DEVELOPMENT OF A PILOT STATION FOR EVALUATING KERBALA WATER TREATMENT PLANT,IRAQ.

A thesis

Submitted to the Council of College Engineering, University of Kerbala in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

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بِسْمِ اللَّهِ الرَّحْمَٰنِ الرَّحِيمِ

"قَالُوا سُبْحَانَكَ لَا عِلْمَ لَنَا إِلَّا مَا عَلَّمْتَنَا ۖ إِنَّكَ أَنتَ الْعَلِيمُ الْحَكِيمُ"

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ABSTRACT

The development and a improve efficiency of water purification plant in Kerbala was investigated.

Research mechanisms were established to improve the efficiency of the station production in terms of quality and quantity to meet the standard requirements to be developed within locally and accepted alternatives.

The study has addressed theoretical analysis and a series of previous tests results through data available at the station from 2014-2019. The model was built and integrated matching the conventional water treatment plant using four values of flow rate (0.475,0.712,0.95,1.18 m^3 /hr) using different turbidities.

This model consists of basic units coagulation, flocculation and sedimentation . By using a dynamic similarity application between the model and prototype , the Froude number must be the same for both model and prototype .

The bentonite soil as well as river soil were used for preparing the required turbidity. A high-rate sedimentation unit was considered in a pilot plant using plates settler and compared with the conventional. Dual media filter with anthracite was added as well as a single filtration unit that existing in plant, with same dimensions, where the layer deep was (0.7 m) divided to (0.35m) sand and (0.35m) anthracite.

The activated carbon filters were linked after the single filter media and the other one after the dual filter media. Best velocity gradient in rapid mixing tank is (G_{rapid}) $\geq 750\ s^{-1}$ and in flocculation basin is $15\ s^{-1} < G_{flocc} < 60\ s^{-1}$ and values of (Gt_{flocc}) = [(10^4) -($15*10^4$)]. The best efficiencies of removal in sedimentation basin with plate settler were 94.07% and 72.07 % using bentonite and river soil respectively. The enhancement in removal efficiency

is about 23% when using bentonite. When using river soil, The best removal efficiencies in sedimentation basin were 74.02% and 66.28% with and without using inclined plate settler respectively.

The enhancement in removal efficiency was about 11% when using plate settler. The efficiency of removal using inclined plates was best especially when doubling the flow rate (1.5-2.5) times. The best conditions when flow rate = 2100 m^3/hr, turbidity at 50 NTU, using river soil, with inclined plate settler, dual filter media and activated carbon filter where the efficiency of removal was 98.23%, while the efficiency of removal using bentonite ,without using inclined plates ,using activated carbon filter and dual filter media at turbidity 200 NTU ,flow rate1050m^3/hr, it was 99.27%.

This indicate ,when using bentonite for Increasing water turbidity has a positive impact in removal efficiency.

In conclusion, a significant increase of (150% -250%) in the production of station, efficiency and moderate economic cost.

Key Words: Plate Settler, Activated Carbon, Bentonite, Turbidity, Removal Efficiency and Water Quality Index.

Dedication

To appreciable god gift·

To warm -hearted persons.

To my parent ,Ibrahim's Father & Mother·

To my brothers and sister·

I dedicate this modest effort·

Ibrahim

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NOTATIONS

DMF	Dual Media Filter
SMF	Single Media Filter
μC	Micron (One thousandth of a millimeter).
G	Velocity Gradient (T^{-1}).
G.t	Camp Number (dimension less)
r.p.m	Revolution Per Minute .
G_{floc}	Velocity Gradient in Flocculation Tank (T^{-1}).
G_{mix}	Velocity Gradient in Rapid Mixing (T^{-1}).
(G_{floc}) opt.	Optimum Velocity Gradient in Flocculation Tank (T^{-1}).
EPA	Environmental Protection Agency .
DAF	Dissolved Air Flotation .
Vs	Settling Velocity.
t_0	Retention Time .
Q	Discharge,(L^3/T) or (M^3/T).
E	Efficiency .
N	Number of Plate Settler.
Ah	Horizontal Area.
Ls	Length of the Plate Settler.
e_p	Thickness of the Plate .
AT	Total Area .
θ	Angle of Inclination of Plate (degree).
D	Spacing Between the Plates .
$Q_{opt.}$	Optimum Angle of Inclination of Plate (degree).

SOR (V_o)	Surface Over Flow Rate .
TRE	Turbidity Removal Efficiency .Percent
Sc	Shape Factor.
Lr	Plate Settler Length /Distance Between Two Plates .
Re	Reynolds Number.
R	Hydraulic Radius.
ν	Kinematic Viscosity.
β	Constant .
Vh	Horizontal Velocity .
Fr	Froude Number.
g	Acceleration of Gravity .
ρsl and ρ	The Mass Density of Sludge and Fluids.
k	Constant.
Vf	Linear Velocity.
Vsl	Sludge Bed .
Es	Size of the Sieve in mm .
Uc	The Ratio Between the Sieve Size .
th	The Length of Time to Limiting Head Loss .
tb	The Length of Time to Turbidity Penetration .
$\emptyset$	Filter Coefficient in Model for Removal Depth .
L	Depth in the Filter Bed .
C	Particle Concentration.
d	Grain Size.
μ	Dynamic Viscosity of the Fluid .

GAC	Granular Activated Carbon .
RGF	Rapid Gravity Filters
CMF	Capping Sand Filters.
AC	Activated Carbon.
PAC	Polyaluminium Chloride.
PH	Is a Measure of the Acidity or Basicity of a Solution .
WHO	World Health Organization.
TDS	Total Dissolve Solids .
ZP	Zeta Potential.
P	Power Input Per Unit Volume.
A	Cross Section Area (m^2) .
ρw	Density of Water .
F_D	Drag Force .
V_r	Velocity of Paddles Relative to the Fluid in Rank .
A_p	Cross –Sectional Area of Paddles Perpendicular to Direction of Motion .
CD	Drag Coefficient for the Paddles.
V_S	Terminal Velocity (m/hr.).
d	Diameter of Particles (mm).
F_g	Force of Gravity (m/s^2).
ρ_p	Density of Particle(K_{g/m^3}).
$\forall_p$	Volume of Particle(m^3).
F_b	Buoyant force .
P_O	Partial Mass of Particles.

V_f	Average Flow Velocity.
V_{sc}	Particle Settling Velocity .
K	A Factor Allowing for Consideration of the Wall Thickness Plus any Dead Spaces.
t	Thickness of Plate.
D	Spacing Between Two Plates.
H	Depth of Sedimentation Tank.
$V_{\bar{o}}$	Critical Settling Velocity with Plate Settler.
V_o	Critical Settling Velocity Without Plate Settler.
W	Plate s Width .
Co	Concentration of Suspended Solids in the Inflow to.
L	The Length of the Settling Basin.
W	The Width of the Settling Basin.
D	The Depth of the Settling Basin .
H	The Differential Elevation Statistical Parameter Characterizing the Distribution of the Settling Velocity (W).
r	Radius of Basin ,(m).
V_{sp}	Settling Velocity of Prototype Units.
V_{sm}	Settling Velocity of Model Units .
λV_s	Scaling Factor .
R_p	Radius of Tank of Prototype of Units.
R_m	Radius of Tank of Model of Units.
λ_R	Scaling Factor of Radius of Tank.
A_p	Area in Prototype .

A_m	Area in Model.
λ_A	Scaling Factor of Area.
h_p	Depth of Tank in the Prototype of Units.
h_m	Depth of Tank in the Model of Units
λ_h	Scaling Factor of Depth.
Q_p	Flow Rate in the Prototype of Units.
Q_m	Flow Rate in the Model of Units.
λ_Q	Scaling Factor of Flow Rate.
t_p	Detention Time in the Prototype of Units.
t_m	Detention Time in the Model of Units.
λ_t	Scaling Factor of Detention Time .
$(Vr)_p$	Rotational Velocity in the Prototype of Units.
$(Vr)_m$	Rotational Velocity in the Model of Units.
λv_r	Scaling Factor of Rotational Velocity .
HMP	Heavy Metal Pollution Index.
NSF WQI	National Sanitation Foundation Water Quality Index .
OWQI	Oregon Water Quality Index.
BCWQI	British Columbia Water Quality Index.
CCME	Canadian Council of Ministries Water Quality Index

CHAPTER ONE

Introduction

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1. Introduction

All waters, contents of many impurities such suspended solid and dissolved particles. the source of these impurities are mostly from the dissolution of minerals , industrial waste discharges, decay of vegetation and dead animals ,earth erosion and domestic waste discharges. Also it may consist of organic and inorganic materials.

Many of biological organisms, like bacteria ,algae, viruses , and protozoa present in water cause deteriorate water quality, as well as cause environmental problems which are harmful to public health.

Thus, it should be removed by suitable methods to get of suitable water for drinking and use it for various domestic and industrial purposes.

For separating the suspended and dissolved particles both of coagulation and flocculation processes are used in water treatment .

The main objective is to enhance the separation of particulate in sedimentation and filtration units . There are several factors that the application of coagulation and flocculation depends upon like particle size ,shape, source of suspended particles, particle charge, and density. Suspended solids has a negative charge in water . Because of these negative charge , they repel each other. therefore the suspended solids will not associate each with each other to settle and will remain in suspension, thus needs a proper coagulation and flocculation is employed. (Prakash et.al., 2014).

Coagulation can be defined as the process of destabilization by charge neutralization. The neutralized particles means there is no repel of particles each other and it can be brought together. Coagulation is important in removal of the colloidal-sized suspended matter. Many of chemical coagulant, such as aluminum salts, iron salts or polymers, are used to added to the source water to make bonding among particulates. Flocculation is the process of add energy to the Brownian diffusion energy through a mechanical mixing process that causes the formation of : Orthokinetic " chemical flocculation . The agglomeration of particles comes from the rate of collisions between the colloids and bringing about the attachment and aggregation of the particles into larger and denser floc. and are more amenable to separate by settling or filtration. (Weber et. al.,1970). Sedimentation process comes after coagulation and flocculation processes , which carried out by settling under gravity in sedimentation basin .

