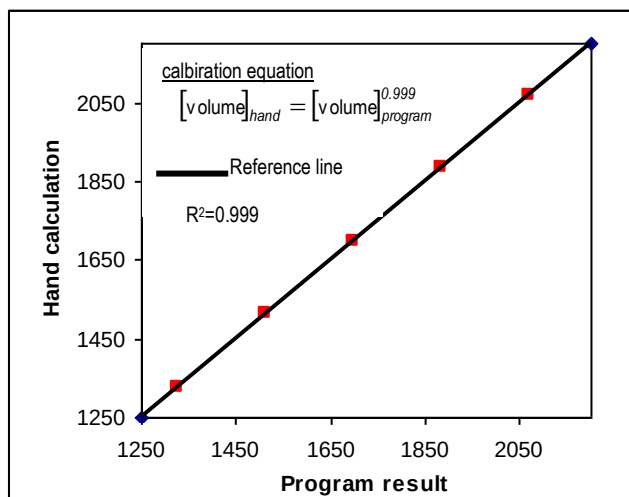


Fig. (3.17): Verification for Volume of Primary Clarifier

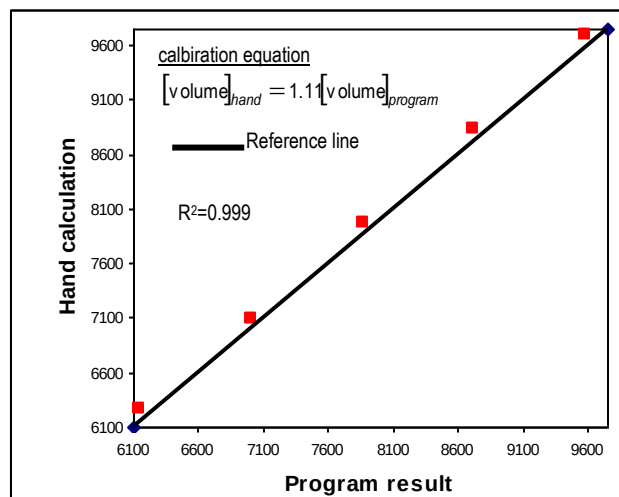


Fig. (3.18): Verification for Volume of Aerobic Reactors

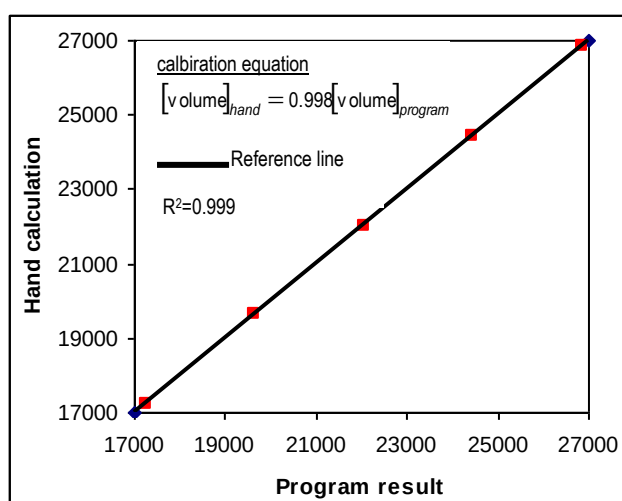


Fig. (3.19): Verification for Volume of Extended Aeration

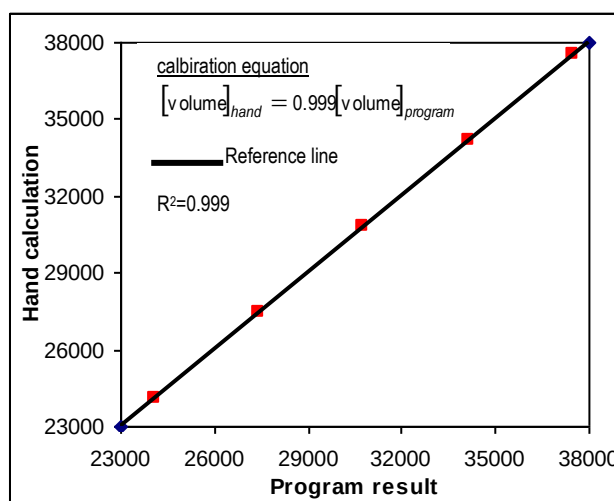


Fig. (3.20): Verification for Volume of Oxidation Ditch

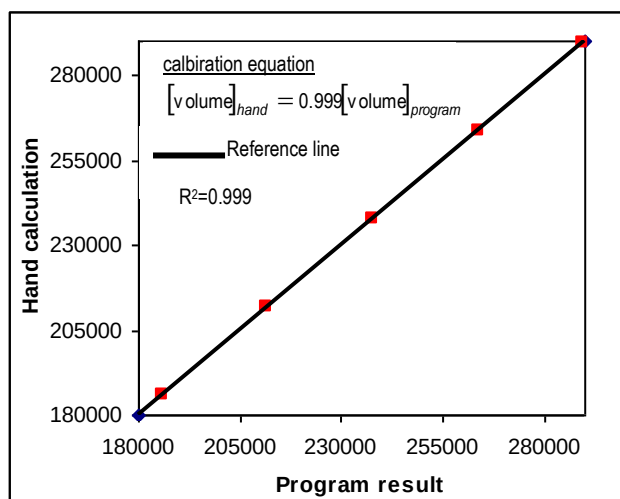


Fig. (3.21): Verification for Volume of Aerated Lagoon

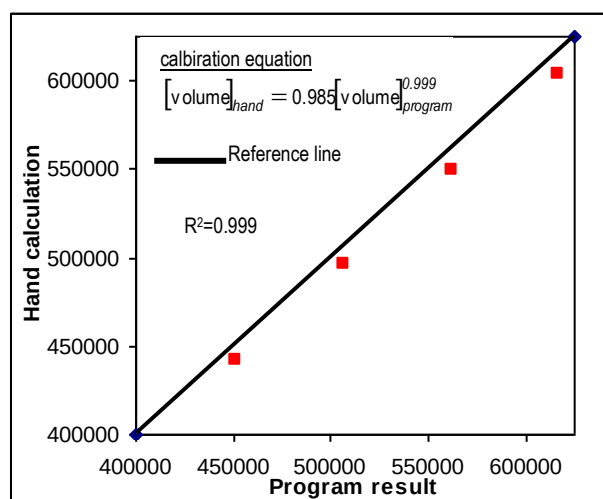


Fig. (3.22): Verification for Volume of Aerobic Pond

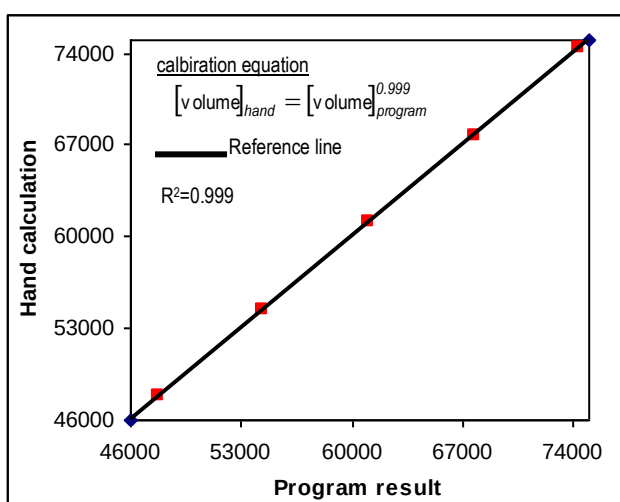


Fig. (3.23): Verification for Volume of Anaerobic Pond

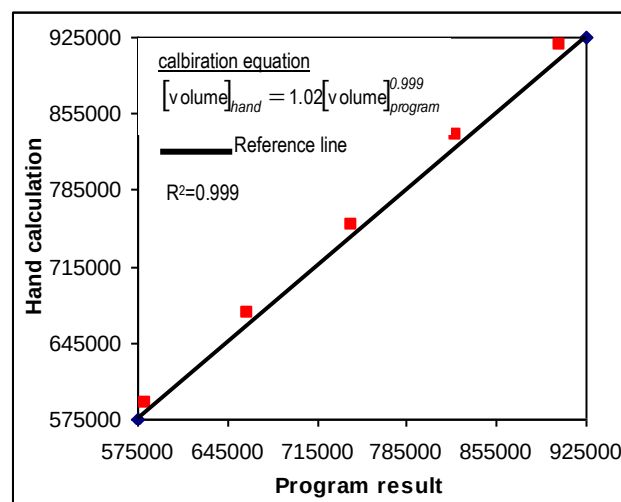


Fig. (3.24): Verification for Volume of Facultative Pond

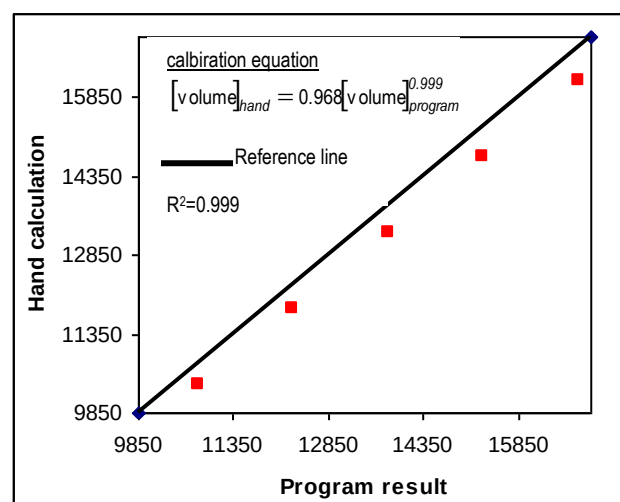


Fig. (3.25): Verification for Volume of Trickling Filter

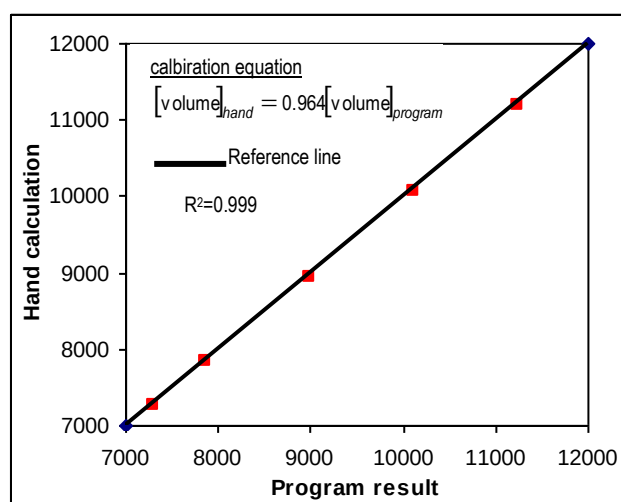


Fig. (3.26): Verification for Volume of Rotating Biological Contactor

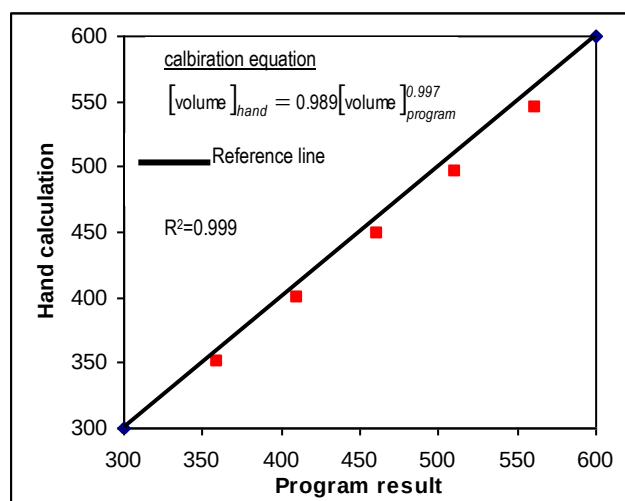


Fig. (3.27): Verification for Volume of Bio-Tower

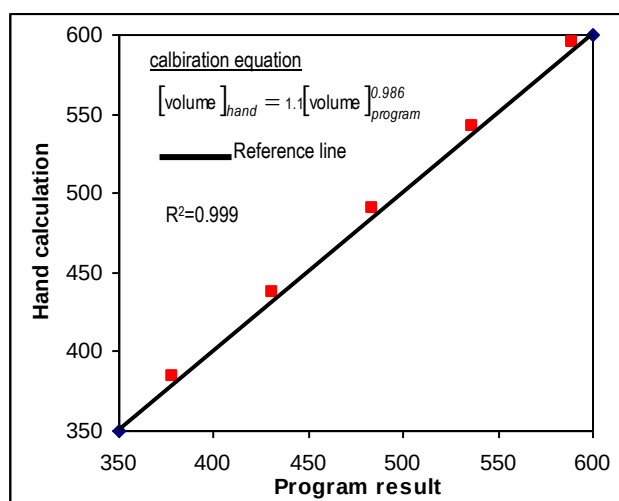


Fig. (3.28): Verification for Volume of Air required for Conventional Activated Sludge

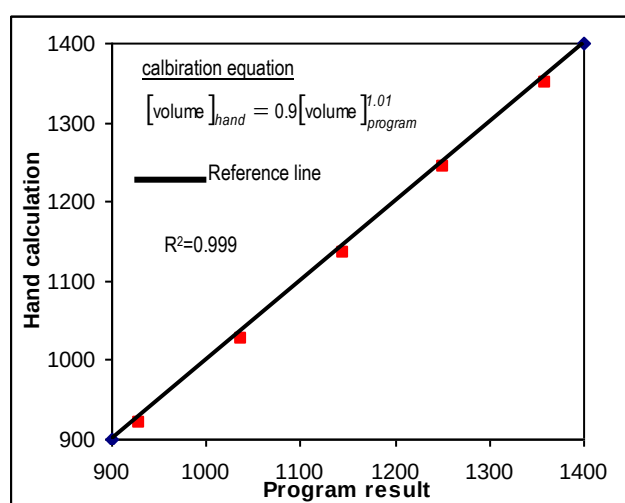


Fig. (3.29): Verification for Volume of Air required for Extended Aeration

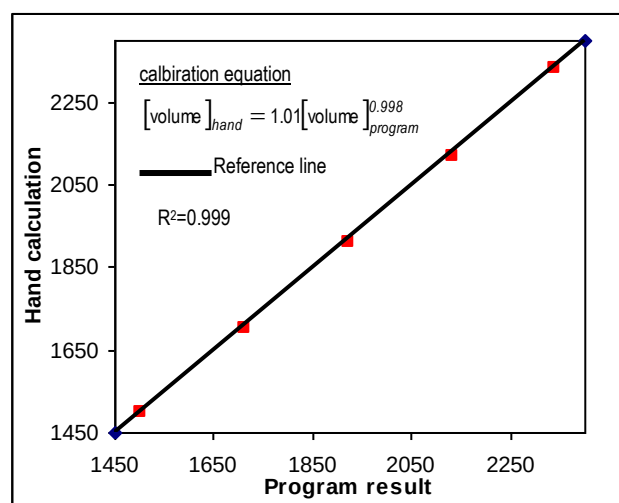


Fig. (3.30): Verification for Volume of Air required for Oxidation Ditch

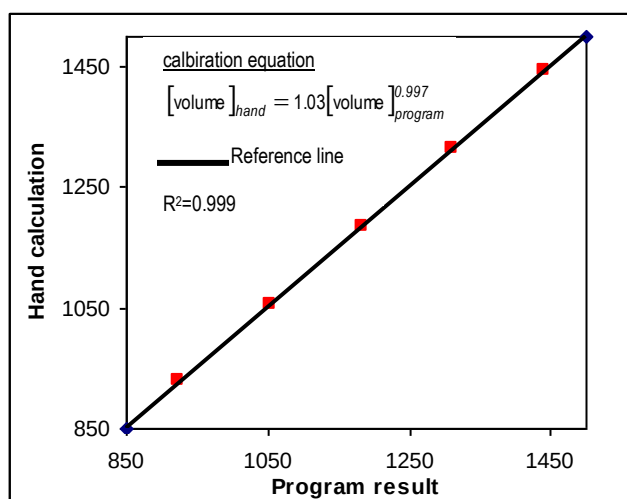


Fig. (3.31): Verification for Volume of Air required for Aerated Lagoon

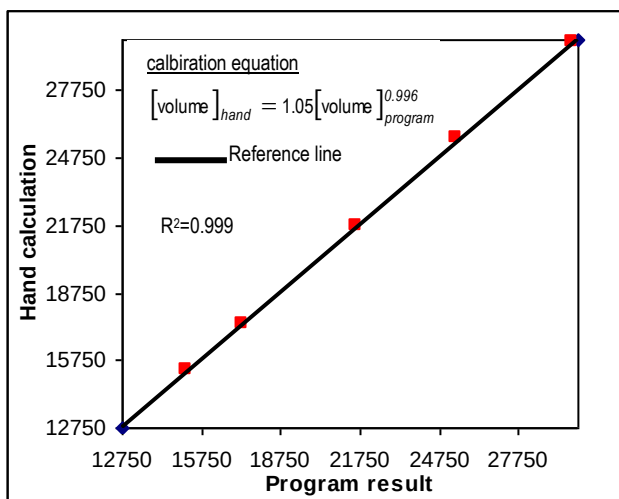


Fig. (3.32): Verification for Volume of Secondary Clarifier for Conventional Activated Sludge

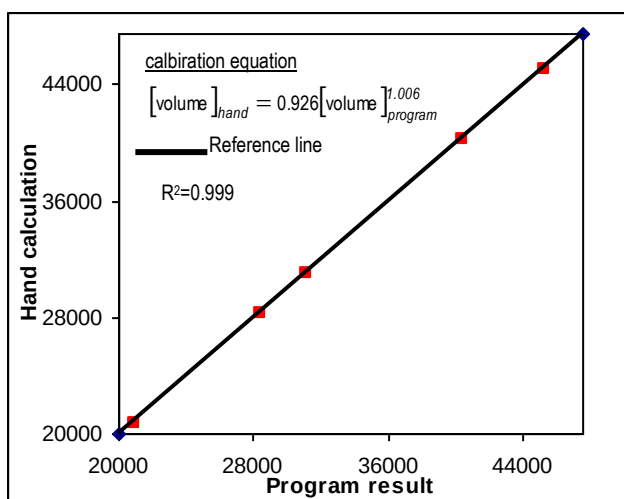


Fig. (3.33): Verification for Volume of Secondary Clarifier for Extended Aeration

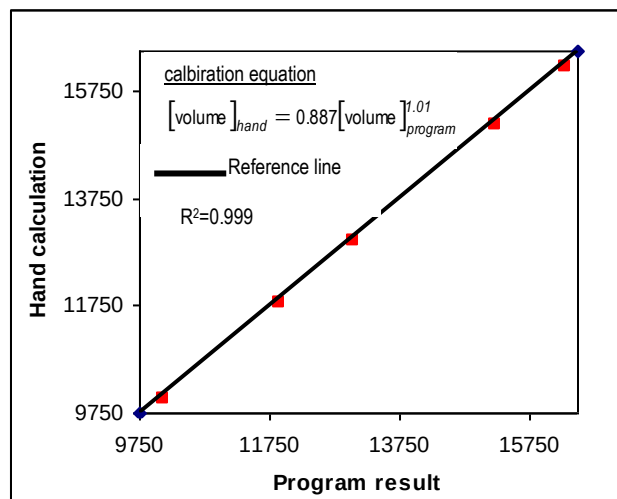


Fig. (3.34): Verification for Volume of Secondary Clarifier for Oxidation Ditch

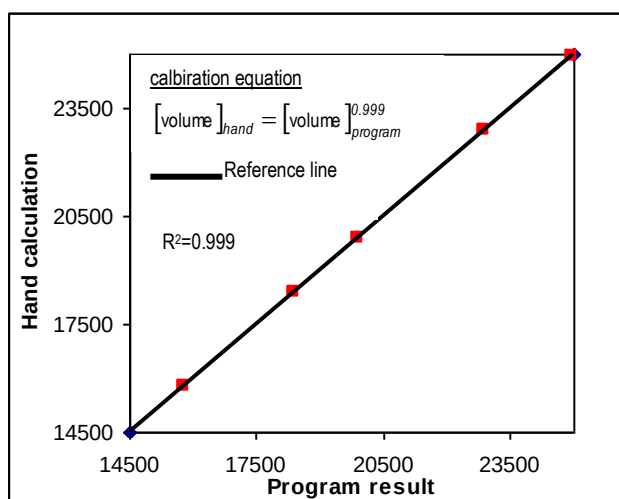


Fig. (3.35): Verification for Volume of Secondary Clarifier for Aerated Lagoon

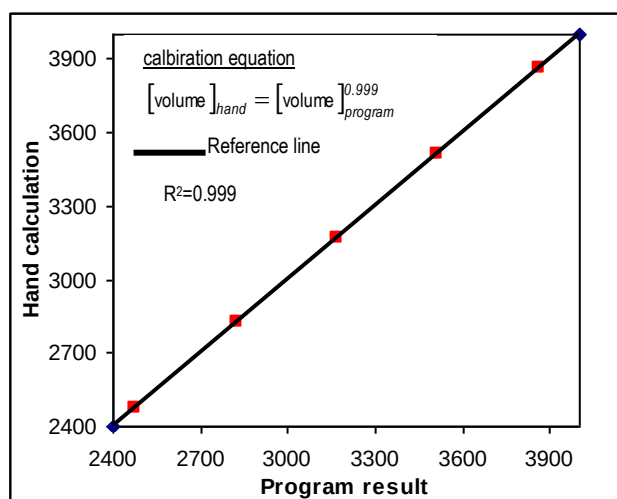


Fig. (3.36): Verification for Volume of Secondary Clarifier for Attach-Growth System