1.1 Problem Statement

During the past decades that passed through wars and blockades in Iraq, and as a result of the lack of funds for the establishment and development of drinking water treatment plants, these plants have suffered from a build-up of operating problems and a lack of equipment and vital equipment that go into the continuity of the work of these plants , decreased water quality .

Increasing water demand for domestic consumption is due to increase in population and special development some devices and equipment that require the use of water to operate and maintain it .

All of these problems requires the establishment of new water treatment plants for drinking purposes with modern technology that contributes to bridging the shortage of supplied water, in return, the new construction can be compensated and the cost reduced by developing and improving the performance of the existing plants by increasing the quantity and quality of water required for processing.

Above extensions and increases do not need to add any facility such as filters or any of the processing units. Most of the water treatment plants were established during the past decades to provide all households in the country with a supply of clean and safe drinking water, however a large number of them are of productivity and quality that do not suit the purpose for which they were implemented and for one reason or many reasons including

(Swartz C.D.,2000):Implementation and installation of non-standard treatment systems.

The operation of the plant by people who have no knowledge of the basic principles of water treatment and thus problems arise in the quality and quantity of treated water without knowing the real reasons behind it.

Financial constraints that caused the deterioration of the plant as a result of the inability to meet the deficiencies and damages that result from the operation of the plant, which leads to the accumulation of problems and malfunctions in the plant, causing water treatment failure as required.

Deficiencies in the design and construction of the stations, which led to their neglect during operation. With regard to the problems of water treatment plant in Kerbala, Iraq, there are many problems were monitored below.

The increasing demand for water, especially during religious events, to which very large numbers of visitors are flocking to it and requires the provision of very large quantities of water. Due to establishment of numbers of random zones ,which requires the provision of very large quantities of water. The data of on the laboratory tests of the water treatment plant in Kerbala for the period from 2014 -2019 ,indicate that the station is working almost remove turbidity only, while the remain of the parameters have not changed except slightly.

Because of large excesses on the network of water transport lines , requiring supplied more quantity of water that cannot be provided because they did not taken into consideration in the design plan.

Because of the result of global warming and low rain, led to decreased of water in the stream of rivers ,therefor release of the water stored in barrage and lakes to the stream of rivers. This stored water is stagnant and low turbid water but the concentration of solids was high , this phenomenon didn't take into consideration when designing water stations currently faced.

The presence of plants within the sedimentation basins is noted and as shown in the plate (1-1) . Also there is nothing to compensate for the lack of media in filter from start of operation station till now .

Plate (1-1): The appearance of plants inside the sedimentation basin at the water treatment plant in Kerbala.

Therefore, it became necessary to develop the units of the plant to raise their efficiency and raise water quality.

1.2 Aim and Objective

The aim of the current work is to evaluate and enhance water treatment plant in Kerbala governorate (Iraq) by developing a pilot plant for adoption a specific quality and quantity of the requirements for drinking water. This can be done through the following objectives :

1. Changing the flow rate of rapid mixing unit using optimum values of velocity gradient, detention time, aluminum sulfate dose.

2. Investigate the velocity gradient to obtain the optimum range of flocculation velocity gradient for flocculation process.
3. To improving the performance of the sedimentation process using plates inside the sedimentation tank.
4. Utilizing of high-rate dual media filters (DMF) as an alternative to the dominant single media filter (SMF) in Kerbala treatment plant .

1.3 Organization of Thesis

This thesis consists of five chapters. Chapter one gives an introduction that includes the problems present in the plant's facilities and how to solve them with the lowest cost and best method, in addition to studying increasing the quality and quantity of water. Chapter two gives literatures review about the main processes of water treatment plant including discuss developing of the pre-filtration units and filtration units. Chapter three describes the theoretical and analytical frame-works including details of Kerbala water treatment: Plant units model, equipment, materials, testing, experimental works design, and procedures.

In chapter four, the result of experimental work and testing are presented and discussed. Finally chapter five deals with the conclusion ,discussion and recommendation for future studies.

Chapter two

LITERATURE REVIEW OF DEVELPOMENT OF THE PREFILTRATION, FILTRATION UNIT PROCESSES AND THEORTICAL ASPECT

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LITERATURE REVIEW OF DEVELPOMENT OF THE PREFILTRATION,FILTRATION UNIT PROCESSES AND THEORTICAL ASPECT

2.1 Introduction

The primary goal of water treatment is to achieve a sufficient and constant supply of bacteriologically, chemically, and physically acceptable water. Water treatment facilities are made up of a number of interconnected unit processes, one of which is filtration, and the plant's overall performance is determined by all of them. In addition to the filtration process, the high rate filtration unit requires the best and highest performance of pre-filtration unit processes (coagulation, flocculation, and sedimentation) to get high quality and quantity water products. The necessity of proper working circumstances for high-rate pre-filtration unit operations as well as the filtration process is discussed in this chapter in order to satisfy the needs of improving the quantity and quality of water production by studying the relevant literature.

2.2 The Coagulation - Flocculation Process in water treatment plants (WTPs)

The major operations to remove suspension materials and colloids from water (Degremont,1991):

1. Coagulation: The process of destabilization of the colloidal substance through the rapid mixing after the addition of chemical factors is known as coagulation and flocculants, respectively, to achieve the bonding or adsorption mechanism , leads to their agglomeration. It is considered the first stage in treatment train operations, followed by flocculation, sedimentation, and filtration to achieve liquid-steel separation (Ghernaout, 2020).

2. Flocculation: The process is a slow mixing in which the destabilizing colloidal particles are come into contact with intimate in order to enhance their agglomeration. The rate of flocculation is based upon to the number of particles presents, the velocity gradient , and the relative volume it accommodates (Steel and Terence J. Mc Ghee). This process depends on turbulence to obtain and reinforce collisions (Peavy et al.,1985). After the physical process of mixing and adequate flocculation, most of aggregates will settle out during (1-2) hr of sedimentation, therefore, flocs are formed and can be readily removed by settling or filtration (Marais and Ekama,1986).

2.3 Surface Charge of Colloidal Suspensions

The suspended particles differ in form, composition, source, size, density, and charge. The solids suspended in the water have a negative charge and repel each other when they come close to each other as they have the same surface charge, so the particles will remain in a suspended state instead of clumping together and settling out of the water. For the purpose of making these particles converge and attract to each other, the coagulation process (the first in the water treatment chain) must be applied which will cause instability and destabilization of the particles and make them approach and cohere with each other. Negative charges on the surface of particles and electrical forces are keep the individual particles separated from each other as a result of repulsion forces and thus colloids remain in a suspended state as small particles. (Binnie et al.,2002). Colloids in natural water has negatively charged is predominate.

This negative charge will attracts to its surface ions of the opposite charge.

A compact layer over the surface of colloid is called the stern layer or fixed layer. Ions has a second layer, known as the diffused layer which attracted to the colloid. In the diffused layer ,ions of both charge are attracted.

The stern layer and diffused layer together are called the double layer. The molecules of water in the diffused layer are sufficiently bound to make a shear surface or slipping plane.

The Figure (2-1) shown that the electrical potential at the slipping plane is known as zeta potential. Zeta potential (Zp) can defined as the differences of electric potential across the ionic layer surrounding a charged colloid ion. The value of the zeta potential (Zp) is usually used to refers to colloidal particle stability. When the magnitude of zeta potential is higher ,it means the more stability of the colloidal particles and there is a strong forces of separation(via electrostatic repulsion).While the magnitude of zeta potential is low its indicative of unstable system i.e. particles tends to coagulate or flocculate as outlined in the Table (2-1). (Reynolds and Richards, 1996).

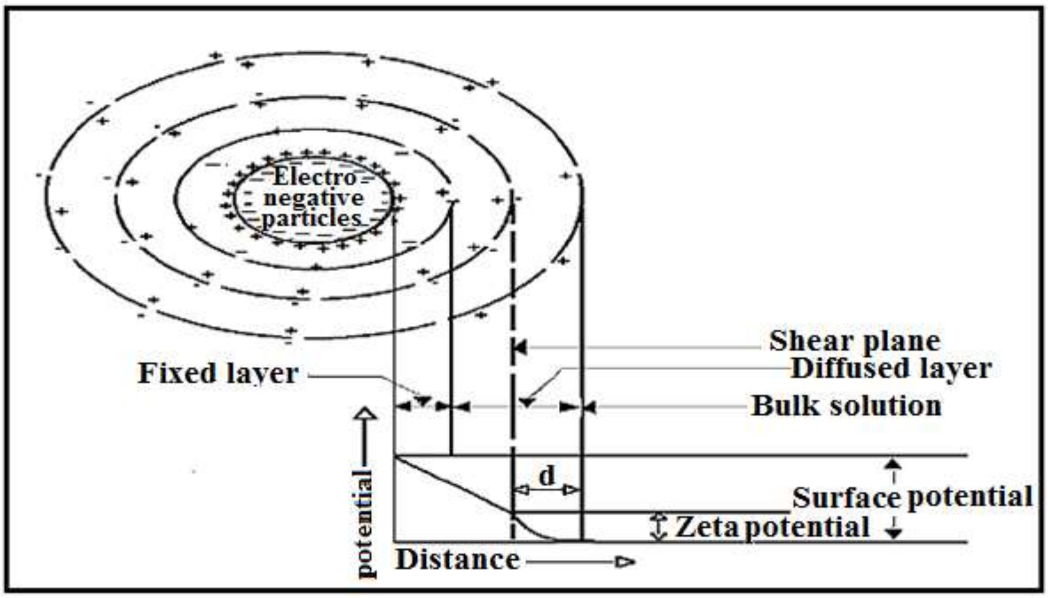


Figure (2-1) :A negative colloidal with its electrostatic field (Reynolds & Richareds,1996).

Table (2-1): Degree of stability of the colloid depends on the stability of the zeta potential (mV). (Reynolds and Richards, 1996).

Zeta potential [mv]	Stability behavior of the colloid
From 0 to ± 5	Not stable (Rapid coagulation or flocculation)
From± 10 to ± 30	Incipient instability
From± 30 to ± 40	Moderate stability
From± 40 to ± 60	Good stability
More than ± 60	Excellent stability

2.4 Mechanism of Coagulation

Addition of inorganic chemical coagulants such as iron and aluminum salts. When added to water, the aqueous iron and aluminum salts separate into the trivalent aluminum ion and the trivalent iron ion. These ions dissolve with water and form soluble aggregates with high positive charges. These soluble agglomerates are attracted to the surface of the negatively charged colloids by absorbing them (Matilainen et al., 2010). There are four different mechanisms by which coagulation can be achieved:

2.4.1 Double-Layer Compression

The double layer compression mechanism is based on the pressure of the diffuse layer surrounding the colloid. This is achieved by increasing the ionic strength of the solution by adding an indifferent electrolyte. The charge density in the diffuse layer increases when added electrolyte. The diffuse layer becomes thinner as a result of "pressing" it towards the particle surface. Therefore, the zeta potential, Zp, is greatly decreased (Reynolds and Richards 1996).

2.4.2 Adsorption and Charge Neutralization

Some chemicals are able to be absorbed on the surface of colloidal particles. Provided that the charge of colloids is opposite to the charge of the adsorbed species, this absorption reduces the surface potential, causing the destabilization of the colloidal particles. In nature, destabilization by adsorption is a stoichiometric. Thus, the higher the concentration of colloid in the water, the higher the dose of the coagulant is required. In the case of increasing the dose of the absorbable species, the suspended particles may destabilize as a result of the charge reversal on the colloidal particles.

2.4.3Enmeshment by a Precipitate (Sweep-Floc Coagulation)

Chemical coagulants such as aluminum sulfate ($Al2(SO4)_3$), ferric chloride ($FeCl_3$), used to form the precipitates of $Al(OH)_3$ and $Fe(OH)_3$.

These precipitates physically Surround the suspended particles as they settle, especially during flocculation. The flocs are formed around colloidal particles when the colloidal particles themselves serve as nuclei for the formation of precipitate , and can be enhanced the sweep- floc coagulation process. Thus the sedimentation rate increases as the concentration of colloidal particles (turbidity) in the solution (Binnie et al.,2002). The process of particle instability by neutralizing charge to remove less particles than by removing particles using sweeping flocculation is generally. The reason is the greatly improved agglomeration rate, which is due to the increased concentration of solids. Any increase of coagulant dosage in the sweep area leads to gradually increase of sludge volume , but in addition to optimal operational dose, there is slight improvement in particle removal. (Duan and Gregory, 2003).

Figure(2-2) shows the functions of alum and how it used as a coagulant to remove the high turbidity from water (greater than 100 NTU).

Low doses of alum cannot reduce the turbidity value, because there is not enough aqueous aluminum(III) species that can provide effective destabilization.

Any increasing of alum dose, turbidities will be decrease to a minimum value, and the destabilization occurs completely. The mechanism of adsorption and charge neutralization will be the govern at this stage.

A Zp which is near zero may be corresponds to optimum dosage often (but not always) . An increase in the dose of alum will lead to the opposite result of stabilizing suspended particles, and this is due to the charge reflection on colloids. Too high added doses of alum lead to the formation of a precipitate of aluminum hydroxide ($Al(OH)_3(s)$), because the amount of Al (III) in aluminum sulfate (alum) added to water cannot completely dissolve, because it has reached a point Saturation (exceeding the solubility limit of the hydroxide).

This huge precipitate entrap particles and settle down rapidly forming the "sweep-floc" region of coagulation (Sanks ,1979).

Low turbidity in water (no more than 10 NTU) cannot be removed , alum polymers cannot remove turbidity by adsorption and neutralized due to insufficient contact opportunity.

The removal process is govern by sweep-floc. coagulation (Sanks, 1979). High raw water turbidity may needs a lesser amount of coagulant for performed a good coagulation ,while raw water with low turbidity may need more amount of coagulant . For this reason it is sometimes advantageous to add turbidity to relatively clear water. Bentonite clay is generally used for this purpose .(Peavy et al.,1985).

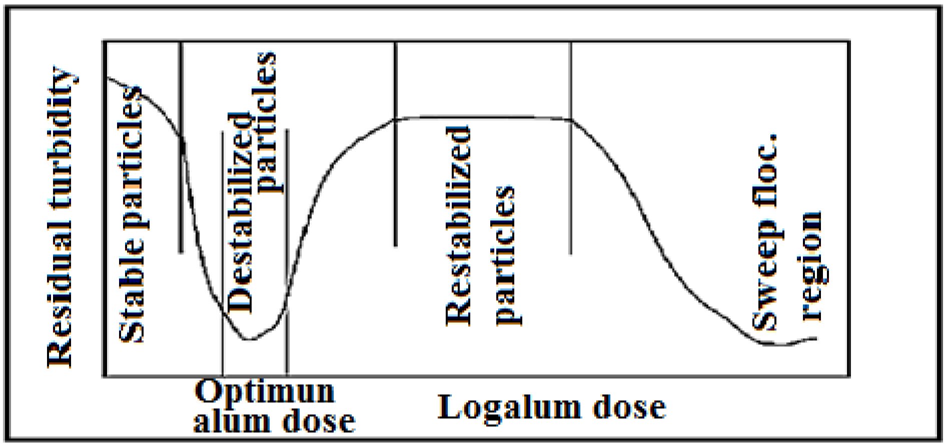


Figure (2-2):Alum dose versus water turbidity for coagulation /flocculation (Snoeyink and Jenkins1980).

2.4.4 Interparticle Bridging

Synthetic polymeric compounds are coagulants that are effective in destabilizing suspended matter and colloids in water. These coagulant polymeric materials are described as having a large molecular size and having multiple electrical charges along the molecular chain of carbon atoms. The bridging process was summarized by (Bagwell et al.,2001) as follows:

The simplest form of bridging shown in Figure 2-3(a). a colloidal particle will attach to a polymer molecule at one or more sites. Colloidal attachment occur due to the columbic attraction when the charges are of opposite charge or from hydrogen bonding, van der Waals forces , or from ion exchange. The second reaction demonstrated in figure 2-3(b),the length of the polymer molecule remaining from the colloid particle in the first stage of the reaction extends out into the solution. If a second particle having some vacant adsorption sites contacts , then the attachment can occur to form a bridge. Therefore , the polymer works as the bridge.

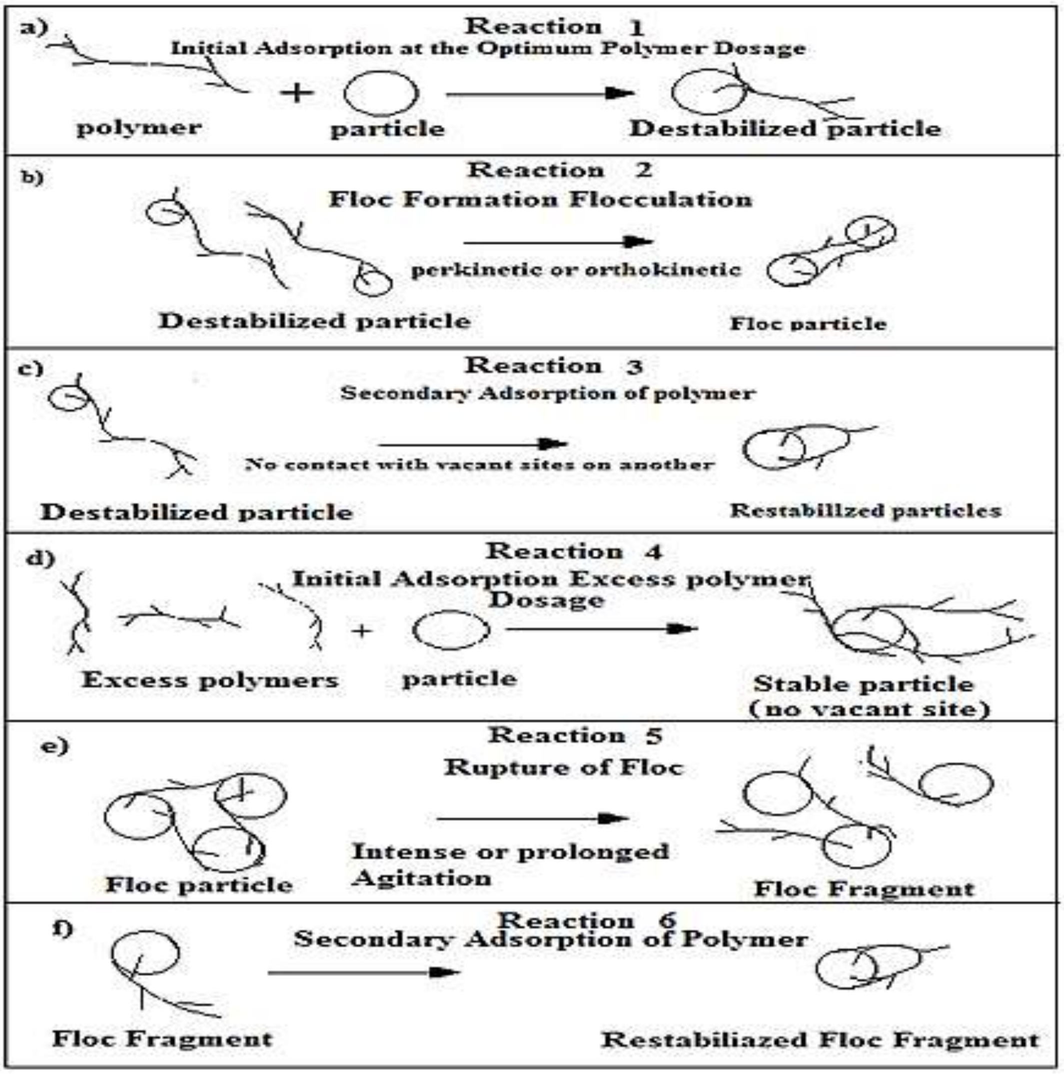


Figure (2-3): Schematic representation of bridging model for destabilization of colloids by polymers (Bagwell et al., 2001).