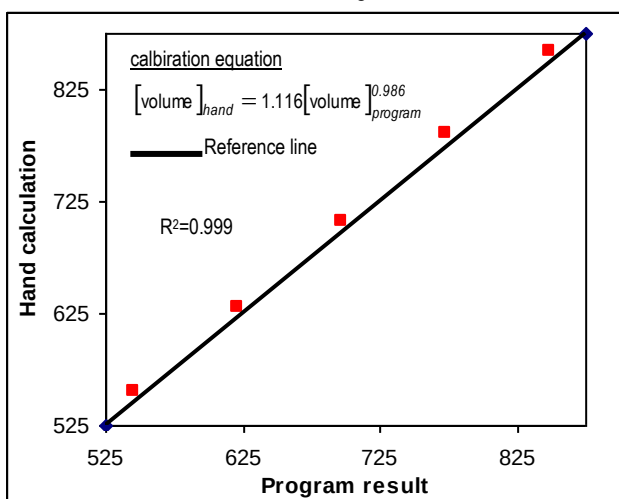


Fig. (3.37): Verification for Volume of Thickener for Conventional Activated Sludge

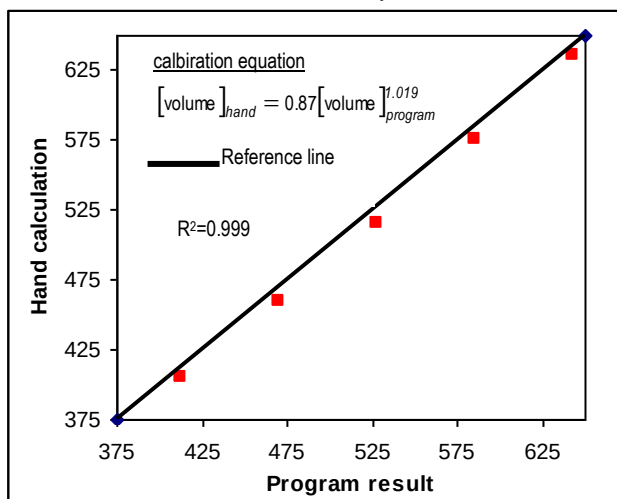


Fig. (3.38): Verification for Volume of Thickener for Stabilization pond

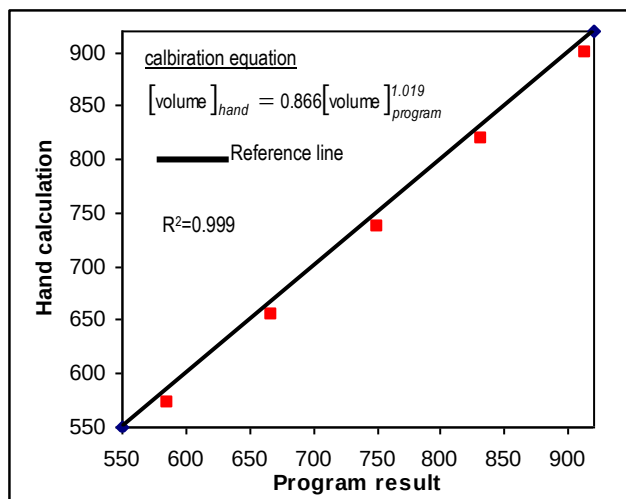


Fig. (3.39): Verification for Volume of Thickener for Attach-Growth System

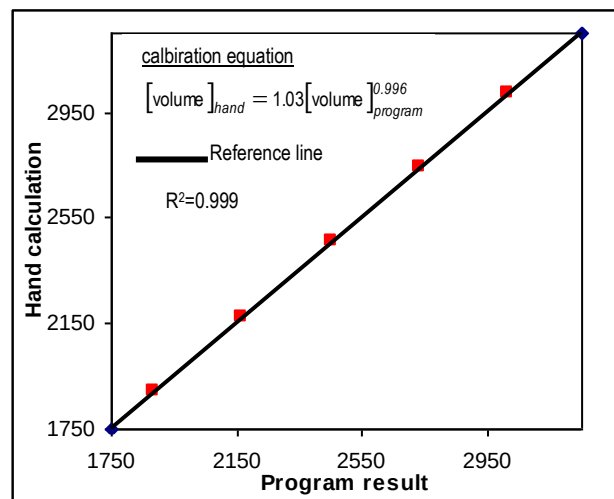


Fig. (3.40): Verification for Volume of Digester for Conventional Activated Sludge

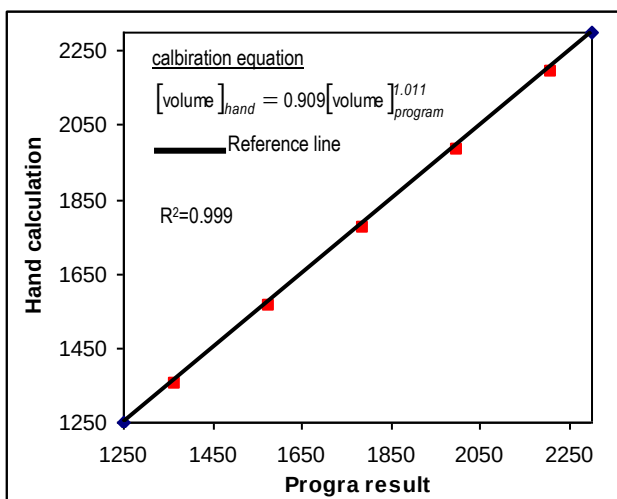


Fig. (3.41): Verification for Volume of Digester for Stabilization pond

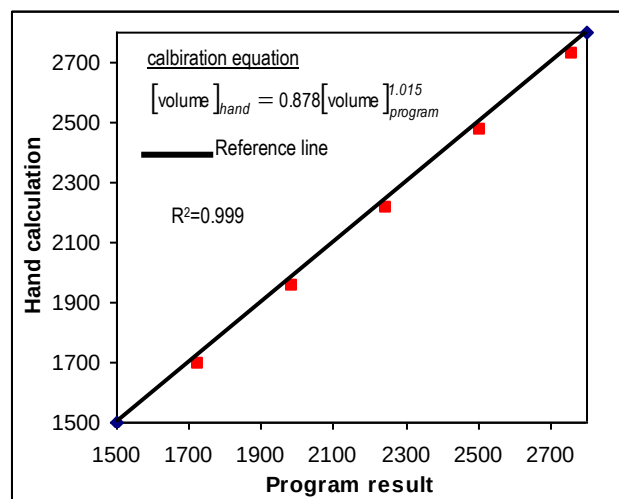


Fig. (3.42): Verification for Volume of Digester for Attach-Growth System

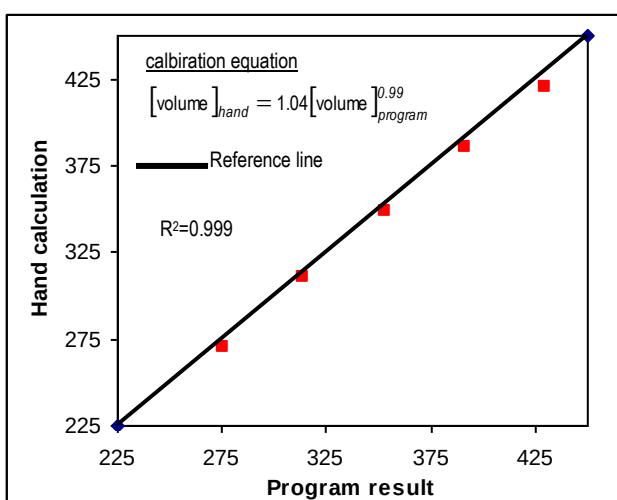


Fig. (3.43): Verification for Volume of Holding Tank for Extended Aeration

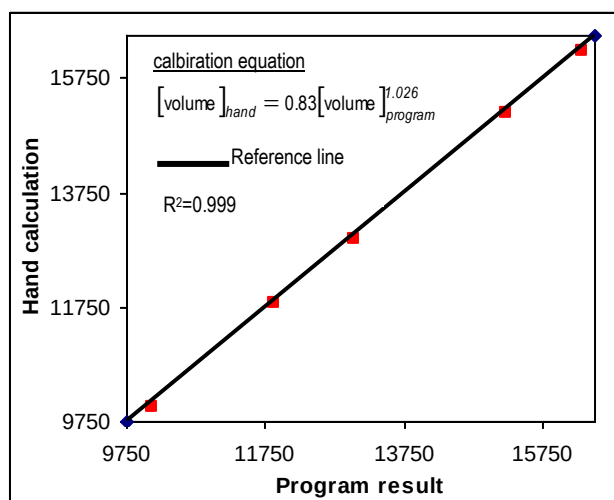


Fig. (3.44): Verification for Volume of Holding Tank for Oxidation Ditch

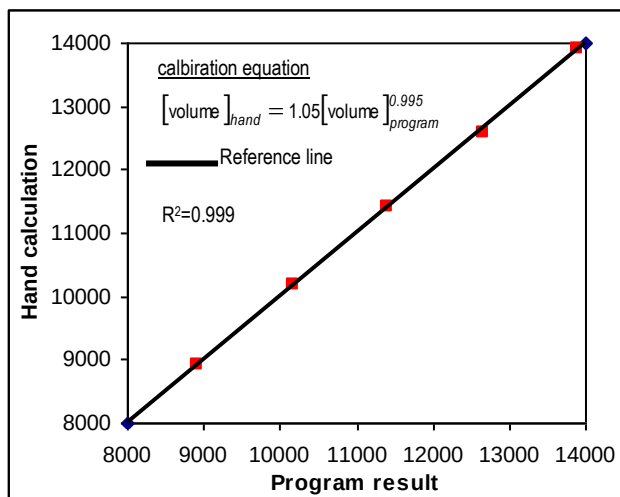


Fig. (3.45): Verification for Volume of Holding Tank for Aerated Lagoon

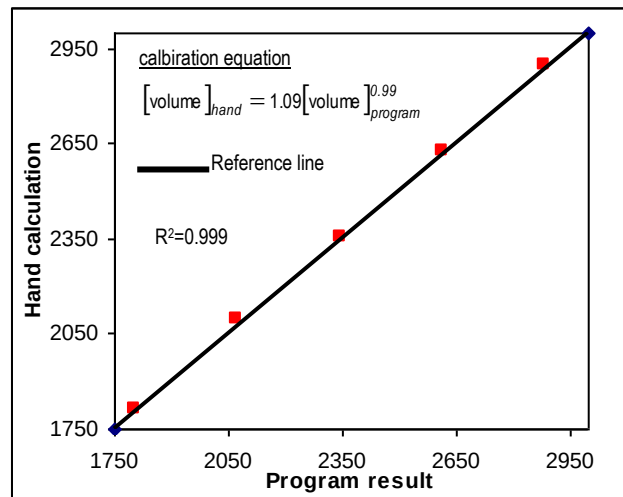


Fig. (3.46): Verification for Volume of Gas Produced for Conventional Activated Sludge

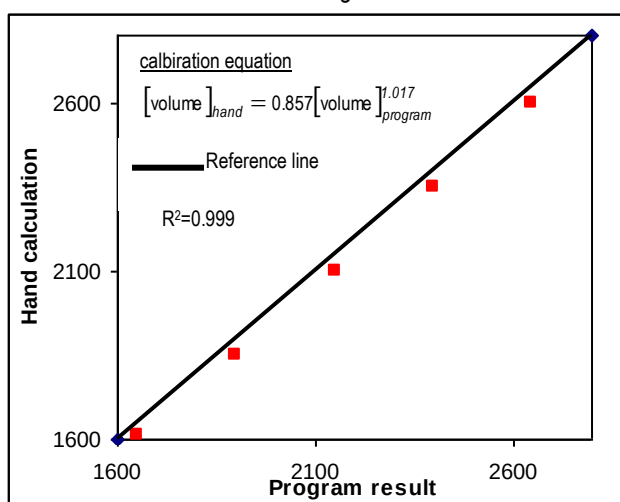


Fig. (3.47): Verification for Volume of Gas Produced for Stabilization pond

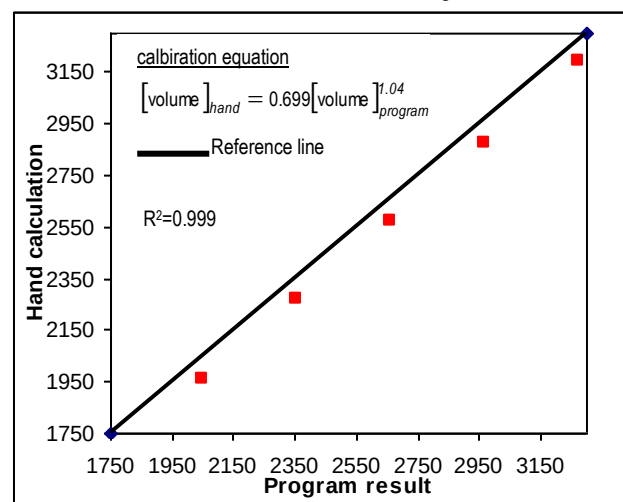


Fig. (3.48): Verification for Volume of Gas Produced for Attach-Growth System

Chapter Four

Analysis of Results

4.1 Introduction

In this chapter the statistical models which are described the relations between the factors that affecting the design of the studied wastewater treatment plants were established. The effects of different parameters on the behavior of the model, and hence on the design of WWTP_s are considered as well.

4.2 The Regression Analysis Technique

The regression analysis were done by using " Data Fit" program version 8.0 (<http://www.oakdaleengr.com>).

The coefficient of determination (R^2) reflects the overall accuracy of any predictive model ⁽⁵³⁾.

4.2.1 The Dependent Variables (y)

In the present work, the volume of each treatment unit, quantity of total air required for aerobic reactors, and volume of gas production were assumed to be the dependent variables (y).

4.2.2 The Independent Variables

Multiple independent variable (also named as explanatory variables or predictors) means that there is more than one independent variables ($x_1, x_2, \dots, x_k$) and a single dependent variable (y), in the regression models. Table (4.1) shows these variables.

Table (4.1): Independent variables for the present study

Variable	Description
X ₁	Population factor, capita
X ₂	Temperature, °C
X ₃	Specific sewage production, l/c.d
X ₄	TSS production, g/c.d
X ₅	BOD production, g/c.d
X ₆	Area served by network(catchment area), ha
X ₇	Infiltration rate, l/s.ha
X ₈	Design period, y
X ₉	Recirculation ratio
X ₁₀	Dispersion factor

Table (4.2) shows the independent variables affecting the biological units for each wastewater treatment plants.

Table (4.2): Independent variables for the biological units of each wastewater treatment plants

Types of wastewater treatment plants	independent variables
Conventional Activated Sludge	X ₁ , X ₂ , X ₃ , X ₄ , X ₅ , X ₆ , X ₇ , X ₈
Extended Aeration	X ₁ , X ₂ , X ₃ , X ₅ , X ₆ , X ₇ , X ₈
Oxidation Ditch	X ₁ , X ₂ , X ₃ , X ₅ , X ₆ , X ₇ , X ₈
Aerated lagoon	X ₁ , X ₃ , X ₆ , X ₇ , X ₈
Aerobic pond	X ₁ , X ₃ , X ₄ , X ₆ , X ₇ , X ₈
Anaerobic Pond	X ₁ , X ₃ , X ₄ , X ₆ , X ₇ , X ₈
Facultative Pond	X ₁ , X ₂ , X ₃ , X ₄ , X ₅ , X ₆ , X ₇ , X ₈ , X ₁₀
Trickling Filter	X ₁ , X ₂ , X ₃ , X ₄ , X ₅ , X ₆ , X ₇ , X ₈ , X ₉
Rotating Biological Contactor	X ₁ , X ₂ , X ₃ , X ₄ , X ₅ , X ₆ , X ₇ , X ₈
Bio-Tower	X ₁ , X ₂ , X ₃ , X ₄ , X ₅ , X ₆ , X ₇ , X ₈ , X ₉

4.3 Regression Models for Designing Wastewater Treatment Plants

In the present study, multiple non-linear regression models in three forms were used for each one of design requirements to choose which form gives the best fitting of data.

The regression models that were proposed and investigated can be seen in table (4.3).

Table (4.3): The Proposed Models

No.	Equation Description
A	$y = b_1x_1 + b_2x_2 + \dots + b_kx_k + G$
B	$y = \exp(b_1x_1 + b_2x_2 + \dots + b_kx_k + G)$
C	$y = b_1x_1 + b_2x_2 + \dots + b_kx_k$

Where;

y = dependent variables.

$x_1, x_2, \dots, x_k$ = the independent variables.

$b_1, b_2, b_3, \dots, b_k$ = are model coefficients, and

G = model constant term.

4.4 Results of Study

The result of present study can be seen in table (4.4).

Chapter Five

Conclusions And Recommendations

5.1 Conclusions

Based on result study the following points can be concluded:

- 1- A computer program for the design of wastewater treatment plants which are (conventional activated sludge, extended aeration, oxidation ditch, aerated lagoon, aerobic pond, anaerobic pond, facultative pond, trickling filter, rotating biological contactors, and bio-tower wastewater treatment plants) was developed considering the effect of the environmental factors. The verification between program results and those obtained by hand calculations for most important factor shows a good agreement.
- 2- The most appropriate significant independent variables are:

❖ Population:

It was found to be the most significant variable affecting design of all wastewater treatment units for all types of present wastewater treatment plants.

❖ Temperature:

It was found to be a significant variable that affecting on the models of volume of biological unit for (conventional activated sludge, extended aeration, oxidation ditches, facultative ponds, trickling filters, rotating biological contactors, and bio-towers), quantity of total air required for aeration process for plants that need aeration, quantity of gas produced, and volume of thickeners for conventional activated sludge and trickling filters.

❖ Sewage Contribution:

It significantly affects the models volume of final settling tanks and volume of aeration basins for all types of present wastewater treatment plants.

❖ TSS production:

It significantly affects the models (volume of thickeners, digesters volume, and quantity of gas generated) for all present wastewater treatment plants that found in it.

❖ BOD production:

It reliably affects the volume of total air required for all present wastewater treatment plants, volume of biological units for (conventional activated sludge, extended aeration, oxidation ditches, facultative ponds, trickling filters, rotating biological contactors, and bio-towers), and volume of holding tank for plants that need it.

❖ Area served by network :

It has a significance effects on design requirements.

❖ Infiltration rate:

It increases the plant influent flow and decrease the concentration of BOD in the sewage because the infiltration caused by the high water table and defects in the network pipes.

❖ Design period:

It has a significance effects on design requirements.

❖ Recirculation ratio:

It was found to be a significant variable on the models of volume of biological units for trickling filters and bio-towers.

❖ Dispersion factor:

It was found to be a significant variable on the models of volume of biological units for facultative ponds only.

5.2 Recommendations for Future studies:

Some further studies can be focusing on the following objectives:

1. The graphical user interface (GUI) does not mean a simple sketch to explain the shape of unit, but it is interface with AutoCAD program, that means after finishing the calculations for each units the user will obtain a drawing for each unit and final drawing to whole treatment plant.
2. Hydraulic design of municipal wastewater treatment plants.
3. Factors affecting the choice of industrial wastewater treatment plants.
4. The environmental effects (gases emissions, insects, odor, pathogenic, noises and other nuisance effects) of each type of treatment.
5. Cost analysis (construction cost) for all units of treatment plant includes liquid system and sludge system with more details of estimating materials and equipments.

Table (4.5): Correlation Matrix for Volume of Primary Sedimentation Tanks.