In the event that the expanded polymer does not come into contact with another particle, the polymer will bent on itself and absorbed on its surface as shown in the figure 2-3 (C). The original particle is destabilized. If the amount of the polymer dose is exceeded, the piece of polymer may saturate the colloidal surfaces, so that no sites are available on the surfaces for bridging among particles. This reaction can be shown in figure 2-3(d) causes particle restabilization. restabilization of the particles also occurs when there is severe stirring in the solution that causes destruction of bridges or bonds formed from the surface of the polymer. These interactions can be seen in Figure 2-3 (e) and

2-3(f). Equation (2-1) demonstrated that the coagulant salts release hydrogen ions when it hydrolysis in water .These hydrogen ions react with alkalinity and neutralize alkalinity.

When added of (1mg/L) of alum will result hydrogen and will neutralize (0.5Mg/L) of alkalinity. If the water contain low alkalinity ,the excess reduction will damage its buffering. The capacity and (pH) values will reduce rapidly. For getting of best coagulation ,it should be maintained of the value of (pH), also the alkalinity must be exist for formation of hydroxide floc. It should be artificially buffered when the waters alkalinity are low. This is usually done by adding of lime [Ca(OH)2]or soda ash (Na2CO3) (Peavy et., al 1985).

$$Al_2(So_4)_3+12H_2o \longrightarrow 2Al(H_2O)_6{}^{+3} + 3So_4{}^{-2} \quad \text{-----------} \quad (2\text{-}1)$$

2.5 Flocculation Kinetics

After the coagulation process, the flocculation process begins through quiet mixing, as the size of the particles that were previously infinitesimally small materials not visible to the naked eye increases to visible suspended particles. This mixing process causes the tiny particles to converge and thus come into contact with each other.

The collision takes place between the particles, linking them together to become of a larger size and visible masses called a micro- Floc. As a result of additional collisions and interaction with inorganic polymers (formed as a result of the coagulant) or as a result of adding organic polymers, the particle size continues to increase. Coagulation aids that are high molecular weight polymers can also be added, this step will cause an increase in mass weight and aid in formation Bridging and linking the mass and thus increasing the sedimentation rate, so that the water is ready for sedimentation process (MRWA,2003). Flocculation transfer is carried out through three main mechanisms, as shown below:

1. Perikinetic flocculation is the agglomeration of small particles caused by random thermal motion and collide with other particles (Brownian diffusion).The thermal energy of fluid caused particle movement is the driving force. These particles are so small, their size is less than approximately1µm in diameter (Han and Lawler ,1992). This mechanism leads particles to be continually moving inside the water and can caused collisions between two particles.
2. Orthokinetic flocculation: The induced energy in the fluid cause the agglomeration of particles .The destabilized particles moves with the streamlines and eventually result contacts between particles (Binnie et al., 2002). Han and Lawler ,1992 shows that orthokinetic flocculation most likely occurs when the size of two particles are greater than approximately 1µm in diameter and have similar in size (within a factor of 10 in size ratio).
3. Differential settling occurs due to the different settling velocities of particles. The particle size proportional with settling velocity of particles which have same densities , in nonhomogeneous suspension of differential particles gives additional transport for promoting flocculation. It's often likely happened when at least one of the flocculated particles diameter is larger than 10 µm and the other is different in size (Han and Lawler 1992), (Thomas et al.1999). A summary of what was added above, when a colloidal suspension has been destabilized, primary floc. particles are formed and grow in size through contact with other particles as a result of Brownian motion .

This process is sometimes called "Kinetic flocculation". As particles grow in size the influence of Brownian effects is diminished and the rate of particle aggregation correspondingly reduced. To accelerate the rate of particle collision, velocity gradients are created within the body of dispersing fluid. This controlled use of the velocity gradient to promote flocculation is sometimes called "Orthogenetic flocculation" (Casey,1997).

2.6 Coagulant Chemicals

Estimating the required quantitative of chemical doses in water treatment depends on many factors such as salt concentration, type of coagulant, pH value, temperature, size of particle, nature of the colloids, mixing, alum concentration, bench-scale, experimental tests, including the jar test to determine the susceptibility to treatment and estimating (Ramaley et al.,1987) ,(O'Melia,1985) and (Weisner et al.,1987). One of the most common and used types of coagulants are:

- Alum (aluminum sulfate), $AL_2(SO4)_3$ • 14H20.
- Polyaluminum chloride, AL(OH)x(CL): Use it in some waters, needs to adjust pH lower and the production of a small amount of sludge
- Ferric chloride, $FeC1_3$: In practical applications it is more effective than the alum .
- Ferric sulfate, $Fe_2(SO4)_3$: More economical and effective in some water.
- Cationic polymers can be used with iron or aluminum coagulant, or used alone as primary coagulant .

Alum is known to be one of the most common coagulant chemicals used in water treatment plants, but ferric chloride or ferric sulfate has a better stable mass formation in the treatment of some waters due to its greater effectiveness in constantly removing organic matter compared to coagulants that contain aluminum. In addition, polyaluminium chloride often produces stable mass in cold water and a better shape, thus producing lower doses and less sludge from iron and alum residues (Edward .E. Baruth, 2004).

2.7 The Requirements of Rapid Mixing

For the purpose of dispersing the coagulants chemicals in the entire water and its rapid homogeneity within a short period of violent excitement, mixers of the highest possible speed should be provided. This makes the coagulation process as effective as possible (Hudson ,1981). Because the reactions of the coagulants are rapid, it is best to disperse the chemical quickly through rapid mixing, before the reactions are completed, (Syed R.Q et al., 2002). So when designing the rapid mixing unit careful attention must be paid when designing it (Peavy et al.,1985).

Designing of a flash mixing unit is accomplished based on design parameters, which are taken from jar test.

There are many parameters for design rapid mixing are: type of chemical, chemical dose, velocity gradient and mixing time, (Dharmappa et al.,1994).

Syed R.Q et al.,2002 shows that the design value of the velocity gradient is based on the geometry of the mixing unit, dosage rate of coagulant ,detention time the in mixing unit, velocity gradients value about (700-1000)s^{-1}

The best value of velocity gradient that operate in flash mixing from 700 to 1000 s^{-1} with detention time of 120 sec (Peavy et al.,1985). The selection of an suitable critical velocity gradient in flocculation (gentle mix.) is more important than in rapid mixing (G-value) design (Vrale et al.1971).

2.8 The Requirements of Flocculation Process

Several researchers at the beginning of the last century demonstrated importance of the slow mixing process and the growth of flocculation, explained that the procedure of slow mixing of the added coagulant aims to obtain two things, the first is the spread of the coagulant in the water and its even distribution to ensure that it is mixed with the water in a homogeneous manner(Bachman ,1939). The second thing is to add energy to the Brownian diffusion energy through a mechanical mixing process that causes the formation of "Orthokinetic" chemical flocculation. Kawamura, 1976 indicate in practical experiment that the optimum value of velocity gradient (G-value) of $40s^{-1}$and a (Gt.-value) of $4.5X10^4$ usually leads in satisfactory alum flocculation. If G value Large with short times, it produce small size of flocs, while the value of G low with long times tend to produce larger size flocs, lighter flocs. Therefore large ,dense flocs can be easily removed in sedimentation basin ,it may be useful to vary the velocity gradient over the length of flocculation tank .(Peavy et al.,1985).

The small flocs that produced at high velocity gradient will become larger flocs at lower velocity gradient .

During the transfer of the flocculants through the flocculation basin, it grows significantly.

These particles require less energy to transport , and in the case of increased energy, they lead to the fragmentation of the large flocculants. When the flocs particles travel through the basin it will come grow large ,because the G values vary over the length of the flocculation basin (Peavy et al.,1985).

Wilson et al.,1983 indicated that the optimum value for velocity gradient (G-value) is $(30\text{-}70)s^{-1}$.

Orvichion et al.,1988 and Tebbutt, 1998 suggested value of velocity gradient (G) from 20 to 70 s^{-1} for good flocculation. Lower values of (G) cause inadequate flocculation while higher values will cause a shear the larger floc particles, and the normal detention time in mechanical low mixing tank between 20 to 30 min. The typical value of the product (G.t) is important within limits 5 to $10x10^4$ is often quoted - with mechanical low mixing.

2.9 Sedimentation Process

Clarification can be done by two main categories: those two categories used only to remove settable solids, the first one by plain sedimentation or after flocculation process, and the other which combine flocculation and sedimentation process into single unit. Conventional sedimentation basins and high rate sedimentation such as lamella plate settler, tube settler, plate settler and dissolved air flotation (DAF) fall in the first category. The other category involves solids contact units such as sludge blanket and clay recirculation clarifiers (Edward, 2005).

2.10 Theory of Sedimentation

The water treatment plant consists of a number of important units involved in the process of filtering and purifying water, but it does not match the sedimentation unit that removes up to 90% of suspended solids and that has an impact on work performance.(Smethurst ,1988). Sedimentation is a process of water remaining for adequate time mostly stable in order to make the flow velocity of water less than while settling velocity of the solid particles which they settles down by gravity. The efficiency of sedimentation based on the detention time.

The sedimentation can be made effective by the surface area of the sedimentation tank that makes the particle transport independent of others.

The depth of settling basin is one of the parameters that are included in sedimentation efficiency, it should be taking into account the accumulation of sludge and preventing the return of particles to flow.

2.11 The High Rate Sedimentation Process

In sedimentation tanks design, there are three controlling parameters: Settling velocity (Vs) of the particle that should remove , retention time (t_0), and quantity of water to be treated.