	X ₁	X ₃	X ₆	X ₇	X ₈	y
X ₁	1	0.014	-0.034	-0.036	-0.025	0.842
X ₃	0.014	1	0.025	0.026	0.018	0.451
X ₆	-0.034	0.025	1	-0.065	-0.045	0.165
X ₇	-0.036	0.026	-0.065	1	-0.048	0.198
X ₈	-0.025	0.018	-0.045	-0.048	1	0.076
y	0.842	0.451	0.165	0.198	0.076	1

Table (4.6): Correlation Matrix for Volume of Aerobic Reactors For Conventional Activated Sludge.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.426
X ₂	-0.020	1	0.015	0.015	0	-0.036	-0.038	-0.026	0.825
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.038
X ₄	0.008	0.015	-0.006	1	0	0.015	0.015	0.011	0.073
X ₅	0	0	0	0	1	2.14E-018	0	0	0.286
X ₆	-0.020	-0.036	0.015	0.015	2.14E-018	1	-0.038	-0.026	0.058
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.073
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.020
y	0.426	0.825	-0.038	0.073	0.286	0.058	0.073	0.020	1

Table (4.7): Correlation Matrix for Volume of Extended Aeration.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.022	0.009	0	-0.022	-0.024	-0.016	0.485
X ₂	-0.022	1	0.017	-2.67E-018	-0.041	-0.043	-0.030	0.760
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.024
X ₅	0	-2.67E-018	0	1	0	0	0	0.361
X ₆	-0.022	-0.041	0.017	0	1	-0.043	-0.030	0.068
X ₇	-0.024	-0.043	0.018	0	-0.043	1	-0.032	0.085
X ₈	-0.016	-0.030	0.012	0	-0.030	-0.032	1	0.023
y	0.485	0.760	-0.024	0.361	0.068	0.085	0.023	1

Table (4.8): Correlation Matrix for Volume of Oxidation Ditch.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.628
X ₂	-0.023	1	0.017	-2.674E-018	-0.041	-0.044	-0.030	0.569
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.032
X ₅	0	-2.674E-018	0	1	2.14E-018	0	0	0.452
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.105
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.128
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.043
y	0.628	0.569	-0.032	0.452	0.105	0.128	0.043	1

Table (4.9): Correlation Matrix for Volume of Aerated Lagoon.

	X ₁	X ₃	X ₆	X ₇	X ₈	y
X ₁	1	0.014	-0.034	-0.036	-0.025	0.841
X ₃	0.014	1	0.025	0.026	0.018	0.451
X ₆	-0.034	0.025	1	-0.065	-0.045	0.165
X ₇	-0.036	0.026	-0.065	1	-0.048	0.198
X ₈	-0.025	0.018	-0.045	-0.048	1	0.076
y	0.841	0.451	0.165	0.198	0.076	1

Table (4.10): Correlation Matrix for Volume of Aerobic Ponds.

	X ₁	X ₃	X ₄	X ₆	X ₇	X ₈	y
X ₁	1	0.011	0.011	-0.027	-0.029	-0.020	0.927
X ₃	0.011	1	-0.008	0.020	0.022	0.015	-0.119
X ₄	0.011	-0.008	1	0.020	0.021	0.015	0.038
X ₆	-0.027	0.020	0.020	1	-0.053	-0.037	0.178
X ₇	-0.029	0.022	0.021	-0.053	1	-0.039	0.214
X ₈	-0.020	0.015	0.015	-0.037	-0.039	1	0.082
y	0.927	-0.119	0.038	0.178	0.214	0.082	1

Table (4.11): Correlation Matrix for Volume of Anaerobic Ponds.

	X ₁	X ₃	X ₄	X ₆	X ₇	X ₈	y
X ₁	1	0.011	0.011	-0.027	-0.029	-0.020	0.842
X ₃	0.011	1	-0.008	0.020	0.022	0.015	0.446
X ₄	0.011	-0.008	1	0.020	0.021	0.015	0.016
X ₆	-0.027	0.020	0.020	1	-0.053	-0.037	0.173
X ₇	-0.029	0.022	0.021	-0.053	1	-0.039	0.206
X ₈	-0.020	0.015	0.015	-0.037	-0.039	1	0.082
y	0.842	0.446	0.016	0.173	0.206	0.082	1

Table (4.12): Correlation Matrix for Volume of Facultative Ponds.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	X ₁₀	y
X ₁	1	-0.017	0.007	0.007	0	-0.017	-0.018	-0.013	-0.012	0.576
X ₂	-0.017	1	0.013	0.013	0	-0.032	-0.033	-0.023	-0.021	0.319
X ₃	0.007	0.013	1	-0.005	0	0.013	0.014	0.010	0.009	0.092
X ₄	0.007	0.013	-0.005	1	0	0.013	0.014	0.009	0.009	0.098
X ₅	0	0	0	0	1	0	0	0	0	0.273
X ₆	-0.017	-0.032	0.013	0.013	0	1	-0.033	-0.023	-0.021	0.116
X ₇	-0.018	-0.033	0.014	0.014	0	-0.033	1	-0.025	-0.022	0.138
X ₈	-0.013	-0.023	0.010	0.009	0	-0.023	-0.025	1	-0.016	0.055
X ₁₀	-0.012	-0.021	0.009	0.009	0	-0.021	-0.022	-0.016	1	0.558
y	0.576	0.319	0.092	0.098	0.273	0.116	0.138	0.055	0.558	1

Table (4.13): Correlation Matrix for Volume of Trickling Filters.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	X ₉	y
X ₁	1	-0.017	0.007	0.007	0	-0.018	-0.019	-0.013	0.008	0.318
X ₂	-0.017	1	0.013	0.013	0	-0.033	-0.034	-0.024	0.015	0.719
X ₃	0.007	0.013	1	-0.005	0	0.013	0.014	0.010	-0.006	-0.031
X ₄	0.007	0.013	-0.005	1	0	0.013	0.014	0.010	-0.006	0.035
X ₅	0	0	0	0	1	-2.13E-018	1.8E-018	0	0	0.467
X ₆	-0.018	-0.033	0.013	0.013	-2.13E-018	1	-0.034	-0.023	0.015	0.035
X ₇	-0.019	-0.034	0.014	0.014	1.8E-018	-0.034	1	-0.025	0.016	0.046
X ₈	-0.013	-0.024	0.010	0.010	0	-0.023	-0.025	1	0.008	0.008
X ₉	0.008	0.015	-0.006	-0.006	0	0.015	0.016	0.011	1	0.134
y	0.318	0.719	-0.031	0.035	0.467	0.035	0.046	0.008	0.134	1

Table (4.14): Correlation Matrix for Volume of Rotating Biological Contactor.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.020	-0.014	0.306
X ₂	-0.020	1	0.015	0.015	0	-0.036	-0.038	-0.026	0.723
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.319
X ₄	0.008	0.015	-0.006	1	0	0.150	0.150	0.011	
X ₅	0	0	0	0	1	2.13E-018	0	0	0.475
X ₆	-0.020	-0.036	0.015	0.150	2.13E-018	1	-0.038	-0.026	0.029
X ₇	-0.020	-0.038	0.016	0.150	0	-0.038	1	-0.028	0.031
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	-0.002
y	0.306	0.723	-0.319	0.044	0.475	0.029	0.031	-0.002	1

Table (4.15): Correlation Matrix for Volume of Bio-Towers.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	X ₉	y
X ₁	1	-0.018	0.007	0.007	0	-0.018	-0.019	-0.013	0.008	0.192
X ₂	-0.018	1	0.013	0.013	0	-0.032	-0.032	-0.024	0.015	-0.306
X ₃	0.007	0.013	1	-0.005	0	0.013	0.014	0.01	-0.006	-0.087
X ₄	0.007	0.013	-0.005	1	0	0.013	0.014	0.01	-0.006	0.018
X ₅	0	0	0	0	1	-2.13E-018	1.8E-018	0	0	0.221
X ₆	-0.018	-0.032	0.013	0.013	-2.13E-018	1	-0.034	-0.024	0.015	0.024
X ₇	-0.019	-0.034	0.014	0.014	1.8E-018	-0.034	1	-0.025	0.016	0.031
X ₈	-0.013	-0.024	0.01	0.01	0	-0.024	-0.025	1	0.011	0.007
X ₉	0.008	0.015	-0.006	-0.006	0	0.015	0.016	0.011	1	-0.815
y	0.192	-0.306	-0.087	0.018	0.221	0.024	0.031	0.007	-0.815	1

Table (4.16): Correlation Matrix for Volume of Final Settling Tanks For Conventional Activated Sludge.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.020	-0.014	0.785
X ₂	-0.020	1	0.015	0.015	0	-0.036	-0.038	-0.026	0.468
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	0.192
X ₄	0.008	0.015	-0.006	1	0	0.015	0.015	0.011	0.051
X ₅	0	0	0	0	1	2.13E-018	0	0	0.160
X ₆	-0.020	-0.036	0.015	0.015	2.13E-018	1	-0.038	-0.026	0.092
X ₇	-0.020	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.138
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.062
y	0.785	0.468	0.192	0.051	0.160	0.092	0.138	0.062	1

Table (4.17): Correlation Matrix for Volume of Final Settling Tanks For Extended Aeration.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.836
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	0.338
X ₃	0.009	0.017	1	0	0.017	0.019	0.012	0.211
X ₅	0	-2.67E-018	0	1	2.14E018	0	0	0.186
X ₆	-0.023	-0.041	0.017	2.14E018	1	-0.044	-0.030	0.133
X ₇	-0.024	-0.044	0.019	0	-0.044	1	-0.032	0.135
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.085
y	0.836	0.338	0.211	0.186	0.133	0.135	0.085	1

Table (4.18): Correlation Matrix for Volume of Final Settling Tanks For Oxidation Ditch.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.844
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	-0.044
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	0.425
X ₅	0	-2.67E-018	0	1	2.14E-018	0	0	0.011
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.189
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.193
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.108
y	0.844	-0.044	0.425	0.011	0.189	0.193	0.108	1

Table (4.19): Correlation Matrix for Volume of Final Settling Tanks For Aerated Lagoon.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.770
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	0.263
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	0.351
X ₅	0	-2.67E-018	0	1	2.14E-018	0	0	0.183
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.167
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.202
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.127
y	0.770	0.263	0.351	0.183	0.167	0.202	0.127	1

Table (4.20): Correlation Matrix for Volume of Final Settling Tanks For Attach Growth System.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.843
X ₂	-0.020	1	0.015	0.015	0	-0.036	-0.038	-0.026	-0.030
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	0.438
X ₄	0.008	0.015	-0.006	1	0	0.015	0.015	0.011	0.012
X ₅	0	0	0	0	1	2.14E-018	0	0	3.55E-005
X ₆	-0.020	-0.036	0.015	0.015	2.14E-018	1	-0.038	-0.026	0.182
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.215
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.090
y	0.843	-0.030	0.438	0.012	3.55E-005	0.182	0.215	0.090	1

Table (4.21): Correlation Matrix for Volume of Gravity Thickeners For Conventional Activated Sludge.

	X ₁	X ₂	X ₃	X ₄	X	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.732
X ₂	-0.020	1	0.015	0.016	0	-0.036	-0.038	-0.026	0.373
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.111
X ₄	0.008	0.016	-0.006	1	0	0.015	0.015	0.011	0.456
X ₅	0	0	0	0	1	2.14E-018	0	0	0.147
X ₆	-0.020	-0.036	0.015	0.015	2.14E-018	1	-0.038	-0.026	0.142
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.170
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.066
y	0.732	0.373	-0.111	0.456	0.147	0.142	0.170	0.066	1

Table (4.22): Correlation Matrix for Volume of Gravity Thickeners For Ponds.

	X ₁	X ₃	X ₄	X ₆	X ₇	X ₈	y
X ₁	1	0.011	0.011	-0.027	-0.029	-0.20	0.783
X ₃	0.011	1	0.008	0.020	0.022	0.015	-0.052
X ₄	0.011	-0.008	1	0.020	0.021	0.015	0.554
X ₆	-0.027	0.020	0.020	1	-0.053	-0.036	0.161
X ₇	-0.029	0.022	0.021	-0.053	1	-0.039	0.192
X ₈	-0.20	0.015	0.015	-0.036	-0.039	1	0.077
y	0.783	-0.052	0.554	0.161	0.192	0.077	1

Table (4.23): Correlation Matrix for Volume of Gravity Thickeners For Attached Growth System.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.019	0.008	0.008	0	-0.019	-0.021	-0.014	0.805
X ₂	-0.019	1	0.014	0.014	0	-0.035	-0.037	-0.026	0.216
X ₃	0.008	0.014	1	-0.006	0	0.014	0.015	0.011	-0.072
X ₄	0.008	0.014	-0.006	1	0	0.014	0.015	0.011	0.411
X ₅	0	0	0	0	1	2.14E-018		0	0.185
X ₆	-0.019	-0.035	0.014	0.014	2.14E-018	1	-0.037	-0.026	0.165
X ₇	-0.021	-0.037	0.015	0.015	0	-0.037	1	-0.027	0.196
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.027	1	0.079
y	0.805	0.216	-0.072	0.411	0.185	0.165	0.196	0.079	1

Table (4.24): Correlation Matrix for Volume of Anaerobic Digesters For Conventional Activated Sludge.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.876
X ₂	-0.020	1	0.015	0.016	0	-0.036	-0.038	-0.026	0.251
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.074
X ₄	0.008	0.016	-0.006	1	0	0.015	0.015	0.011	0.322
X ₅	0	0	0	0	1	2.14E018	0	0	0.103
X ₆	-0.020	-0.036	0.015	0.015	2.14E018	1	-0.038	-0.026	0.091
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.110
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.088
y	0.876	0.251	-0.074	0.322	0.103	0.091	0.110	0.088	1

Table (4.25): Correlation Matrix for Volume of Anaerobic Digesters For Ponds.

	X ₁	X ₃	X ₄	X ₆	X ₇	X ₈	y
X ₁	1	0.011	0.011	-0.027	-0.028	-0.020	0.937
X ₃	0.011	1	-0.008	0.020	0.021	0.015	-0.084
X ₄	0.011	-0.008	1	0.20	0.021	0.014	0.183
X ₆	-0.027	0.020	0.20	1	-0.052	-0.036	0.071
X ₇	-0.028	0.021	0.021	-0.052	1	-0.038	0.086
X ₈	-0.020	0.015	0.014	-0.036	-0.038	1	0.099
y	0.937	-0.084	0.183	0.071	0.086	0.099	1

Table (4.26): Correlation Matrix for Volume of Anaerobic Digesters For Attached Growth System.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.922
X ₂	-0.020	1	0.015	0.015	0	-0.036	-0.038	-0.026	0.125
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.044
X ₄	0.008	0.015	-0.006	1	0	0.015	0.015	0.011	0.267
X ₅	0	0	0	0	1	2.14E-018	0	0	0.115
X ₆	-0.020	-0.036	0.015	0.015	2.14E-018	1	-0.038	-0.026	0.094
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.113
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.097
y	0.922	0.125	-0.044	0.267	0.115	0.094	0.113	0.097	1

Table (4.27): Correlation Matrix for Volume of Holding Tank For Extended Aeration.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.485
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	0.760
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.024
X ₅	0	-2.67E-018	0	1	2.14E-018	0	0	0.361
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.067
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.038	0.084
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.038	1	0.022
y	0.485	0.760	-0.024	0.361	0.067	0.084	0.022	1

Table (4.28): Correlation Matrix for Volume of Holding Tank For Oxidation Ditch.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.628
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	0.569
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.032
X ₅	0	-2.67E-018	0	1	2.14E-018	0	0	0.452
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.105
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.128
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.043
y	0.628	0.569	-0.032	0.452	0.105	0.128	0.043	1

Table (4.29): Correlation Matrix for Volume of Holding Tank For Aerated Lagoon.

	X ₁	X ₃	X ₆	X ₇	X ₈	y
X ₁	1	0.014	-0.034	-0.036	-0.025	0.842
X ₃	0.014	1	0.025	0.026	0.018	0.451
X ₆	-0.034	0.025	1	-0.065	-0.045	0.165
X ₇	-0.036	0.026	-0.065	1	-0.048	0.198
X ₈	-0.025	0.018	-0.045	-0.048	1	0.076
y	0.842	0.451	0.165	0.198	0.076	1

Table (4.30): Correlation Matrix for Volume of Total Air Required For Conventional Activated Sludge.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.723
X ₂	-0.020	1	0.15	0.15	0	-0.036	-0.038	-0.026	-0.428
X ₃	0.008	0.15	1	-0.006	0	0.015	0.016	0.011	-0.091
X ₄	0.008	0.15	-0.006	1	0	0.015	0.015	0.011	0.088
X ₅	0	0	0	0	1	-2.13E-018	0	0	0.459
X ₆	-0.020	-0.036	0.015	0.015	-2.13E-018	1	-0.038	-0.026	0.166
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.194
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.084
y	0.723	-0.428	-0.091	0.088	0.459	0.166	0.194	0.084	1

Table (4.31): Correlation Matrix for Volume of Total Air Required For Extended Aeration.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.714
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	-0.410
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.061
X ₅	0	-2.67E-018	0	1	2.14E-018	0	0	0.505
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.161
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.189
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.081
y	0.714	-0.410	-0.061	0.505	0.161	0.189	0.081	1

Table (4.32): Correlation Matrix for Volume of Total Air Required For Oxidation Ditch.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.758
X ₂	-0.023	1	0.017	-2.67E-018	-0.041	-0.044	-0.030	-0.240
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.054
X ₅	0	-2.67E-018	0	1	2.14E-018	0	0	0.529
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.165
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.194
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.081
y	0.758	-0.240	-0.054	0.529	0.165	0.194	0.081	1

Table (4.33): Correlation Matrix for Volume of Total Air Required For Aerated Lagoon.

	X ₁	X ₂	X ₃	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.023	0.009	0	-0.023	-0.024	-0.017	0.679
X ₂	-0.023	1	0.017		-0.041	-0.044	-0.030	-0.493
X ₃	0.009	0.017	1	0	0.017	0.018	0.012	-0.064
X ₅	0		0	1	2.14E-018	0	0	0.482
X ₆	-0.023	-0.041	0.017	2.14E-018	1	-0.044	-0.030	0.158
X ₇	-0.024	-0.044	0.018	0	-0.044	1	-0.032	0.185
X ₈	-0.017	-0.030	0.012	0	-0.030	-0.032	1	0.081
y	0.679	-0.493	-0.064	0.482	0.158	0.185	0.081	1

Table (4.34): Correlation Matrix for Quantity of Gas Produced For Conventional activated sludge.

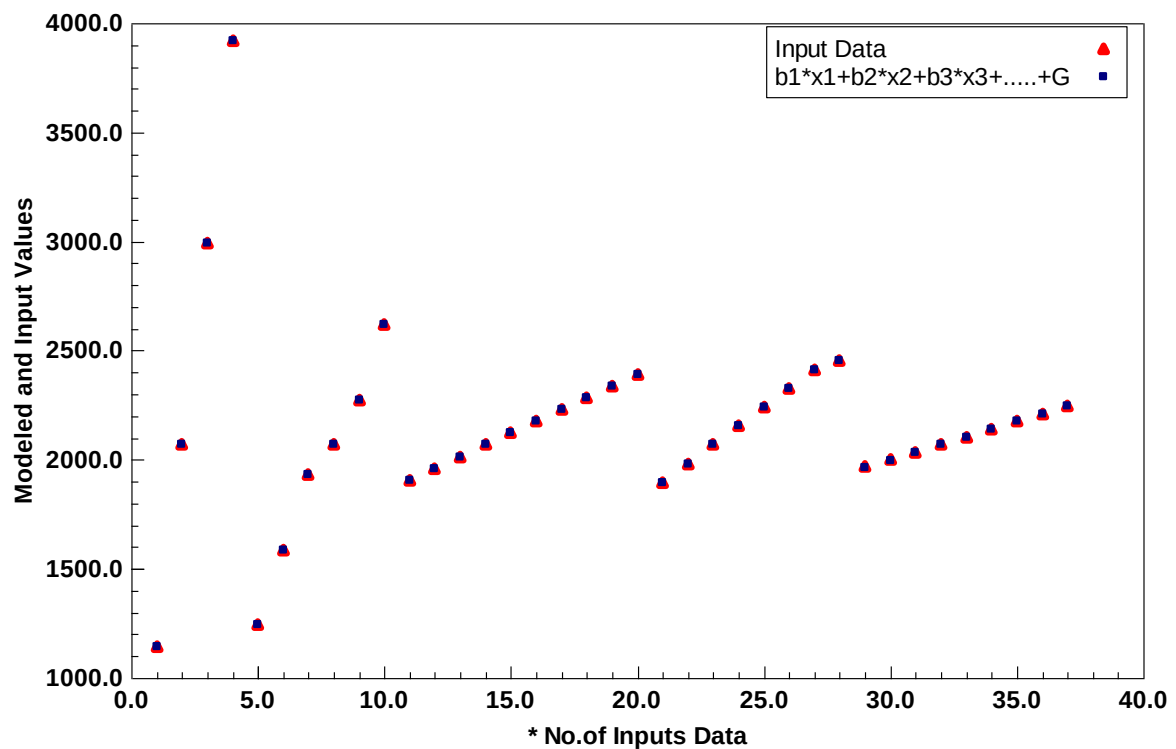
	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.883
X ₂	-0.020	1	0.015	0.016	0	-0.036	-0.038	-0.026	0.244
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.072
X ₄	0.008	0.016	-0.006	1	0	0.015	0.015	0.011	0.313
X ₅	0	0	0	0	1	2.14-E018	0	0	0.100
X ₆	-0.020	-0.036	0.015	0.015	2.14-E018	1	-0.038	-0.026	0.087
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.106
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.089
y	0.883	0.244	-0.072	0.313	0.100	0.087	0.106	0.089	1

Table (4.35): Correlation Matrix for Quantity of Gas Produced For Stabilization ponds.

	X ₁	X ₃	X ₄	X ₆	X ₇	X ₈	y
X ₁	1	0.011	0.011	-0.027	-0.029	-0.020	0.918
X ₃	0.011	1	-0.008	0.020	0.021	0.015	-0.026
X ₄	0.011	-0.008	1	0.020	0.021	0.015	0.349
X ₆	-0.027	0.020	0.020	1	-0.053	-0.038	0.086
X ₇	-0.029	0.021	0.021	-0.053	1	-0.039	0.104
X ₈	-0.020	0.015	0.015	-0.038	-0.039	1	0.095
y	0.918	-0.026	0.349	0.086	0.104	0.095	1

Table (4.36): Correlation Matrix for Quantity of Gas Produced For Attach Growth System.

	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈	y
X ₁	1	-0.020	0.008	0.008	0	-0.020	-0.021	-0.014	0.908
X ₂	-0.020	1	0.015	0.015	0	-0.036	-0.038	-0.026	0.141
X ₃	0.008	0.015	1	-0.006	0	0.015	0.016	0.011	-0.046
X ₄	0.008	0.015	-0.006	1	0	0.015	0.015	0.011	0.288
X ₅	0	0	0	0	1	2.14E-018	0	0	0.128
X ₆	-0.020	-0.036	0.015	0.015	2.14E-018	1	-0.038	-0.026	0.105
X ₇	-0.021	-0.038	0.016	0.015	0	-0.038	1	-0.028	0.126
X ₈	-0.014	-0.026	0.011	0.011	0	-0.026	-0.028	1	0.095
y	0.908	0.141	-0.046	0.288	0.128	0.105	0.126	0.095	1

**Fig. (4.1): The Input Versus Modeled of Volume of Primary Sedimentation Tanks.**

* No.of input cases to run the program.

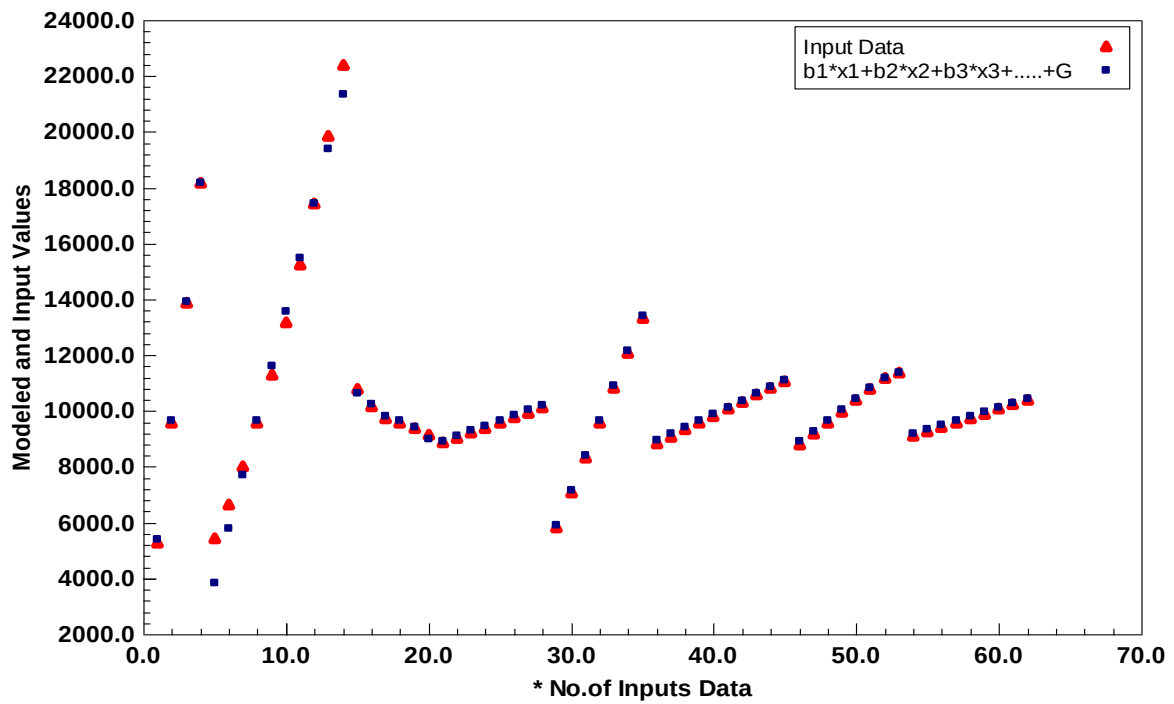


Fig. (4.2): The Input Versus Modeled of Volume of Aerobic Reactors Basins For Conventional Activated Sludge

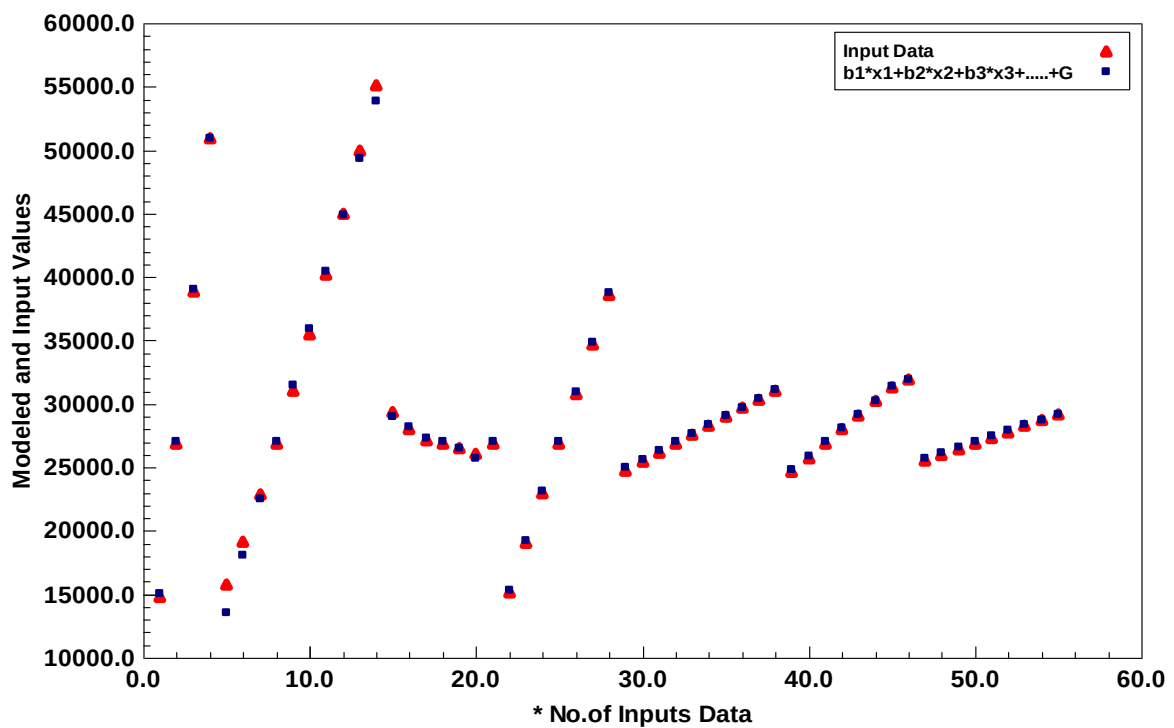


Fig. (4.3): The Input Versus Modeled of Volume of Extended Aeration.

* No. of input cases to run the program.

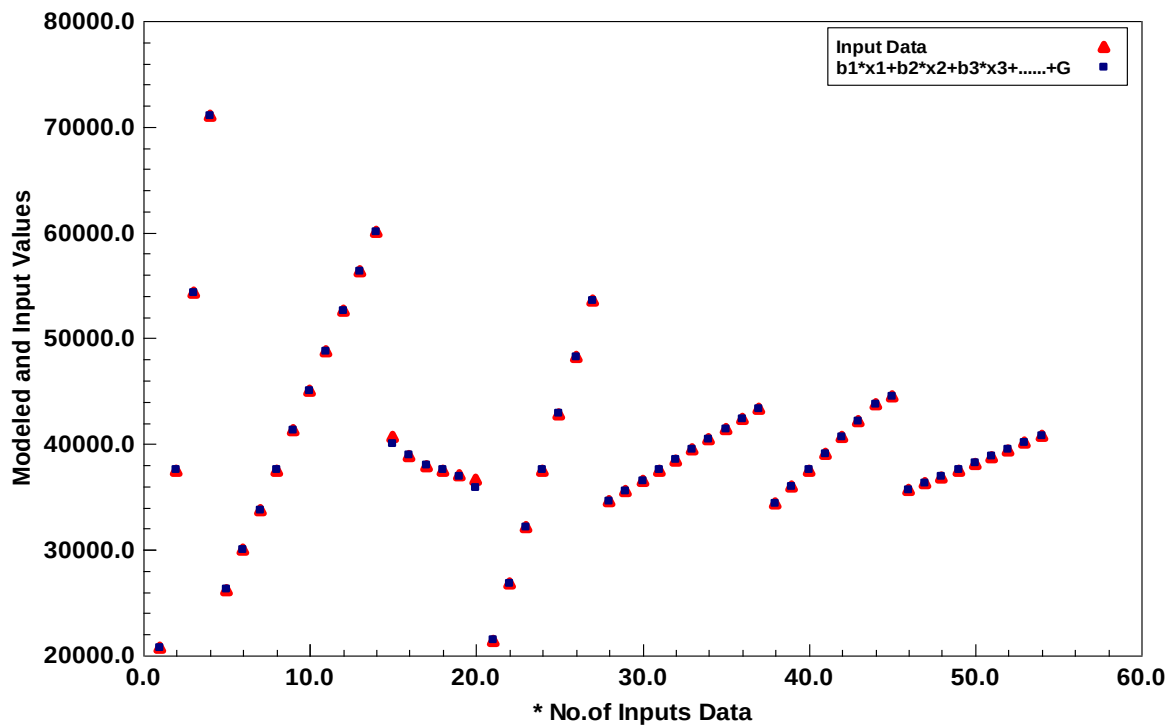


Fig. (4.4): The Input Versus Modeled of Volume of Oxidation Ditch.

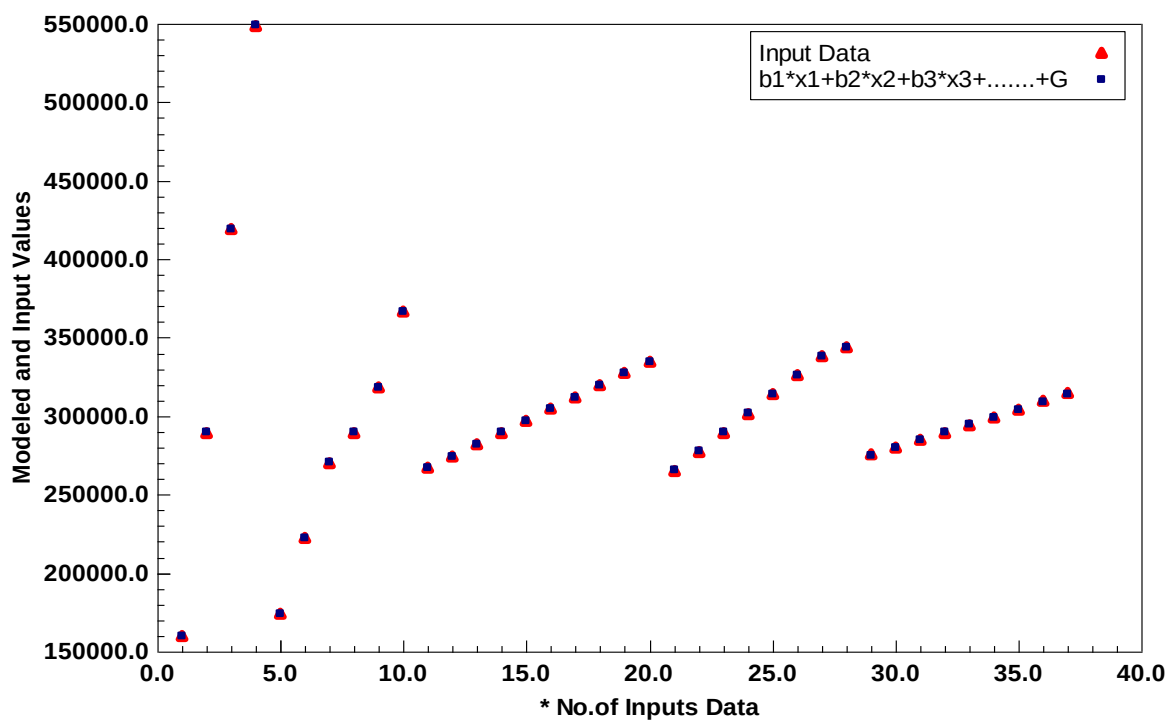


Fig. (4.5): The Input Versus Modeled of Volume of Aerated Lagoon.

* No. of input cases to run the program.

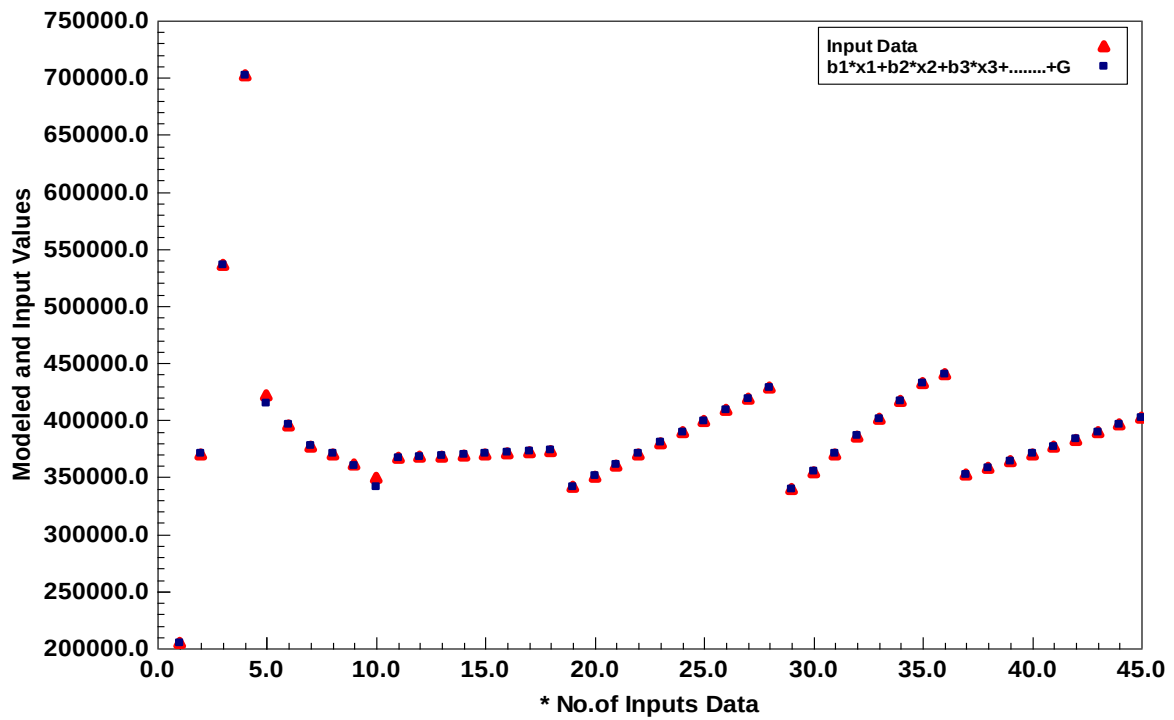


Fig. (4.6): The Input Versus Modeled of Volume of Aerobic Ponds.

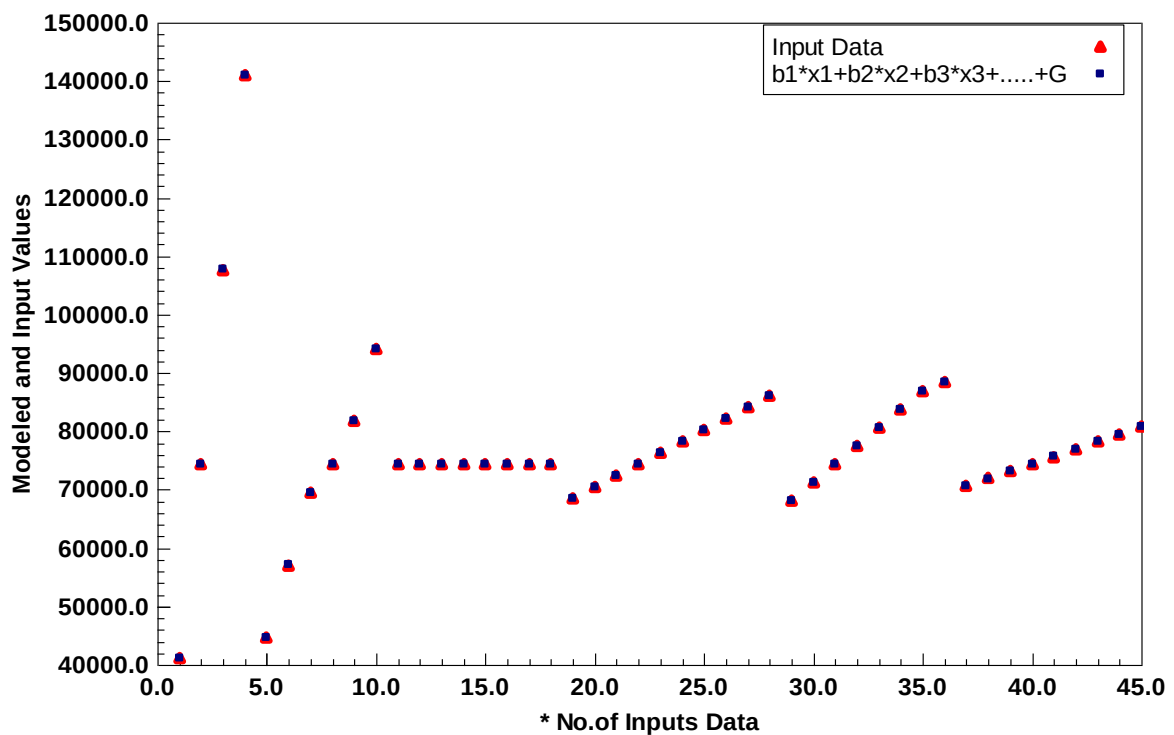


Fig. (4.7): The Input Versus Modeled of Volume of Anaerobic Pond.

* No.of input cases to run the program.