The settling characteristics of particles are classified into two main types (Mackenzie, 2010):

1. Discrete particle settling .
2. Flocculent settling.

In type 1: Particles that do not change in size, shape, and specific gravity over time are known as discrete particles (Peavy et al.,1985).

Discrete particles, settle separately at a constant rate of stability (such as sand and grains) (Dharmappa et al.,1994).

Also can be say that the sedimentation with low concentrations of particles that settle individually. In type 2: Particles has surface properties are such they coalesce ,or combine ,with the others at contact, this leads to changing the shape, size ,may be in specific gravity with each contact ,are referred to as flocculating particles (Peavy et al.,1985). In sedimentation basin design, the principal parameter affecting particle removal efficiency is the surface loading rate. The settling velocity is an important criteria in the design of settling tank ,it's called overflow rate or surface over flow rate. The basin geometry ,overflow rate ,removal system ,inlet and out let zone ,detention time, weir loading rates, and the sludge collection are the important considerations in sedimentation basin. Surface over flow rate is the ratio of discharge upon the surface area of the basin and is equivalent to the ratio of depth basin to detention time .(Syed R.Q et al.,2002).

2.12 Settling Operation in Circular Basin

In a circular tank, the water enters the center of the basin from the bottom to the top and is baffled by flowing radially towards the perimeter of the basin, and the horizontal velocity of the water will decrease continuously with increasing the distance from the center, and this makes the separated particle with a stable velocity (Vo) that is constantly subject to a change in its absolute velocity, Therefore, when the particles deposit to the bottom of the basin, they take a parabolic path line, while the sedimentation of the particles to the bottom of the rectangular trough is a straight path bottom of the rectangular (Peavy et al., 1985).

2.13Laminar-Flow Devices

The application of laminar-flow devices is one well - known modification of conventional sedimentation process that used in water treatment. Plate settlers or tubes settlers are of components of these devices, placed at 45˚to 60˚ with horizontal axis, and provides a greatly increased surface area for settlement when the cross sectional area of basin is limited (Fadel et al.,1990). Plate settlers or tubes settlers are used in an enhanced removal of solids because:

1. Laminar- flow is achieved through tubes settler (hence, almost ideal settling conditions are encountered).

2. Reducing the settling distance that particles moves to enter the sludge zone (hence the surface loading rate will be reduce in the basin).

3.Temperature current, density current, and wave action do not have any effects on sedimentation process as they do in a conventional basin, (Qasim,1999 and Degremont 1991).

Plate settlers are used for providing efficient settling, effectively reduces its surface loading, increase efficiency of solid removal. The water enters to the center of the basin from the bottom toward up and then removed. Because the flow velocity near the plates has zero value .

The solids that fall on the plates are not subject to (drag forces) therefore it can move in an opposite direction to flow of water. When plate settler have been used in a settling tank ,the relationship between efficiencies of particles removal E% depend on total added area of plates with their angle and flow rate i.e. E%=f (V_O, cosθ), (Al-Anbari,2005). The angle of inclination of tubes settler or plates settler is determined depending on the direction of the water flow relative to the direction of the sludge. There are three types of flow direction that can be renowned:

1- Counter-current

2- Co-counter current

3- Cross-flow as shown in Figure (2-4).

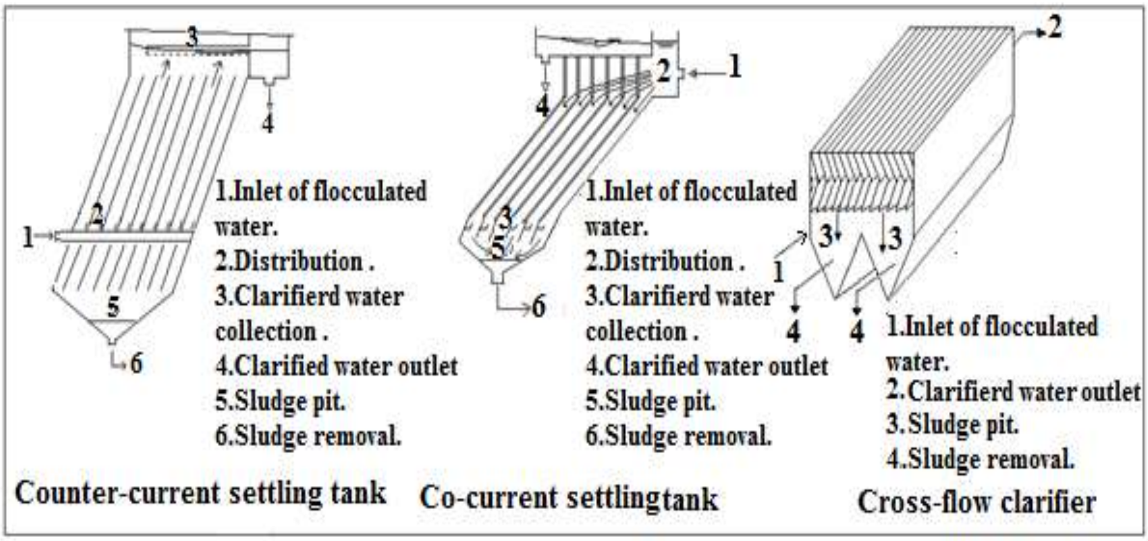


Figure (2-4): Inclined plates settler at different direction of flow. After (Richard &Capon 1980).

Inclined plates used to increase the hydraulic capacity and enhance the quality for existing conventional settling tanks. In treatment plant application, tube settlers increased the hydraulic capacity by 40% with removed turbidity less than 30 NTU . (Hassan & Hassan 2011).

2.14 Some Critical Parameters

A-Area of a plate settler

The number of plate settler (N) cover a horizontal area (Ah) can be determine by equation :

$$N= \frac{Ah\ sin\theta}{Ls(D+e_p)} \quad \text{---(2-2)}$$

Where :-

D, Ls =Distance between two parallel plate, length of the plate respectively.

e_p, Ah =thickness of the plate , horizontal area respectively.

The total area (AT) is equal to:

$$AT= (\frac{Q}{VSin\ \theta}) \ (1+\frac{e_p}{D}) \quad \text{---(2-3)}$$

Where : $V= \frac{Q}{(Ls*D*N)}$, Q = Capacity of the settler (Arboleda,1986).

B-Angle of inclination(θ)

Selection of plate inclination angle with the horizontal (θ) ,effects on the design of the settler .If the value of angle is large, the smaller the area of plate. Yao, mentioned in his model (yao,1970,1973), that the value of (θ_{opt}) lie between (30 to75). For enhance the quality for existing conventional settling tanks and increase the hydraulic capacity , plate settlers or tubes settlers are placed at 45 ° to 60 ° with horizontal axis ,and provides a greatly increased surface area for settlement when the cross sectional area of basin is limited. (Fadel et al.,1990). Inclined plate settlers are similar to the tube settlers except that 45° to 60° inclined plates are used instead of tubes, (Joseph A Salvato et al., 2003). Many of literature studies for settling efficiency of plate settler show that incline angle (θ) must be within range (40-60) (Culp et al.,1968; Schade et aL,1984).

C-Plate spacing (D)

The compactness of the plate settler is Severely affected by the plate spacing ,reducing its value by 50% will increase the projected area twice that can be erected in a given tank. Huisman,1986 ,stated in his study that, when use the plate settler in sedimentation tank ,the clear distance between two plates is not less than 4cm. For the same filtering efficiency , the cost of the settling chamber with plate settler will be about 20-30 % less than the other.

Therefore it should be take considered both of economical and hydraulic effects. The spacing between two parallel plates is dependent on the value ratio between the amount of sludge and water inside "Lamella cell" for treatment plant of water.(Grimes et al.,1978).

D-Equivalent surface load

Each of (Yao,1970) ,(Hazen,1904) and (Camp,1946) suggested that can be apply the concepts critical particle on tray settlers to allow application of the over flow rate. In the horizontal flow settling tank ,the case of settling of discrete particle ,the main dimension is the horizontal surface area of the basin (A) while the depth of the tank plays no role in the settling rate calculation ,where the particles are removed beyond it.(Verhoff,1977) and (Degremont,1991). The efficiency of particle removal depends on the settling over flow rate (Vo).

$Vo=SOR=Q/A$ ($m^3/m^2/hr$) -------------------------------------- (2-4)

Therefore in the design of settling tank ,it can be increase a settling area (A) by using many of plates settler, or introduction of tube settlers ,or reducing depth of basin.

E-Hydraulic Condition

The flow pattern in the sedimentation tank approximately always turbulent , After a while, reduction in sedimentation occurs due to the lateral movements of settling particles. flow conditions can be determined through Reynolds' number(Re).

$$Re = \frac{V_{h*R}}{\nu} \text{ --(2-5)}$$

Where:-

R ,ν = The hydraulic radius, the kinematic viscosity respectively .

If Re <600, the flow is laminar , if Re =2000,flow is turbulent .

For rectangular tanks $Re = \frac{Q}{\nu(B+2H)}$ -----------------------(2-6)

For circular tanks $Re = \frac{Q}{2\pi r\nu}$ ------------------------------------(2-7)

Thus, in case to reduce Re in rectangular tanks, it should be increase the width and/or depth. In circular tank ,because of the flow pattern is fixed, Re decreases with radial distance from the center of basin (Casey, 1997). In order to ensure the stability of laminar flow condition, it should be keeping Reynolds number (Re) and Frouds (Fr) number within their proper.

Fischerstrom,1955 reported that laminar, stable flow may be occur in settling tanks at Re<500,and $Fr>10^{-5}$.