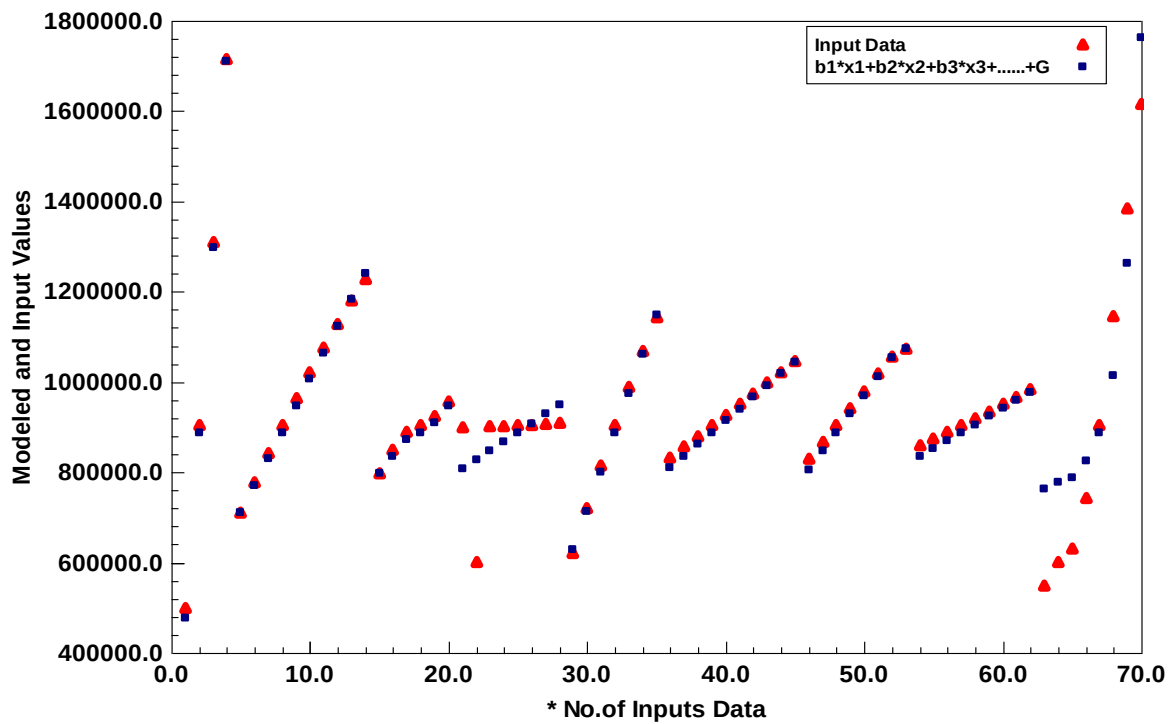


Fig. (4.8): The Input Versus Modeled of Volume of Facultative Pond.

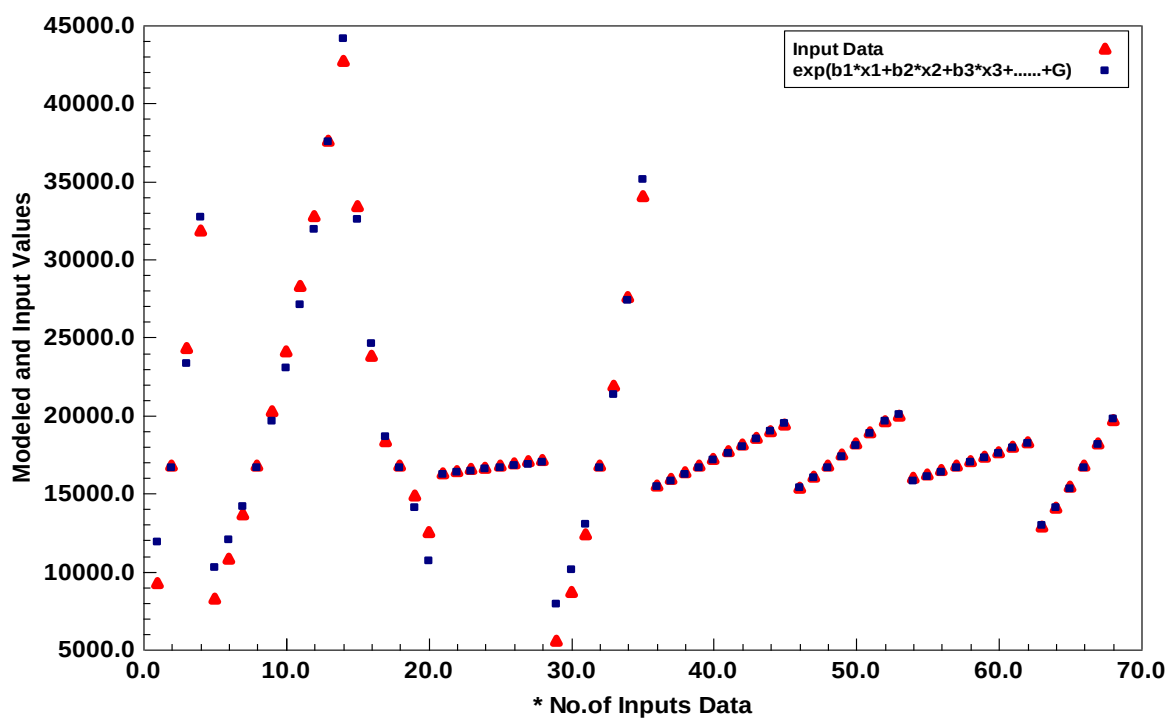


Fig. (4.9): The Input Versus Modeled of Volume of Trickling Filter.

* No. of input cases to run the program.

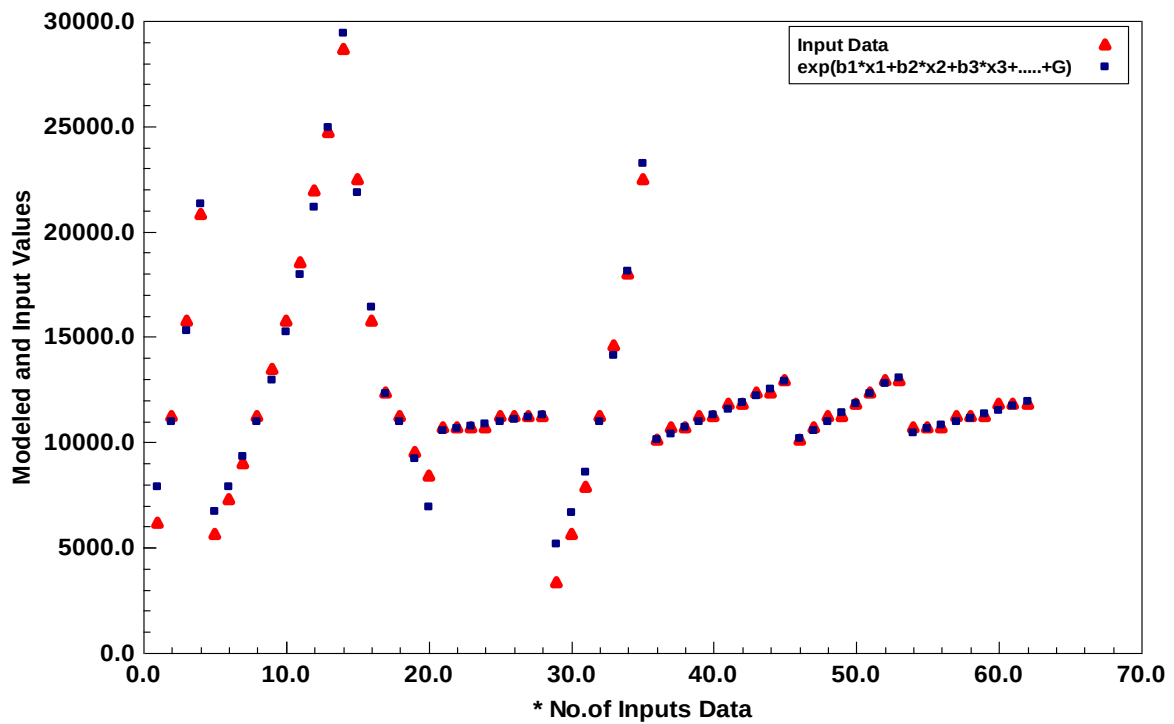


Fig. (4.10): The Input Versus Modeled of Volume of Rotating Biological Contactor.

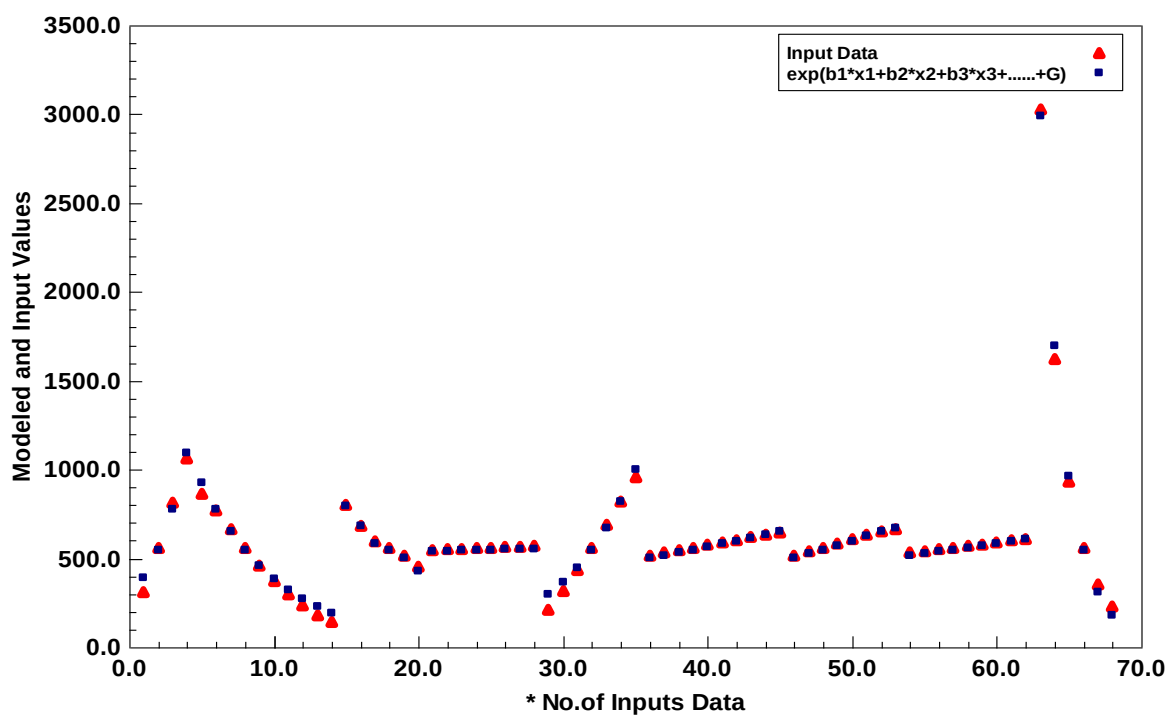


Fig. (4.11): The Input Versus Modeled of Volume of Bio-tower.

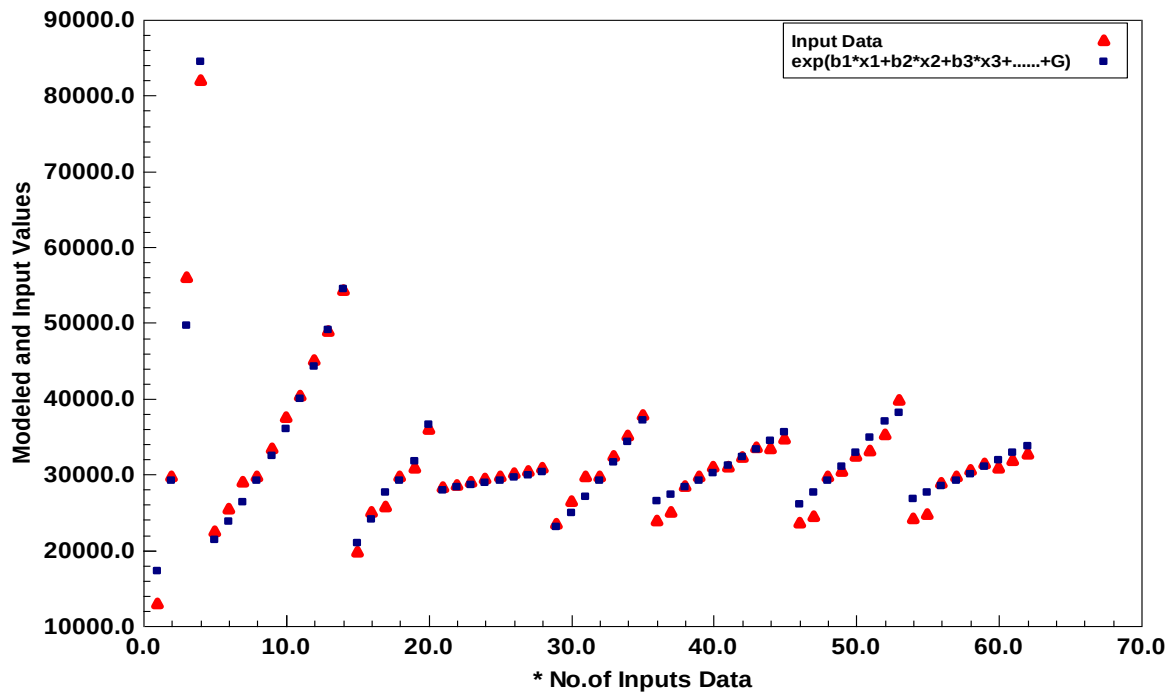


Fig. (4.12): The Input Versus Modeled of Volume of Final Settling Tanks for Conventional Activated Sludge.

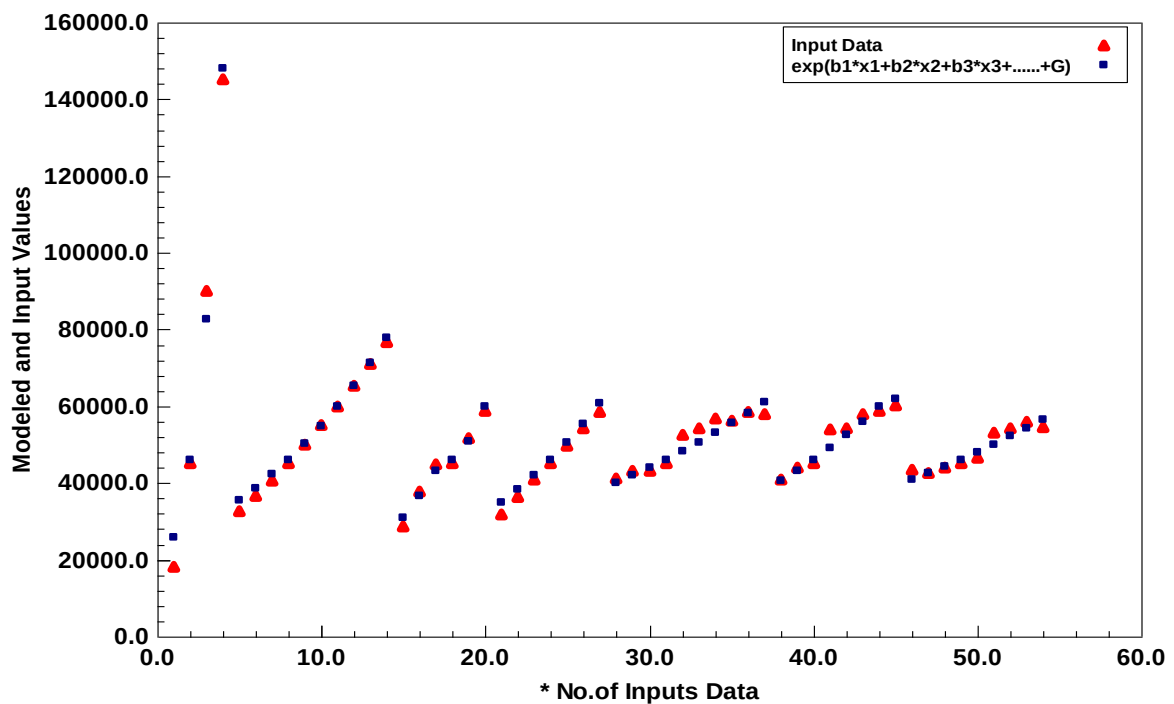


Fig. (4.13): The Input Versus Modeled of Volume of Final Settling Tanks for Extended Aeration.

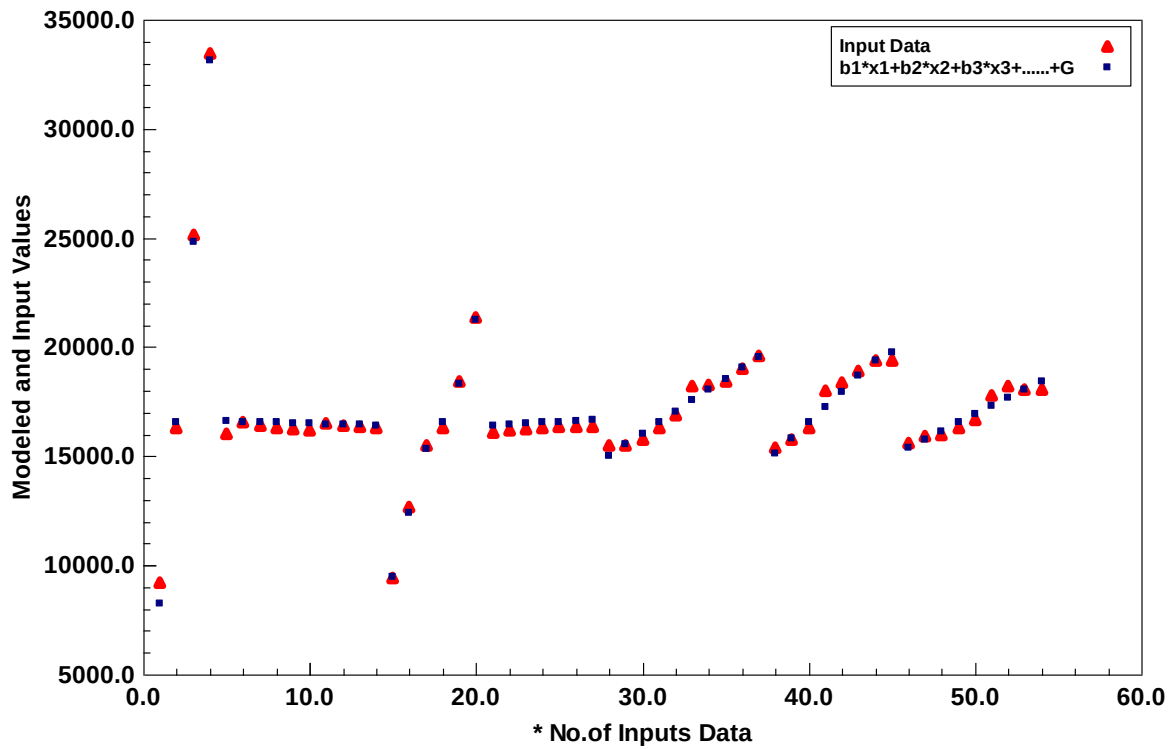


Fig. (4.14): The Input Versus Modeled of Volume of Final Settling Tanks for Oxidation ditch.

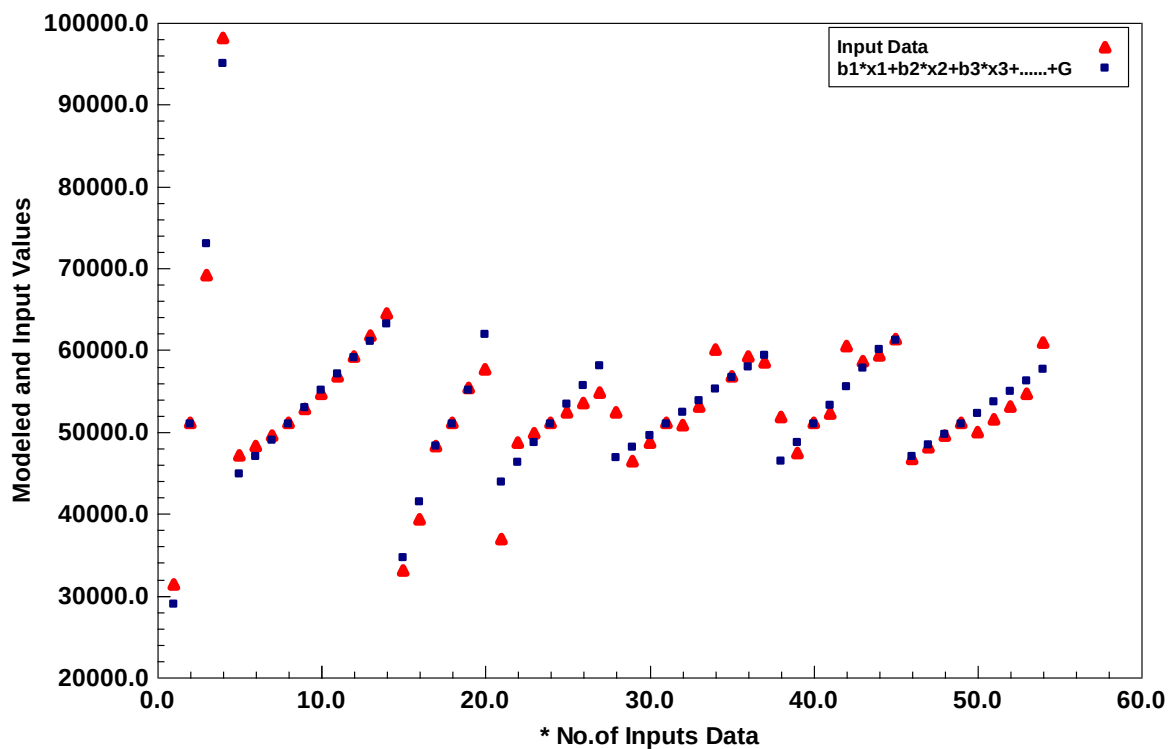


Fig. (4.15): The Input Versus Modeled of Volume of Final Settling Tanks for Aerated lagoon.