According to (Horvath,1984) stable flow is probably occur in a settling tank when the value of Re <500,and Fr >10^{-5} However, it has been suggested that Re =150 be considered a lower limit:

$$Re = \frac{v\,R}{\nu} \text{ --(2-8)}$$

$$Fr = \frac{v^2}{gR} \text{ --(2-9)}$$

These expression can be rewritten when the dimensions are given as.(Szalay,1960).

$$R= \frac{Re\,\nu}{v} = \frac{v^2}{Fr\,g} \text{ -- (2-10)}$$

With reference to the above two equations (2-9) & (2-10) , it can be seen that the Fr number is directly proportional to the velocity and inversely to the hydraulic radius. However, higher flow velocities also have negative consequences, as the Re number increases and so the turbulence becomes more intense as a result. The only solution is to reduce the hydraulic radius. In this way the Fr number is increased, and the Re number reduced at the same time, both of which have beneficial effects. In order to increase the value of the hydraulic radius at a given cross- sectional area, the length of the wetted perimeter must be increased. Thus the hydraulic problem is solved by dividing the flow area with the help of baffles and membranes.

F-Sludge Removal

The angle of plate settler should be as steep as possible in order to obtained continuous gravity drainage, and the sludge on the surface of plate, therefore can be easily remove, and it make continuous self-cleaning. .(Forsell et al.,1975).

There is an advantage in circular basin ,where the simplest sludge removal mechanism is required and requires less maintenance.(walker,1978).

2.15 The Filtration Process

Filtration is defined as: The processes of physical -chemical for separating suspended and colloidal particles from water across a bed of granular materials. Figure (2-5) shows the typical gravity flow filter.

The rapid sand filtration was used for the treatment plant in Kerbala so as to contain many good qualities compared to slow sand filtration and pressure filtration .

Therefore, the same rapid sand filtration was used in the experimental study to simulate the same processing stages at the station.

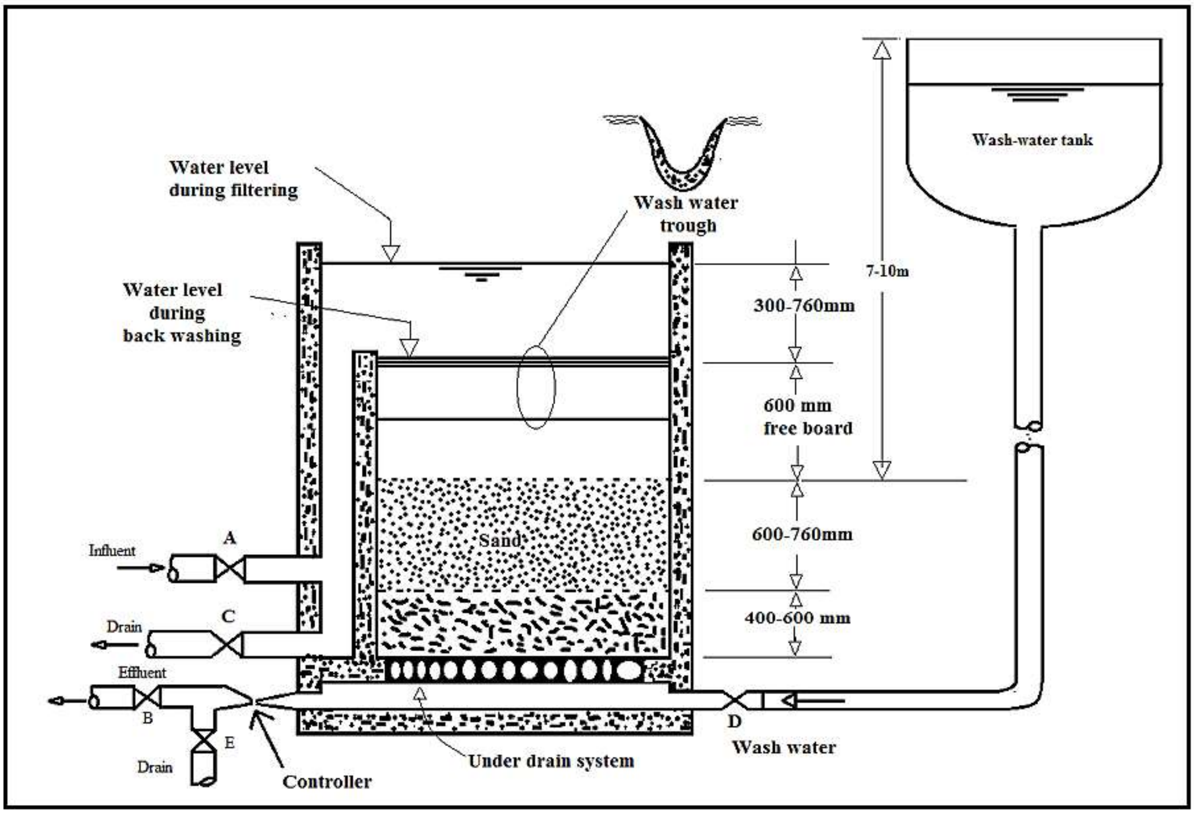
Correctly used high rate filtration can save large sums in building new filtering stations, in addition to the possibility of using it to increase the flow when developing existing stations in a little cost instead of establishing new ones.

The water turbidity level should be reduced to avoid turbidity levels from interfering with subsequent disinfection operations.

The US Environmental Protection Agency (EPA) indicates that the level of turbidity in the treated water is 0.3 NTU at 95% of the monthly average, provided that the value does not exceed 1 NTU.

Granular filtration is the most common filtration process in which suspended impurities or colloidal particles are separated from water via a porous medium.

There are many types of spread media such as sand, charcoal, activated carbon, and garnet. Much and extensive research has been makes during the past sixty years including various details of filtration and the higher filtration rates (7.3-9.8) m / hour. Most of these studies were conducted to achieve the EPA ,drinking water standard for turbidity (1NTU as a monthly average) (AL-Anbari,1997).



Figure(2-5):Typical gravity flow filter operation used in kerbala (WTP).

2.16 Dual - Media Filters

Rapid sand filters contain high quality sand on top, so smaller pores are also on the top, so the top layer of the filter will repel most particles. In order to exploit the depth of the filter in the filtration and not to recede on the surface of the sand layer, it is necessary to adjust the arrangement of the media inside the filter, by placing the large particles on the small ones, and this is done by placing a layer of anthracite (coarse charcoal) over the fine sand layer to form a dual media filter.

The specific weight of the coal is less than the specific weight of the sand. The coal settles more slowly than the sand when backwashing is used and thus the coal will settle at the top. Some dual media filters operate at loading rates of up to (20m /hr).

Many pathogenic organisms are removed from the water by the filtration process, but they cannot be relied upon to provide complete protection of health.(Peavy et al.,1985).

Several studies were presented by(Cleasby,1981a)and (Mohammed,1989) ,which showed the differences in head losses between the dual media filter and sand filter, where the head loss in dual media filter is lower than the head loss in sand filter.

The size of anthracite in a typical dual media filters is twice that of sand (for example, 1 mm anthracite over 0.5 mm sand), the head loss evolution rate of a dual media filter should be about one half the rate of the 0.5 mm sand filter when both filters are operating at the same filtration rate.

The feature of dual and mixed media filters that allow direct filtering of low turbidity water without passing into the sedimentation basin (Peavy et al.,1985).

2.17 The Filtration With Activated Carbon Process.

One of the oldest materials for water and wastewater treatment is activated carbon due to its adsorption property and therefore it has been widely used in removing organic and inorganic pollutants.

The adsorption property of activated carbon depends mainly on the surface chemistry and the porous structure of the porous carbon (Amit Bhatnagar et al.,2012).

Activated Carbone is made from various of carbonaceous rich materials like coal, wood ,lignite and coconut shell(Hassler ,J.W,1980). One of the most widely used adsorbent materials in water treatment is GAC (granular activated Carbone).(Bhatnagar et al., 2013).

Activated carbon has become the most important odor-removing material available. It is characterized by high porosity and is composed of many free valence carbon atoms.

The surface in contact with the solution will attract the particles inside the solution and may retain them through the forces of chemical bonding or van der Waals attraction forces that hold the particles to the surface, and thus the adsorption will remove the solids, liquids and gases from the solution at a reaction rate and complete removal depending on the temperature, molecular size, pH and molecular weight.

In GAC applications it can be specified the grain size distribution in the same manner of application of filter sands ,using uniformity coefficient (d_{60}/d_{10}) and effective size (d_{10}) parameters .

The range of the effective GAC grain size is between (0.6-1.2)mm and the uniformity coefficient must be not more than 2.1 (AWWA,1974). The total rate of adsorption is dependent on size of particles - it varies reciprocally with the square of the particle diameter, when the concentration of solute increases , total rate of adsorption increases, when the temperature increase ,the total rate of adsorption decreases. when the molecular weight decreasing of solute ,the total rate of adsorption decreases (Eckenfelder,1966). Likewise, the rate of adsorption is affected by the pH, whereas the pH increases, the rate of adsorption decreases and is very weak when the pH is increased and exceeds 9.0 (Culp and Culp, 1971).

Using the type of granular activated carbon in fixed bed is preferred to its use as a powdered form because the continuous application is needed. GAC should be replaced typically after three months to one year of operation. GAC columns are designed to operate in a conventional up flow filtration or in an down flow mode. The detention time in GAC columns is generally in the range 5-20 min . In up flow columns which operated according on the countercurrent principle ,the most economic use of granular carbon can be made (Culp and Culp, 1971).

Even though single filter media is achieved a higher efficiency removal of turbidity, all units with activated carbon gave effective removal that obey with the limit of 0.3 NTU advise by the EPA and WHO to minimize microbiological risk .(Dyna ,2018).

Activated carbon has adsorptive properties, highly porosity and large surface area that allows it to remove and retain many of particles that present in water.(Steel,1984).

In order to eliminate or inactivate pathogenic microorganisms, the turbidity of water coming out of the filter should be in the range of 0.1 to 0.5 NTU and as recommended by several authors to prevent minimal drinking water hazards ,(WHO,2006),(EPA,2000),(AWWA : McGraw Hill,2011).