* No. of input cases to run the program.

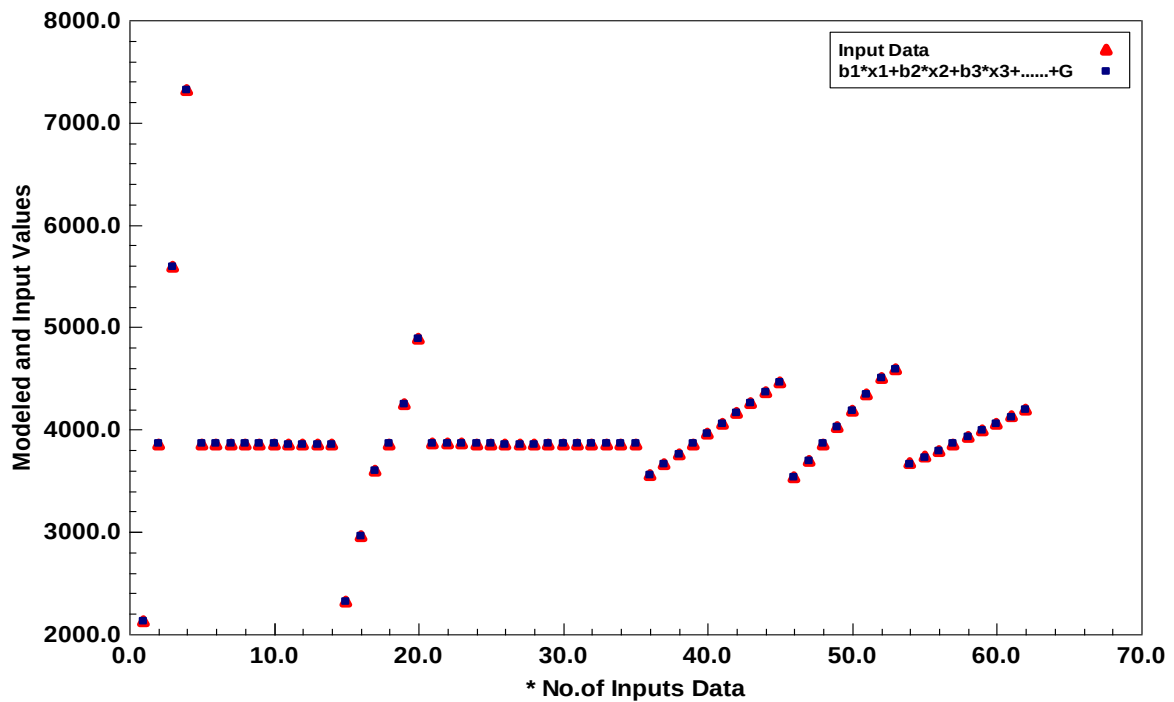


Fig. (4.16): The Input Versus Modeled of Volume of Final Settling Tanks for Attach Growth System.

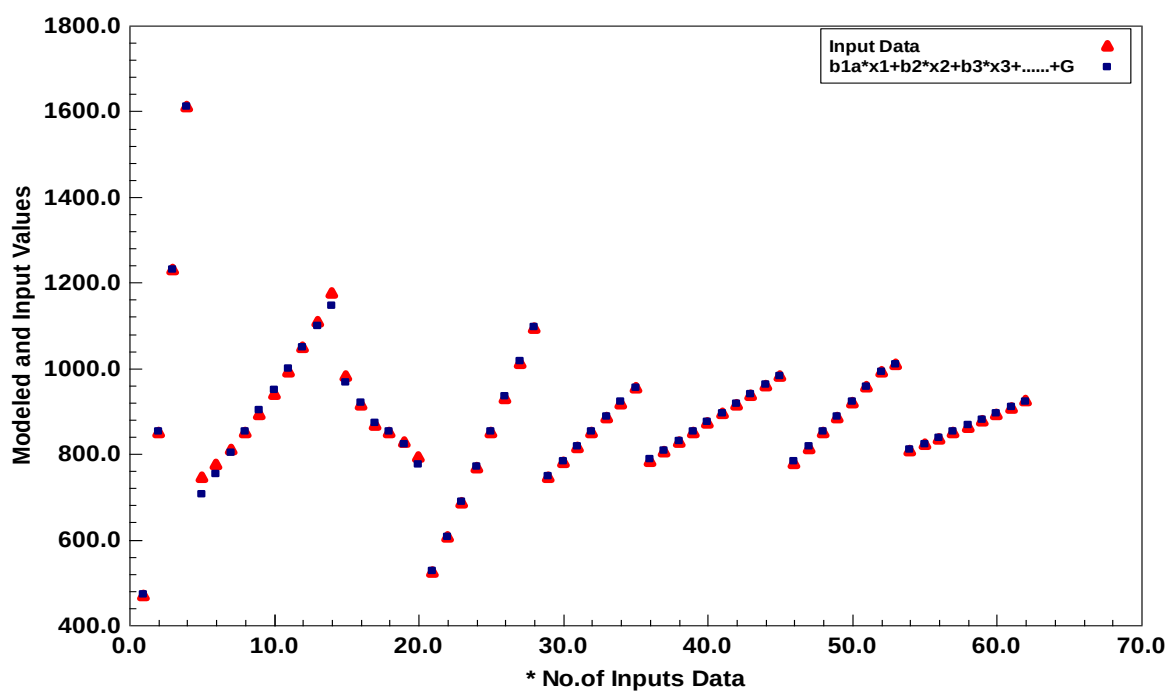


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* No.of input cases to run the program.

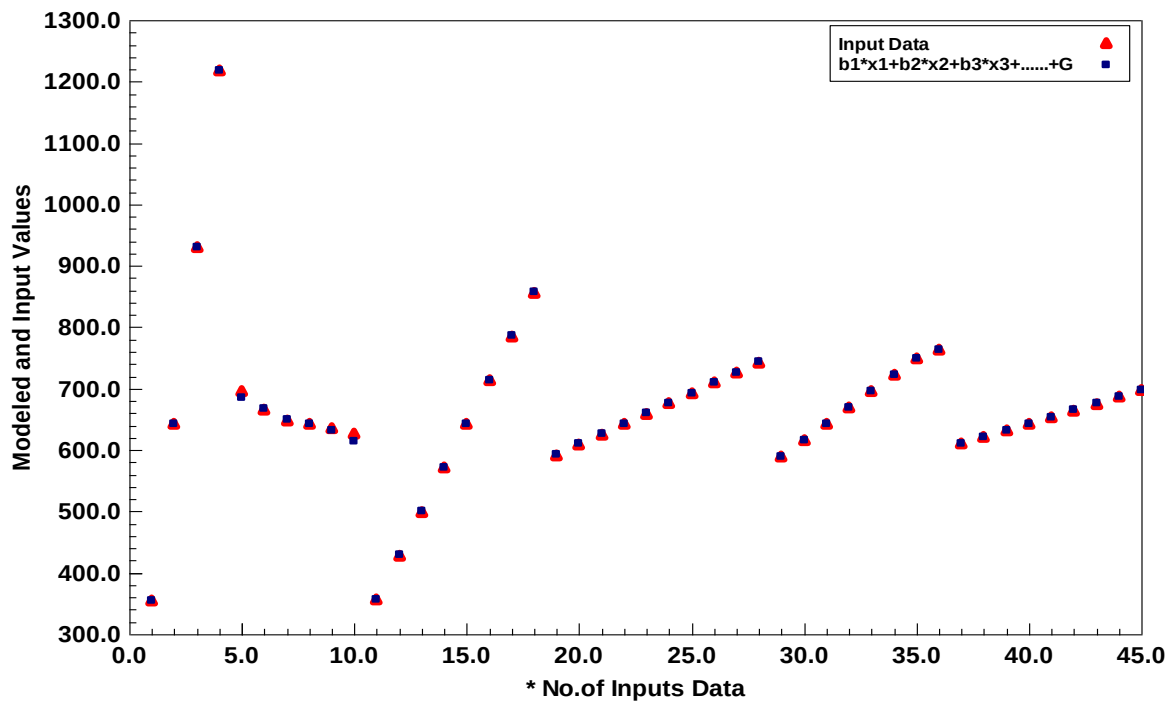


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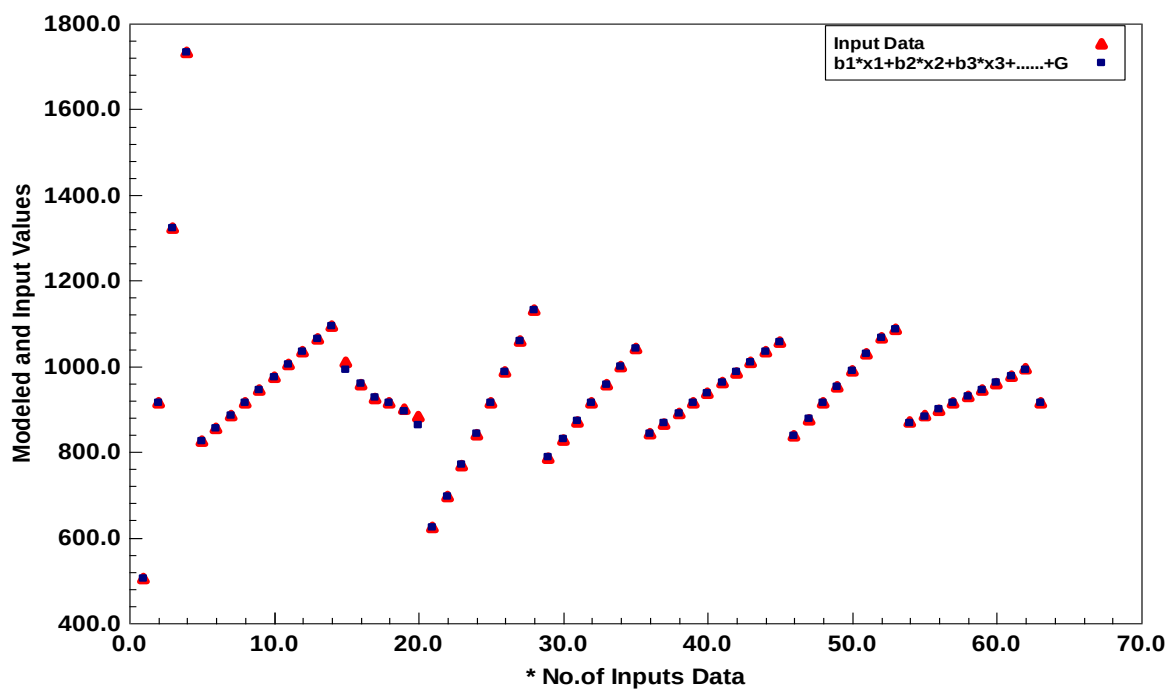


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* No. of input cases to run the program.

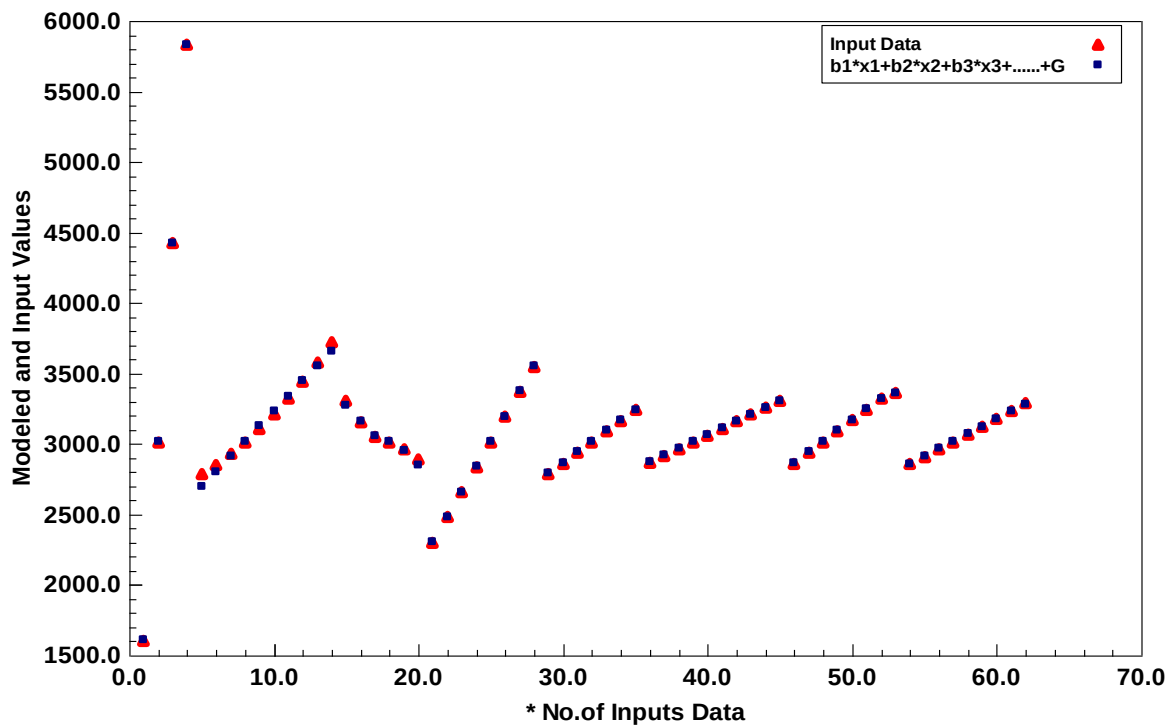


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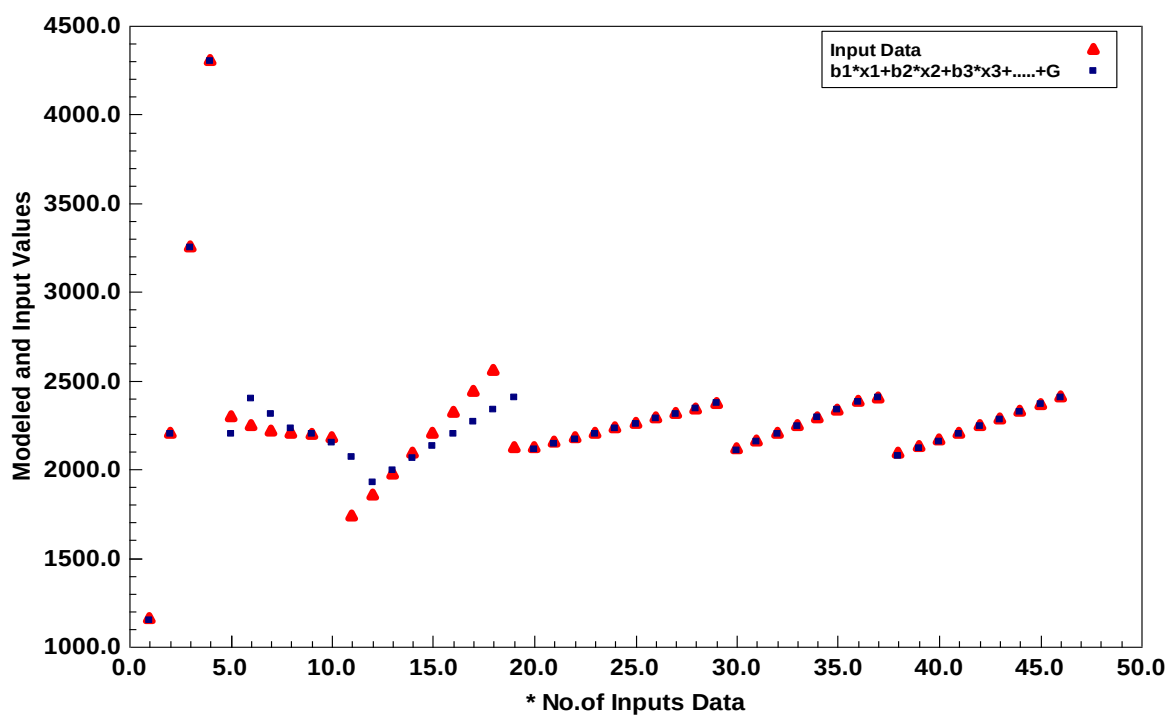


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* No. of input cases to run the program.

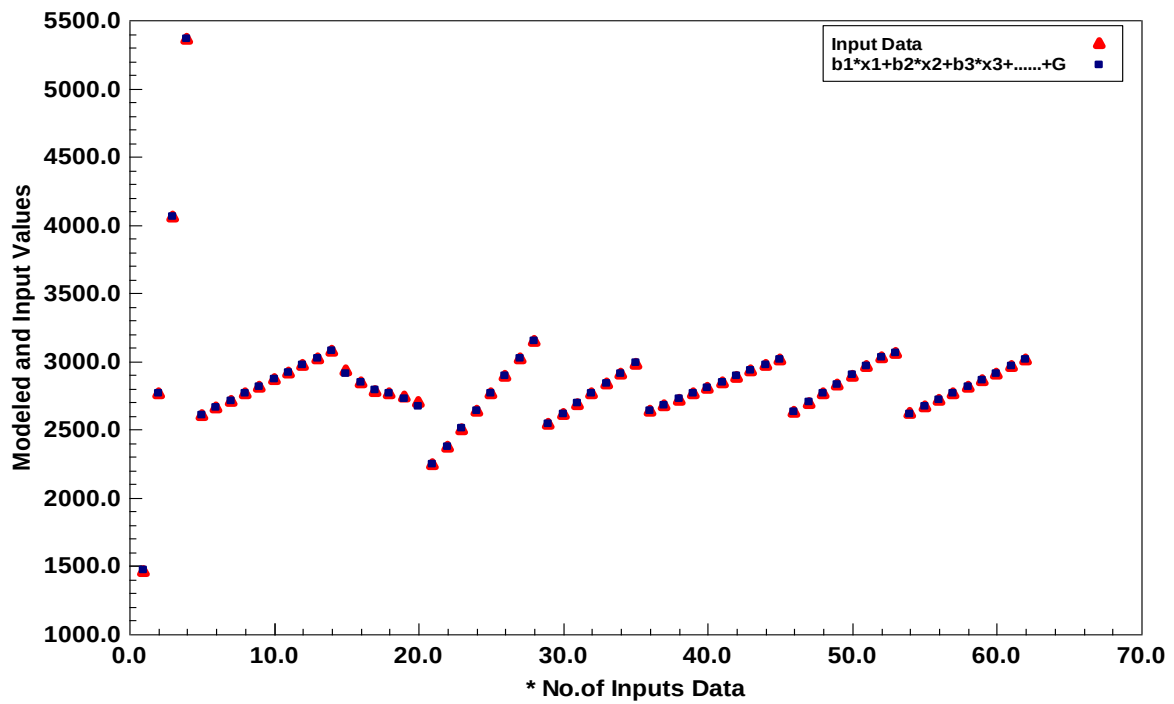


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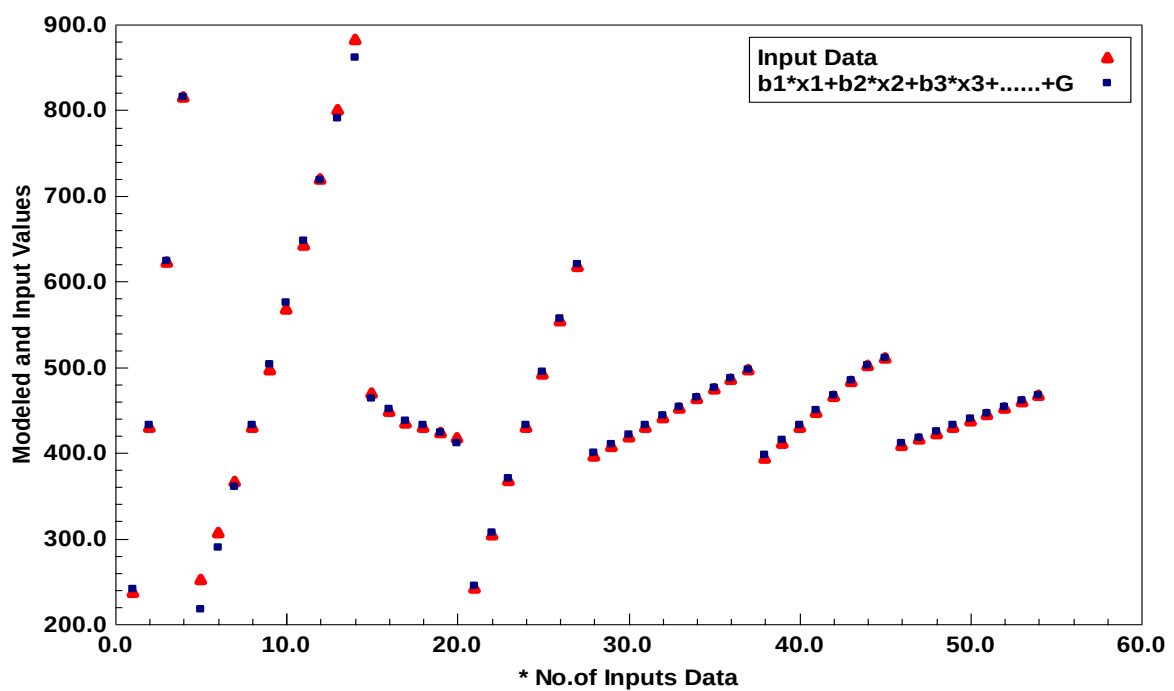


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* No. of input cases to run the program.