Typical filtration systems do not have the ability to effectively remove turbidity and dissolved organic matter, so it is necessary to find other treatments and evaluate them to produce improved and high quality water. (WHO,2006).

One of the main parameters in assessing filtration is turbidity, being quick and simple ,turbidity is associated with molecules in the water ,and these molecules in turn are linked to the presence of bacteria, protozoa and viruses .(EPA,2009) .

As for the aesthetic effects on water quality, they are related to organic matter, which produces secondary disinfection byproducts that have carcinogenic effects/ or with pesticides, agricultural or pharmaceutical preparations, which are synthetic organic compounds that cannot be removed with conventional drinking water treatments easily.(Crittenen et.al.,2012).

Dyna,2018 in her study of (Evaluation of turbidity and dissolved organic matter removal through double filtration technology with activated carbon) found that the organic matter removal efficiencies of the formations using GAC were most effective, this confirmed that the use of GAC is suitable as a filter medium to reduce odor and flavor in addition to the absorption of organic compounds.

2.18 Bentonite as a coagulant material.

Aluminum sulphate is widely used in coagulation process in water treatment . However ,(ALakaparampil J.,2020) illustrated that intake of alum in large quantity may effect on human health causing Alzheimer's disease . in order to minimize the effects associated of alum dosage ,it suggested to mix ratio of bentonite - alum due to removal highest of COD of 93.09% was at ratio of 50:50 by volume with best pH of eight . At initial stage of bentonite using ,the removal of turbidity decrease but started to increase when its reached 50% of bentonite dosage.(Abdullah, R., Abustan et. al., 2013).

Bentonite is widely used as a suspending, stabilizing and binding agent, and as an adsorbent or clarifying agent in many applications. Some times bentonite and/or kaolin are added to water especially when the water has low turbidity and to be flocculated for effective flocculation.,(Schutte, 2007).

Bentonite is added to raw water, especially with low turbidity, it will increase the weight of the suspension and increase the density of the particles in addition to providing a large surface for the absorption of organic compounds. The dose of bentonite clay ranges from 10 - 50 mg / liter. (Cohen and Hannah,1971).

When adding bentonite dosage of up to 0.8 g/L to raw water ,the turbidity increased gradually . Also visual testing showed ,when bentonite increased,the supernatant became more clearness. The mechanism of bentonite adding ,will minimize the electrostatic forces and formed more flocs, thus will decrease turbidity . In addition ,the dosage of bentonite must be increased more than 1.2 g /L(Rohana Abdullah et al., 2013).

2.19 Physical and Chemical Tests Carried Out on the Water

Water has many physical properties that are determined by physical parameters related to the senses of sight, touch, taste, and smell. Turbidity, color, temperature, taste, odor, and suspended solids are within the physical parameters of water (Peavy et al.,1985).The drinking water has a special importance imposed by human need, so the water must be free from chemicals and microorganisms because they pose risks to human health, and it must be free from cloudiness, color, odor and unacceptable taste. The multiplicity and diversity of water sources in the country has an impact on the quality of the water supplied in each region. Therefore, specifications have been set to determine the permissible percentages of this substances in addition to methods for examining and analyzing this water to determine these materials and their conformity with the specified specifications.

As for the chemical analyzes includes: Total hardness, calcium, magnesium, chlorides, sulfates, iron, alkalinity, acidity, and total solids.

2.20 Physicochemical Parameters of Drinking Water in (WTP)

Table (2-2) describes the physical and chemical properties of water for each variable based on the Iraqi standard specification for drinking water (IQS417,2001) as it may affect the consumer's acceptance of its use for drinking, whether these effects are natural or otherwise.

Table (2-2): Physical and chemical properties of water based on (IQS417, 2001).

Properties	Maximum permissible limit	Examination method
Turbidity	5 NTU	Turbid meter
Chloride	250mg/L	Argentometric method
Total hardness	500mg/L as CaCO_3	EDTA Titrimetric
Iron	0.3mg/L	Phenanthroline method
TDS	1000 mg/L	Calculation by analysis and summation
sulfates (So_4)	250 mg/L	Gravimetric method
Calcium (Ca)	75 mg/L	EDTA Titrimetric
Magnesium (Mg)	50 mg/L	Calculation
PH	8.5	PH meter
Electrical Conductivity (E.C)	2000($\mu S/cm$)	conductivity meter
Al Alkalinity	125 mg/L	EDTA Titrimetric
Temperature	-	Mercury thermometer
Color	-	Spectrophotometric
Taste and Odor	-	Senses of taste and smell

2.21 Hydraulic Model of the Water Treatment Units

Because a water treatment plant is a train or chain-like series of treatment units, success of the entire process depends on each unit performing satisfactory.

Relatively simple treatment units are as important as the more advanced treatment systems.

Thompson ,1969 in his doctoral thesis published in 1967,described a scale up method for rectangular settling basins containing many novel features.

By dimensional analysis, the removal efficiency of settling basins can be described in the general case with the help of the following dimensionless groups:

$$\frac{Ce}{Co} = f_1[\frac{Co}{\rho}, .\frac{\rho.v.l}{\mu}, \frac{Q}{W.L.B}, \frac{v^2}{g.D}, \frac{B}{L}, \frac{D}{L}, \frac{H}{L}] = f_2[\frac{Co}{\rho}, \frac{\rho.Q}{\mu D}, \frac{Q}{W.L.B}, \frac{Q^2}{g.D^5}, \frac{B}{L}, \frac{D}{L}, \frac{H}{L}] \text{ ----(2-11)}$$

where:

Co : Concentration of suspended solids in the inflow .

Ce : Concentration of effluent from the basin.

L ,W, B : The length of the settling basin, width of the settling basin, and depth of the settling basin, respectively.

H:The differential elevation statistical parameter characterizing the distribution of the settling velocity (v).

μ: The viscosity of fluid and ρ is the fluid density .

Below dimensionless groups are involved in the above expression:

-Reynolds number: $Re = \frac{V.L}{\mu/\rho} \propto \frac{\rho.Q}{\mu.D}$ --------------------------------------(2-12)

-Froude number: $Fr = \frac{V^2}{g.D} \propto \frac{Q^2}{g.D^5}$ --(2-13)

Both ,(Hart and Gupta,1978) indicated that in the case of neglecting the effect of viscosity, a number dimension less is derived that equals ,or is a form of Froude's number $[\frac{V^2}{g.D}]$.

Thompson,1969 studied two geometrically similar models (denoted A and B) were built to different scales ,he found the Froude number of both models is the same, this means ,that it is approaching to dynamic similarity.

In the same manner , when gravity effects is neglected, it produce a Reynolds number [(V.L)/(μ/ρ)],this number is governs the design according to dynamic similitude . The flow of an open channel or free surface in the tank is affected more by gravity than by viscosity. Therefore, the Froude number is usually taken into consideration when designing the model.

2.22 Water Quality Index.

Water sources are currently suffering from an increase in pollution and neglect for several reasons, including the development of lifestyle, industrial development, increase in the population, depletion of water due to unfair consumption of water, thermal recession that led to less rain, throwing waste of industrial, waste of electric power stations into the water source, bad drainage of wastewater In rivers without treatment, filtering and leaking of irrigation water contaminated with fertilizers and agricultural products to flow into rivers, in addition to the lack of good planning in water management. (Alobaidy.A.H. , 2010).

The determination of water quality in traditional ways, which depends on the comparison of experimental values with current standards, is not easy to assess the quality of water for a large sample containing groups of many parameters.

Water Quality Index (WQI) can be considered a main be a major component of water resources and can be used to simplify the complex parameters of water quality variable.(Salam et al.,2020).

The water quality index is usually expressed as a number of dimensions collects many physical, chemical and biological variables in number one.

The term water quality has been developed to give a reference to the appropriateness of water for human consumption and is widely used in many scientific publications on sustainable management.

A water quality index (WQI) can be defined as a numerical term used to convert several variable data into a single term describing water.

This number can be deduced from the physical and chemical parameters of water (A.Sargaonkar et al.,2003), and gives an indication of whether or not the water is suitable for human consumption (H. J.Vaux,2001) .

By comparing physical and chemical properties for a sample of water ,it can be determined the water quality based on water quality standard . Water quality standards have been established to enable the provision of clear and safe water for people consumption .These are usually based on acceptable levels that have been scientifically evaluated from toxicity, both for humans or water organisms. (Zahraa et al., 2012).

The benefits envisaged from calculating the quality index are as follows:

1. Choosing a correct and appropriate treatment method, as there is water that may not need treatment by calculating the quality index of that water , and that it only needs purification , or it may need the traditional water treatment method , or the water may be polluted and severely poor and requires the use of advanced water treatment technique and this is known as (reverse osmosis, adsorption, and other methods used to purify polluted water.

2.Finding a comparison process between more than one source of raw water, to come up with a better decision in order to direct its use.

3.Diagnosing the health status of water bodies by knowing the amount of water quality indicator.Using of the index in evaluating water quality has been recently innovated.

The calculation of water quality based on a number of physical–chemical and bacteriological parameters by comparison according to the standards (

Abbasi, 2014). There are several methods for calculating the water quality index like:

- Horton,1965 was the first who found an index to describe water quality, he used the method of computing water quality index called weighted arithmetic index technic (Tyagi, et al., 2013).
- Brown et al.,1970 improved Horton index ,his work supported by the national sanitation foundation (NSF) .(Brown, 1972).
- Steinhart et al.,1982 used the ecosystem environmental quality index for Great Lakes .
- In 1995 the Canadian Water Quality Index was proposed by the Canadian council of ministers of the environment (CCME), which is based on harmonic square sum. This indicator has been used in many countries because it is based on harmonic square sum, and thus its results are more realistic if compared with other methods.(Vindo, et al., 2013).
- Subsequently, Bhargava used the first WQI in India, setting a water quality range from 0-100 (Sutadian, and Gitau ,2016) .
- US National Sanitation Foundation Water Quality Index (NSFWQI), Heavy metals pollution index (HMPI),Oregon Water Quality Index (OWQI), and the British Columbia Water Quality Index (BCWQI) are frequently used.