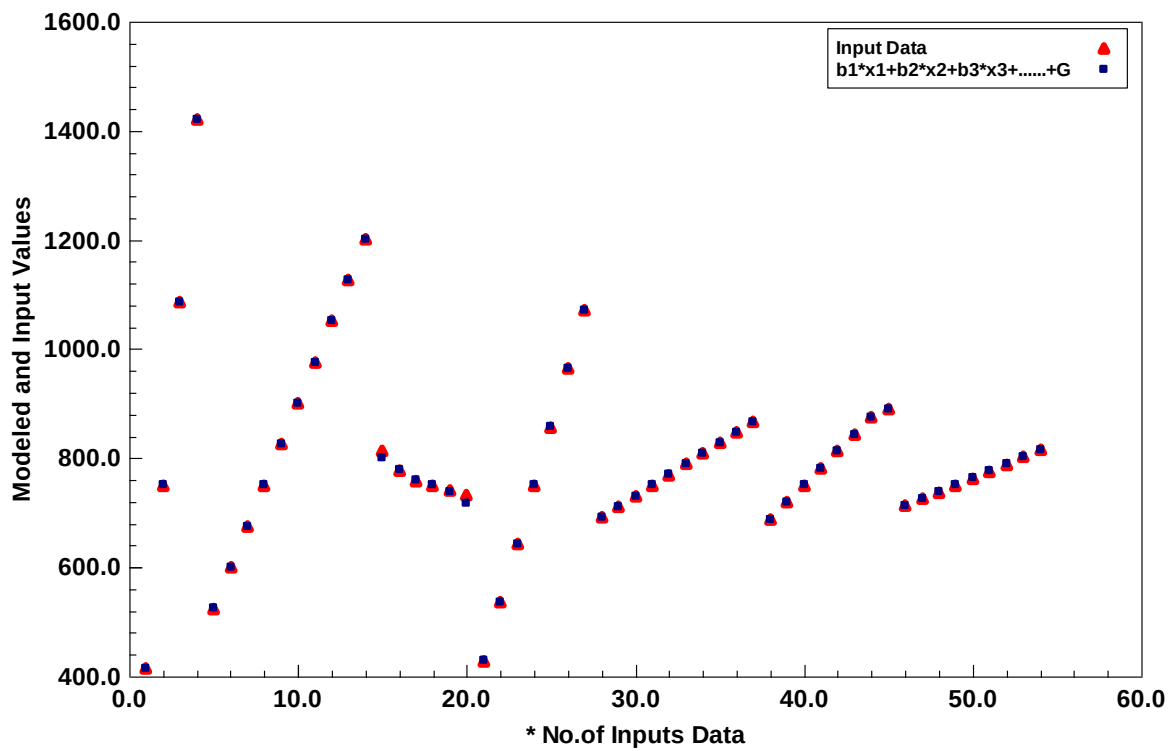


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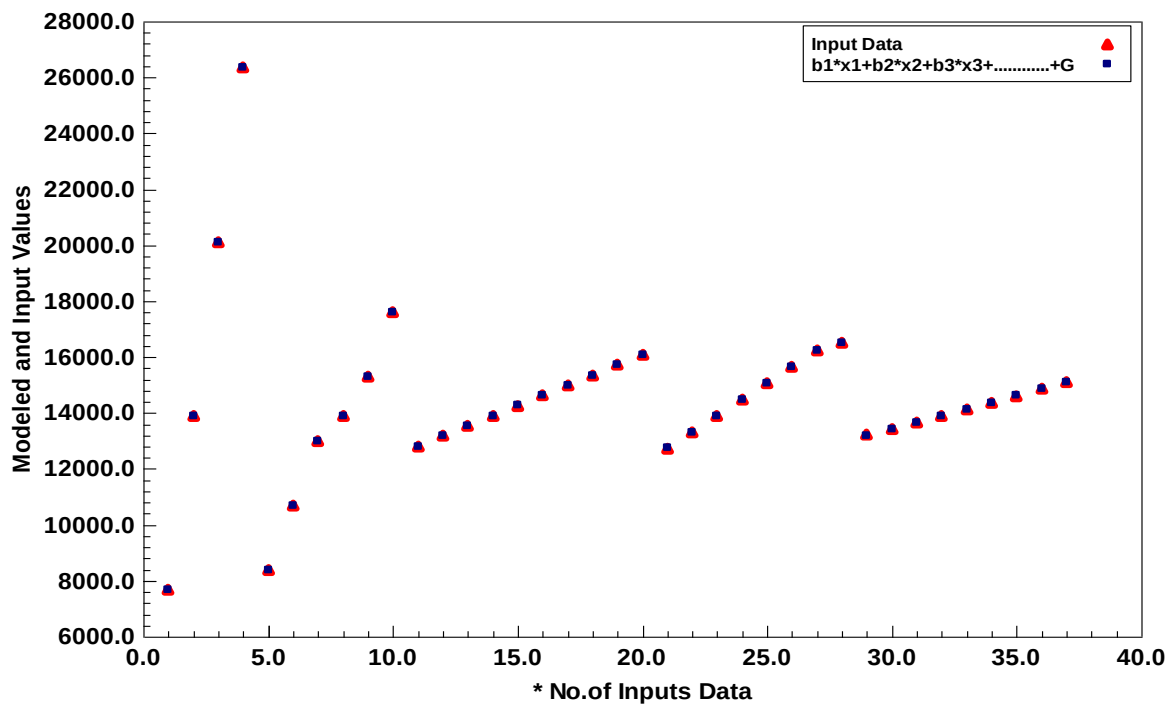


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* No. of input cases to run the program.

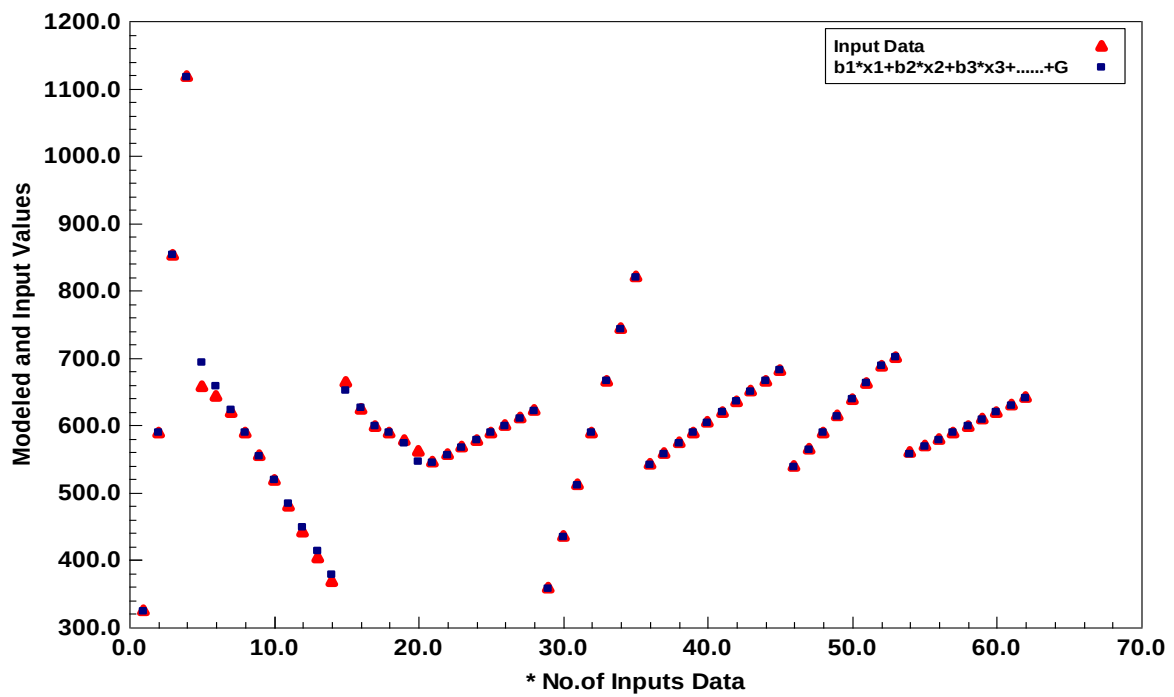


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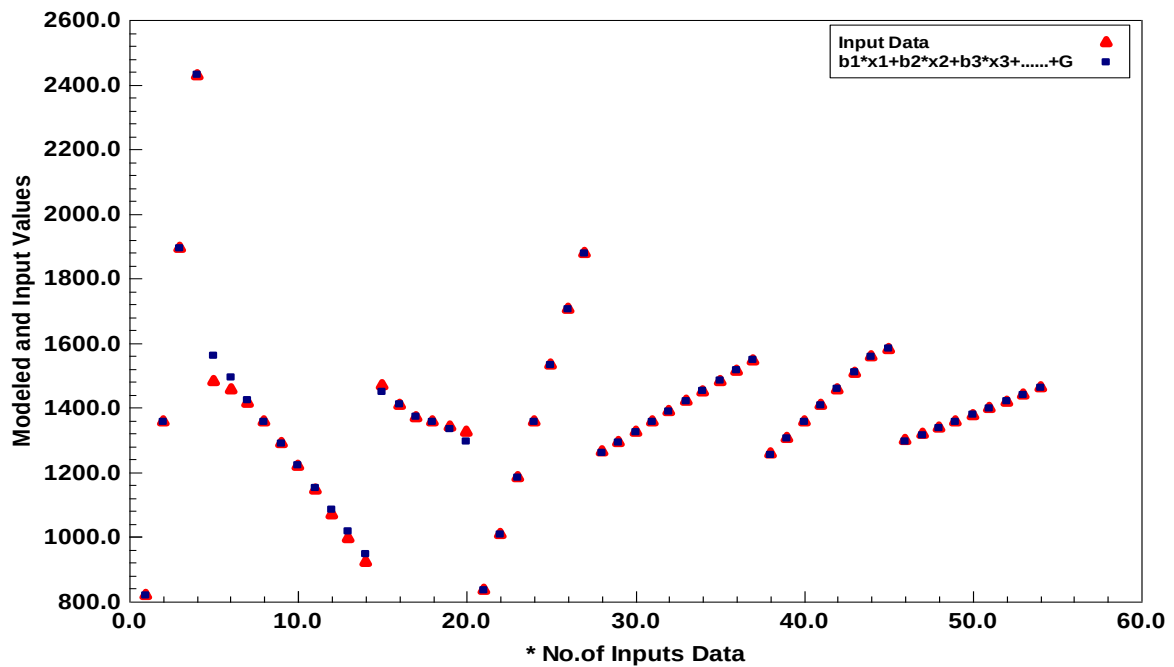


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* No. of input cases to run the program.

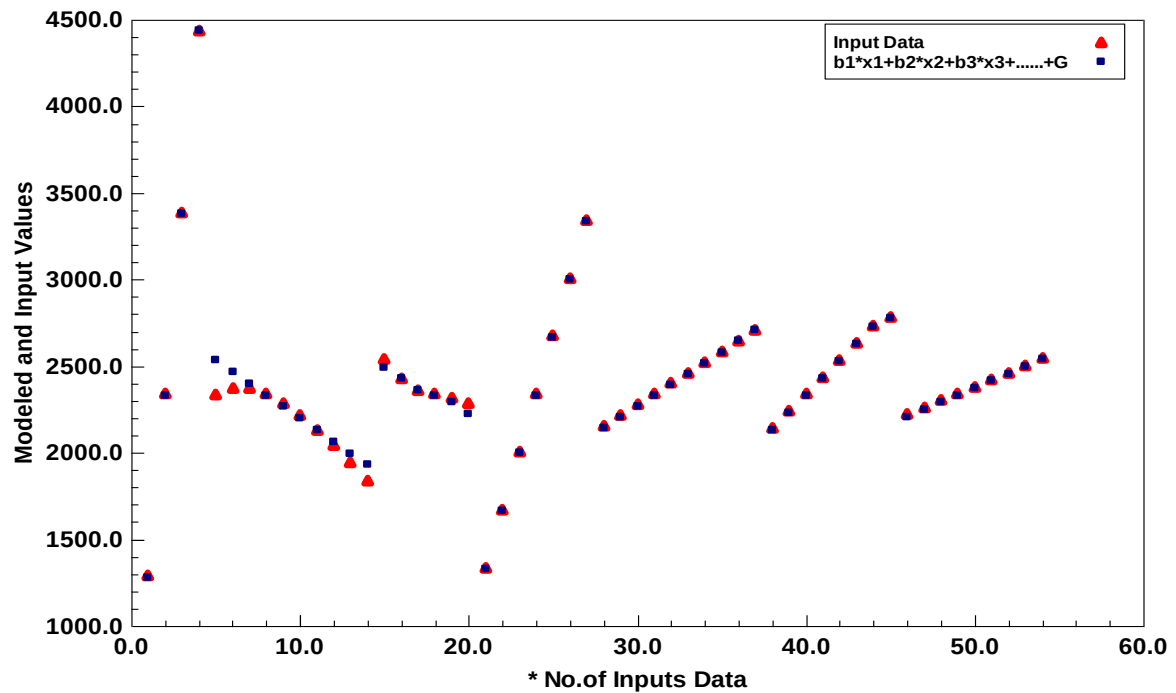


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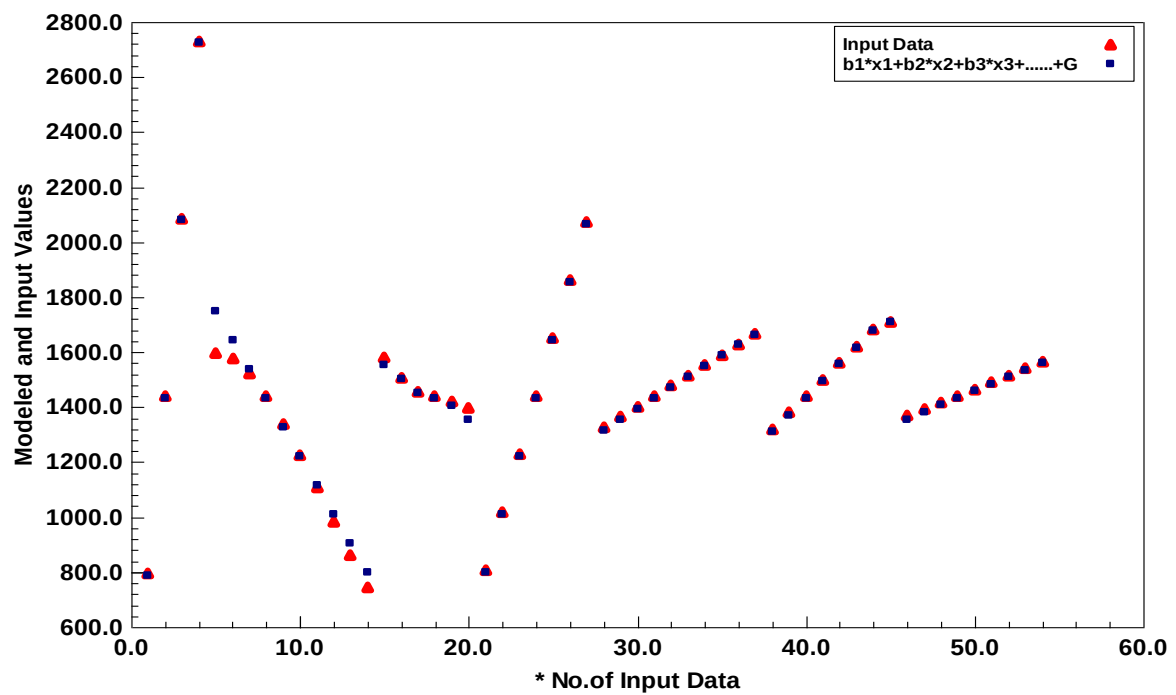


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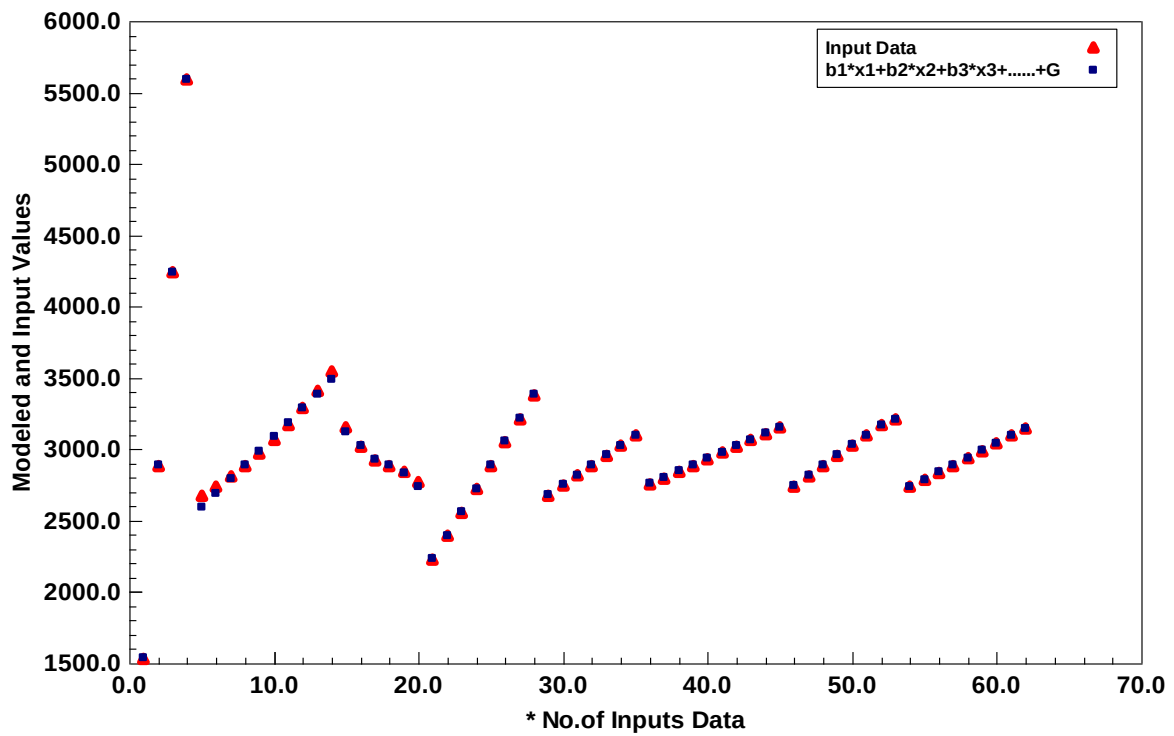


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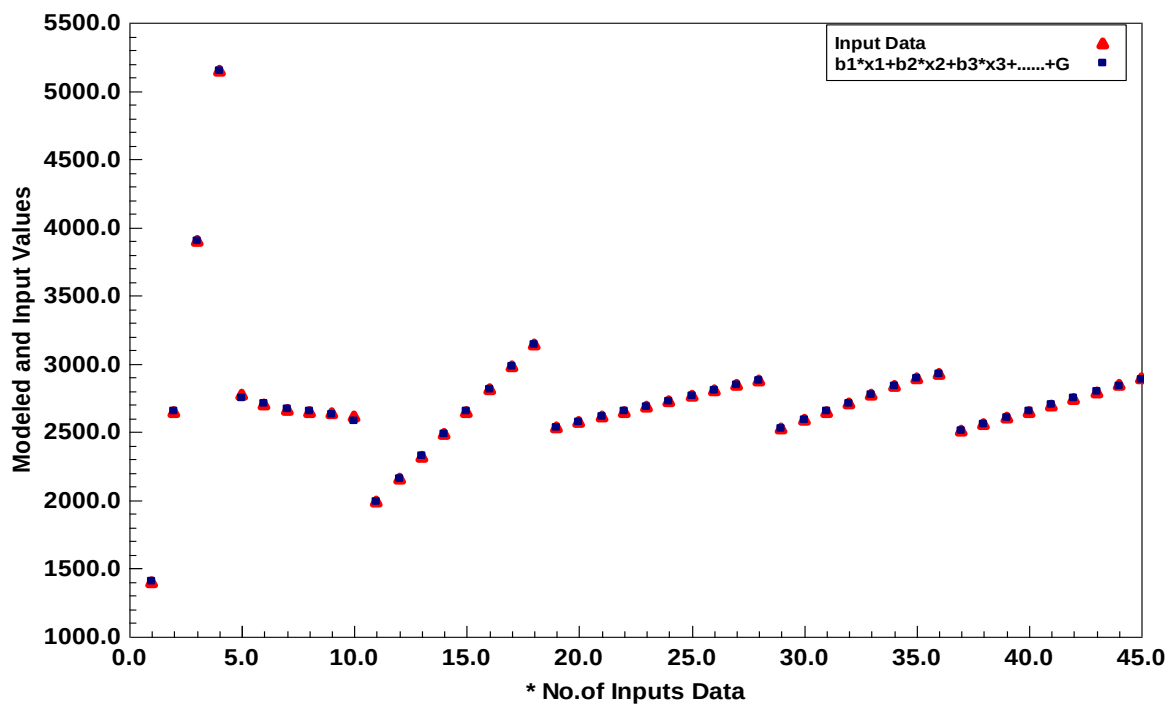


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* No.of input cases to run the program.

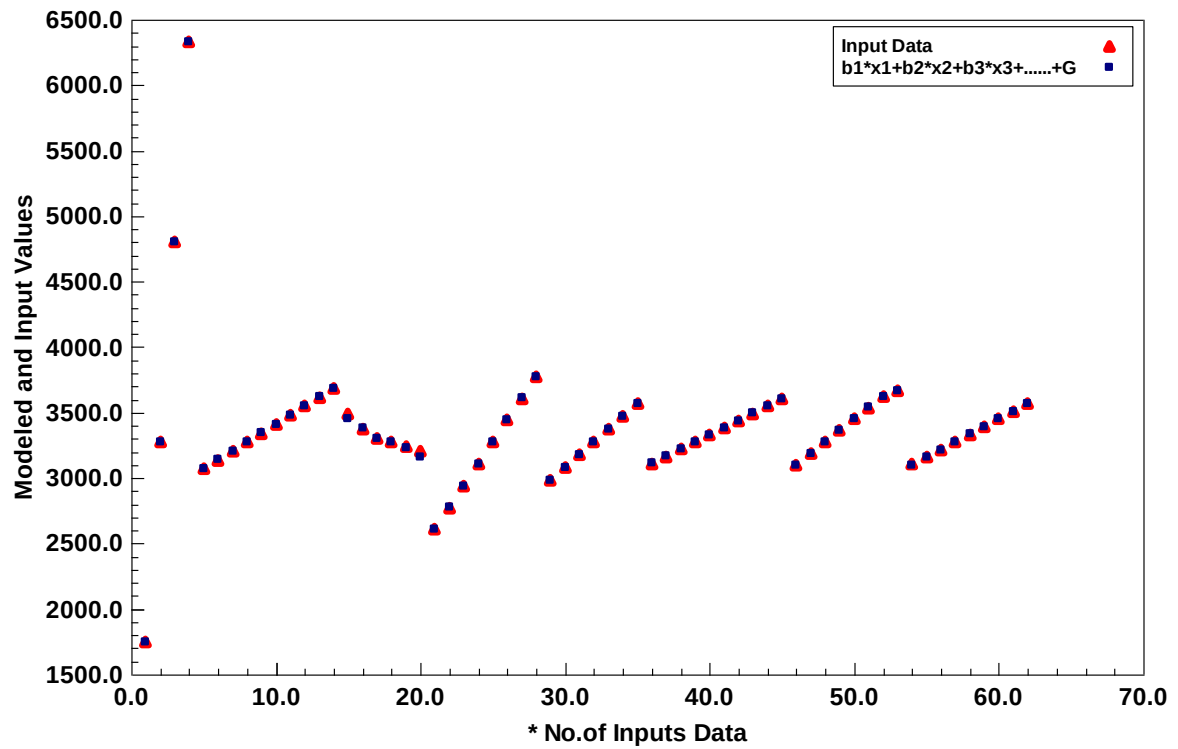


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* No. of input cases to run the program.

CERTIFICATION

We certify that the preparation of this thesis entitled "**Evaluating the Effecting Factors on the Sizing Design of Municipal Wastewater Treatment Plants**" is prepared by "**Nawras Shaker Jawad**" under our supervision at Babylon University, College of Engineering in partial fulfillment of the requirements for the degree of Master in Environmental Engineering.

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TABLE OF SYMBOLS

<u>Symbols</u>	<u>definition</u>	<u>unit</u>
A	Surface area	m ²
AL	Aerated Lagoon	□
An	Area served by network	hectare
ANP	Anaerobic pond	□
AP	Aerobic pond	□
A _s	Specific surface area	m ³ /m ²
ASP	Activated sludge plant	□
bs	Minimum clear spacing of bars	m
b _{1,2,3,...}	Regression coefficients	□
BOD	Biochemical Oxygen demand	mg/L
BT	Bio-tower	□
c	Coefficient of discharge	□
CL	Per capita loading	□
CSF	Clarifier safety factor	□
d	Depth	m
ds	Depth of applied sludge	m
$\frac{dX}{dt}$	Rate of change of microorganisms concentration (VSS)	mg/L.day
D	Dispersion factor	□
E	Efficiency of energy conversion	%
E ₁	Efficiency of BOD removal for first stage	%
E ₂	Efficiency of BOD ₅ removal for second-stage	%
EA	Extent aeration	□
f	Constant factor	□
f _b	Biodegradable fraction of VSS	□
F	Recirculation factor	□
FP	Facultative pond	□
$\frac{F}{M}$	Food-to-microorganism ratio	kg/kg
g	Acceleration due to gravity	m/s ²

gpcd	Gram per capita per day	□
G	Constant coefficient for regression equation	□
h	Unit heat of combustion of algae cells	cal/g
h_L	Head loss	m
H	Axial dispersion coefficient	m^2/h
HL	Hydraulic loading	$m^3/m^2.d$
i	Infiltration rate	L/s/ha
ISV	Initial settling velocity	m^2/h
k	Maximum rate of substrate utilization per unit mass of microorganism	d^{-1}
k_{20}	Reaction rate at 20°C	d^{-1}
k_t	Reaction rate at minimum operating water temperature	d^{-1}
K	Observed reaction rate constant	d^{-1}
K_d	Endogenous decay coefficient	d^{-1}
K_s	Substrate concentration at which rate of substrate utilization is one-half of the maximum rate	kg/m^3
L/c/d	Liter per capita per day	□
L	Length of travel path of a typical particle	m
L_{ds}	Sludge loading rate	$kg/kg.d$
L_i	Influent BOD	mg/L
MLSS	Mixed liquor suspended solids in the aeration tank	mg/L
MLVSS	Mixed liquid volatile suspended solids concentration	mg/L
n	Constant	□
O_2	Theoretical air required	kg/d
OD	Oxidation ditch	□
p	Oxygenation factor representing the oxygen produced per day per unit of biomass synthesized.	□
P_o	Initial Population	capita
P_f	Future population	capita
p_s	Saturation population	capita
P_t	Population in thousands	capita
P_x	Net mass of cell tissue produced	kg/d
PF	Peak factor	□
q	Growth yield coefficient	d^{-1}
Q	Flow rate	m^3/d
Q_{avg}	Average flow rate	m^3/d

Q_m	Minimum design flow rate	m^3/d
Q_p	Peak flow rate	m^3/d
Q_r	Recirculation flow	m^3/d
Q_{wa}	Flow of waste sludge removed from the aeration tank	m^3/d
Q_{wr}	Flow of return sludge from sludge line	m^3/d
r	Recirculation ratio	$\square$
r_g'	Net rate of microorganism growth (VSS)	$mg/L.d$
r_g	Growth rate	$\%$
r_{su}	Substrate utilization rate	$mg/m^3.d$
R	Degree of reduction of the cellular material	$\square$
R^2	Coefficient of multiple determination	$\square$
RBC	Rotating Biological Contractors	$\square$
S	Effluent soluble BOD5 concentration	mg/L
S_0	Influent soluble BOD5 concentration	mg/L
$S_0 - S$	Mass concentration of substrate utilized	mg/L
S_e	Effluent soluble BOD5	mg/L
SL	Solid loading	$kg/m^2.d$
SOR	Surface overflow rate	$m^3/m^2.d$
S_R	Solar radiation incident on the pond surface	$cal/cm^2.d$
t	Detention time	min
t_{an}	Retention time for anaerobic pond	day
t_f	Design period	$year$
T	Minimum operating temperature	$^{\circ}C$
T_a	Ambient air temperature	$^{\circ}C$
T_i	Influent waste temperature	$^{\circ}C$
TSS	Total suspended solids	mg/L
TF	Trickling filter	$\square$
v	Approach velocity in upstream channel	m/s
V	Volume	m^3
V_{gas}	Volume of methane produced at standard conditions	L/d
V_R	Approach velocity in rack chamber	m/s
VSS	Volatile suspended solids	mg/L
w	Maximum width of the bar facing the flow	m
W	BOD loading to filter	kg/d
W_a	Algae biomass produced	g/d
W_{O_2}	Oxygen production as a result of photosynthesis	g/d

WWTP	Wastewater treatment plant	□
X	Concentration of MLVSS maintained in the tank	mg/L
X_o	Microorganisms concentration (VSS) in influent	mg/L
$x_{1,2,3,...}$	Independent variables	□
X_e	Microorganism concentration (VSS) in effluent	mg/L
X_r	Return concentration	mg/L
y	Dependent variables	□
Y	Maximum yield coefficient over finite period of log growth	mg/mg
Y_{obs}	Observed yield	g/g
β	Bar shape factor	□
θ	Hydraulic retention time	d
θ_c	Mean cell residence time	d
θ_f	Temperature correction factor	□
μ_o	Specific sewage production	L/c/d

Note: Any other notation may be explained where it appears in the thesis.

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Nawras Shaker Jawad

2009

بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ
وَيَسْأَلُونَكَ عَنِ الرُّوحِ قُلِ الرُّوحُ مِنْ أَمْرِ
رَبِّي وَمَا أُوتِيتُمْ مِنَ الْعِلْمِ إِلَّا قَلِيلًا

صدق الله العظيم

سورة الإسراء / الآية 85

CERTIFICATION

We certify that we have read this thesis, entitled "**Evaluating the Effecting Factors on the Sizing Design of Municipal Wastewater Treatment Plants**", and as examining committee, examined the student "**Nawras Shaker Jawad**" in its contents and in what is connected with it, and that in our opinion it meets the standard of a thesis for the degree of Master in Environmental Engineering.

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Table (4.4): Results of Study.

Y	Models	R ²	Stand. Err	Correlation matrix (table)	Relationship (Fig)
1-Volume (m³) of:					
● Primary Sedimentation Tanks	$y=0.019x_1 + 6.863x_3 + 0.539x_6 + 2157.51x_7 + 7.079x_8 - 2244.564$	0.999	1.307	(4.5)	(4.1)
● Biological unit for:					
* Conventional Activated Sludge	$y=0.085x_1 + 388.671x_2 - 8.177x_3 + 18.563x_4 + 125.306x_5 + 2.381x_6 + 9567.867x_7 + 30.655x_8 - 17542.249$	0.990	318.453	(4.6)	(4.2)
* Extended Aeration	$y=0.24x_1 + 894.704x_2 - 16.447x_3 + 390.626x_6 + 27283.591x_7 + 88.24x_8 - 45399.04$	0.990	318.453	(4.7)	(4.3)
* Oxidation Ditch	$y = 0.336x_1 + 751.73x_2 - 20.993x_3 + 535.716x_5 + 9.732x_6 + 38947.973x_7 + 127.509x_8 - 53857.893$	0.999	157.590	(4.8)	(4.4)
* Aerated Lagoon	$y = 2.593x_1 + 960.753x_3 + 75.504x_6 + 302051.494x_7 + 991.121x_8 - 314248.998$	0.999	183.025	(4.9)	(4.5)
* Aerobic Ponds	$y = 3.312x_1 - 367.97x_3 + 95.125x_4 + 95.94x_6 + 384023.99x_7 + 1256.31x_8 - 22109.74$	0.999	1799.83	(4.10)	(4.6)
* Anaerobic Ponds	$y = 0.666x_1 + 247.08x_3 - 0.721x_4 + 19.4x_6 + 77609.607x_7 + 254.692x_8 - 80745.1$	0.999	43.046	(4.11)	(4.7)
* Facultative Ponds	$y = 8.211x_1 + 11761.05x_2 + 747.26x_3 + 2029.475x_4 + 8688.11x_5 + 259.625x_6 + 1030116.092x_7 + 3529.974x_8 + 249989.204x_{10} - 1580812.247$	0.915	63112.57	(4.12)	(4.8)
* Trickling Filters	$y = \exp(6.733E - 006x_1 + 0.032x_2 - 0.006x_3 + 0.0007x_4 + 0.025x_5 + 0.0003x_6 + 1.021x_7 + 0.004x_8 + 0.169x_9 + 7.473)$	0.988	782.486	(4.13)	(4.9)
* Rotating Biological Contactor	$y = \exp(6.621E - 006x_1 + 0.033x_2 - 0.006x_3 + 0.001x_4 + 0.025x_5 + 0.0003x_6 + 0.956x_7 + 0.003x_8 + 7.417)$	0.986	566.172	(4.14)	(4.10)
* Bio-Towers	$y = \exp(6.872E - 006x_1 - 0.035x_2 - 0.003x_3 + 0.0004x_4 + 0.02x_5 + 0.0003x_6 + 1.101x_7 + 0.004x_8 - 1.13x_9 + 7.649)$	0.994	30.036	(4.15)	(4.11)
● Final Settling Tanks for:					
* Conventional Activated Sludge	$y = \exp(1.06E - 005x_1 + 0.021x_2 + 0.003x_3 + 0.001x_4 + 0.008x_5 + 0.0003x_6 + 1.474x_7 + 0.006x_8 + 6.982)$	0.971	1739.669	(4.16)	(4.12)

* Extended Aeration	$y = \exp(1.166E - 005x_1 + 0.017x_2 + 0.003x_3 + 0.009x_5 + 0.0005x_6 + 1.63x_7 + 0.008x_8 + 7.143)$	0.982	2549.577	(4.17)	(4.13)
* Oxidation Ditch	$y = 0.166x_1 - 4.27x_2 + 58.937x_3 + 4.816x_5 + 5.057x_6 + 17888.42x_7 + 75.289x_8 - 21907.271$	0.991	336.795	(4.18)	(4.14)
*Aerated Lagoon	$y = 0.44x_1 + 405.068x_2 + 136.604x_3 + 235.451x_5 + 14.032x_6 + 56780.946x_7 + 263.88x_8 - 72377.566$	0.938	2499.616	(4.19)	(4.15)
* Attach Growth System	$y = 0.035x_1 - 0.01x_2 + 12.809x_3 - 0.016x_4 + 0.003x_5 + 1.007x_6 + 4027.56x_7 + 13.219x_8 - 4189.203$	0.999	1.917	(4.20)	(4.16)
● Gravity Thickeners for:					
* Conventional Activated Sludge	$y = 0.008x_1 + 9.834x_2 - 0.96x_3 + 8.165x_4 + 3.449x_5 + 0.218x_6 + 872.306x_7 + 2.839x_8 - 1065.754$	0.997	8.954	(4.21)	(4.17)
* Stabilization ponds	$y = 0.006x_1 - 0.357x_3 + 7.146x_4 + 0.167x_6 + 666.502x_7 + 2.181x_8 - 666.04$	0.999	2.89	(4.22)	(4.18)
* Attach Growth System	$y = 0.008x_1 + 5.945x_2 - 0.648x_3 + 7.253x_4 + 4.245x_5 + 0.237x_6 + 949.561x_7 + 3.11x_8 - 1064.7$	0.999	3.646	(4.23)	(4.19)
● Anaerobic Digesters for:					
* Conventional Activated Sludge	$y = 0.028x_1 + 21.362x_2 - 2.09x_3 + 17.849x_4 + 7.5x_5 + 0.475x_6 + 1904.150x_7 + 10.631x_8 - 2435.935$	0.999	19.459	(4.24)	(4.20)
* Stabilization ponds	$y = 0.021x_1 - 1.64x_3 + 6.786x_4 + 0.286x_6 + 1137.994x_7 + 8.245x_8 - 506.004$	0.954	92.772	(4.25)	(4.21)
* Attach Growth System	$y = 0.026x_1 + 10.359x_2 - 1.191x_3 + 12.942x_4 + 7.406x_5 + 0.417x_6 + 1667.297x_7 + 9.895x_8 - 1979.069$	0.999	6.577	(4.26)	(4.22)
● Holding Tank for:					
* Extended Aeration	$y = 0.004x_1 + 14.314x_2 - 0.263x_3 + 6.25x_5 + 0.109x_6 + 436.256x_7 + 1.41x_8 - 726.229$	0.997	7.282	(4.27)	(4.23)
* Oxidation Ditch	$y = 0.007x_1 + 15.034x_2 - 0.42x_3 + 10.714x_5 + 0.195x_6 + 778.914x_7 + 2.55x_8 - 1077.158$	0.999	3.151	(4.28)	(4.24)
*Aerated Lagoon	$y = 0.124x_1 + 46.12x_3 + 3.62x_6 + 14498.11x_7 + 47.57x_8 - 15083.68$	0.999	8.77	(4.29)	(4.25)

2- Total Air Required (m³/min) for:					
* Conventional Activated Sludge	$y = 0.005x_1 - 7x_2 - 0.526x_3 + 1.086x_4 + 7.712x_5 + 0.155x_6 + 621.325x_7 + 2.05x_8 - 471.144$	0.996	6.426	(4.30)	(4.26)
* Extended Aeration	$y = 0.011x_1 - 13.605x_2 - 0.768x_3 + 17.41x_5 + 0.317x_6 + 1265.802x_7 + 4.183x_8 - 814.638$	0.997	14.771	(4.31)	(4.27)
* Oxidation Ditch	$y = 0.021x_1 - 13.428x_2 - 1.352x_3 + 33.446x_5 + 0.623x_6 + 2489.264x_7 + 8.25x_8 - 2181.973$	0.994	39.268	(4.32)	(4.28)
* Aerated Lagoon	$y = 0.013x_1 - 21.168x_2 - 0.991x_3 + 21.133x_5 + 0.386x_6 + 1541.046x_7 + 5.124x_8 - 1086.653$	0.993	29.024	(4.33)	(4.29)
3- Quantity of Gas Produced (m³/d) for:					
* Conventional Activated Sludge	$y = 0.027x_1 + 19.844x_2 - 1.937x_3 + 16.482x_4 + 6.963x_5 + 0.439x_6 + 1759.839x_7 + 10.208x_8 - 2262.922$	0.999	18.063	(4.34)	(4.30)
* Stabilization ponds	$y = 0.025x_1 - 0.821x_3 + 16.452x_4 + 0.383x_6 + 1533.029x_7 + 9.496x_8 - 1644.918$	0.999	6.733	(4.35)	(4.31)
* Attach Growth System	$y = 0.031x_1 + 13.678x_2 - 1.492x_3 + 16.699x_4 + 9.778x_5 + 0.546x_6 + 2184.79x_7 + 11.636x_8 - 2562.554$	0.999	8.535	(4.36)	(4.32)

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