To evaluate the Kerbala water purification plant project in the Imam Aun area , the water quality scale index was calculated in this study by using two methods:

A . The Weighted Arithmetic Index.

B. Canadian Cabinet Council of Ministers of the Environment Water Quality Index (CCME WQI).

2.22.1 Weighted Arithmetic Method

One of the methods in which the water quality index can be calculated , in the water sample, a total number of physical and chemical parameters are selected for the purpose of calculating water quality index according to (WHO) World Health Organization .

There are three equations playing a very important role in determining the index , which mentioned in chapter three , section (3-10) , (Application of the Water quality index) when calculating WQI using the Weighted Arithmetic method . (Brown et al, 1972).

In this study, Weighted Arithmetic Index was selected in the water quality index account.

2.22.2 The CCME WQI

The other method is the method of Canadian Council of Ministers of the Environment (CCME) Water Quality Index (WQI) is accepted method and commonly applicable model for evaluating the water quality index (SHARMA D., 2011).

Many WQI deals with providing and summarizing the data that describe water quality index in order to be accessible to the public ,there are many method to calculate the water quality index including Canadian Council of the environment (CCME),which are used to make environmental life safe.

For the Canadian index calculation , there are three essential scales (scope, frequency, and amplitude).

The value of CCME WQI ranged between (0-100) ,where the number 100 indicates the best result for the index, while the number 0 indicate the poorest indicator. Within this range ,water quality have been classified in to five categories as poor .marginal ,fair ,good and excellent.(Inass Al-Mallah et al.,2017).

CHAPTER THREE

MATERIALS AND EXPERIMENTAL WORKS

CHAPTER THREE

MATERIALS AND EXPERIMENTAL WORKS

3.1 The Area of Study

Intakes of Karbala water treatment plant structure are constructed adjacent to AL-Euphrates river in Al-Musayyib City that far about 18Km west from water treatment plant for the withdrawing waters purpose.

The plant is located in the city of Imam Aun, about 12 km East away from Karbala Governorate, and about 18 km from AL Musayyib city ,where the course of the Euphrates River. The plant has a designed capacity of 10,500 m³/hr.

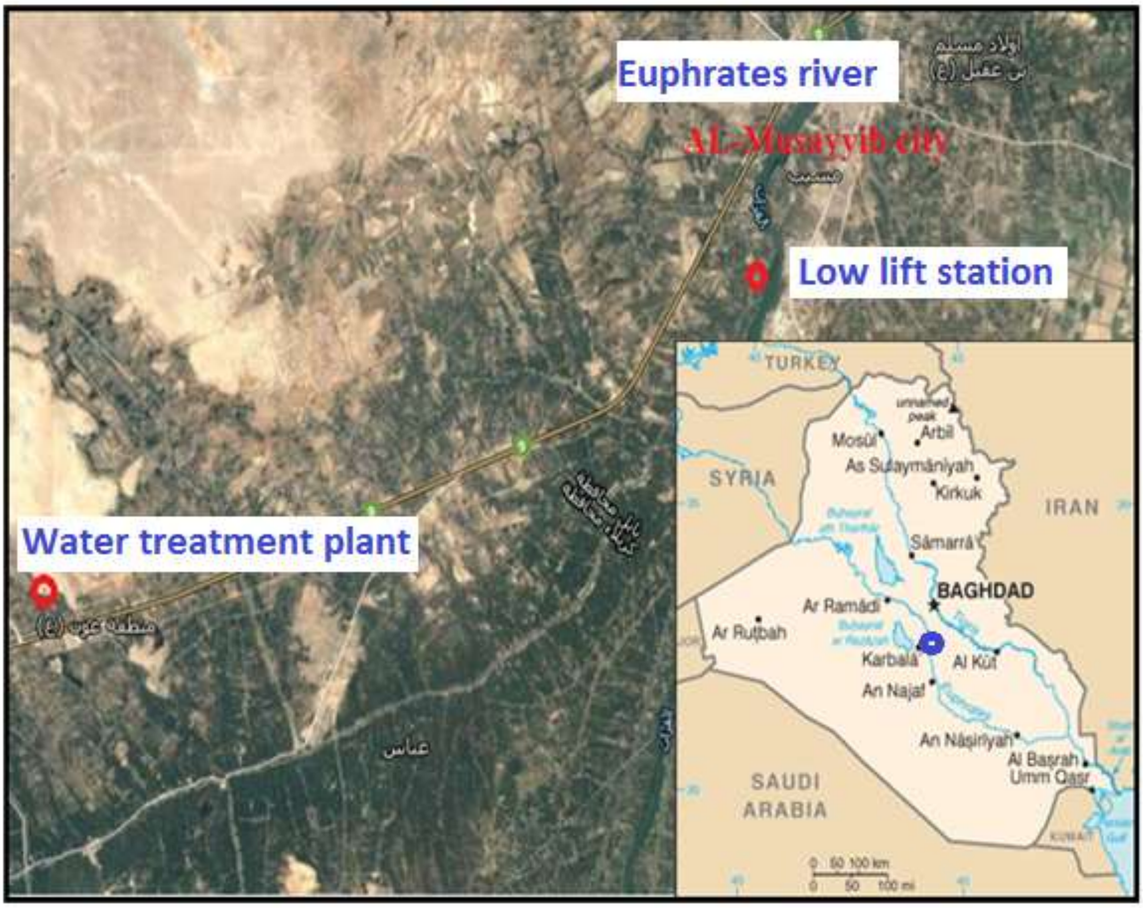
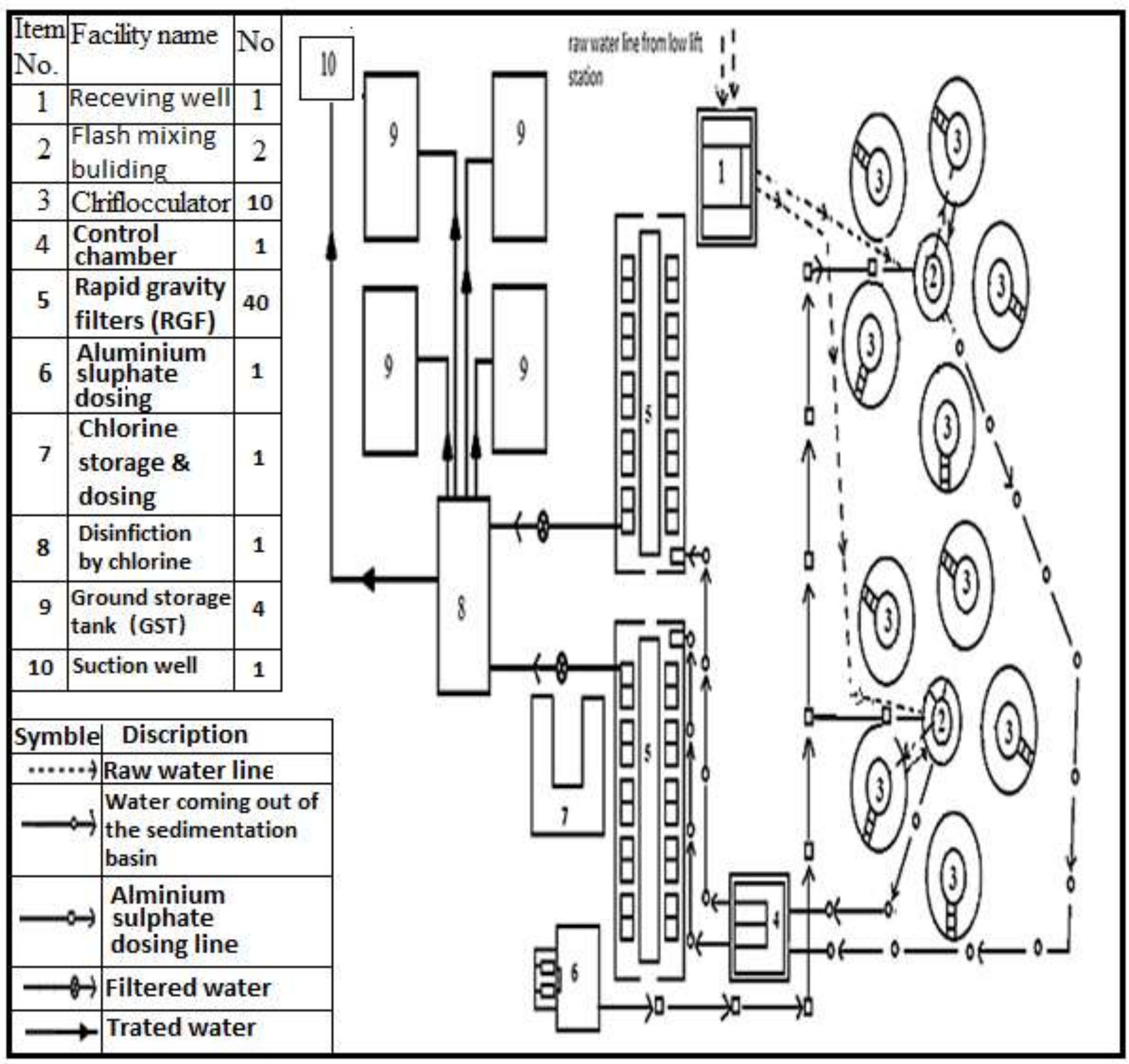


Plate (3-1): location of the Kerbala water treatment plant and low lift station.

3.2 Description of the Sequence Stages for Kerbala Water Treatment Plant.

3.2.1 Description of Water Treatment Plant.

The stages of water treatment for the Kerbala water treatment plant, which consists of 10 main technique components, as shown if figure (3-1).



Figure(3-1): The site plan of Kerbala water treatment plant.

The main 10 technique component are:

1- Low lift station and raw water intakes.

2- Raw water reservoir (Receiving well).

3- Flash mixing tank .
4- Clariflocculator (flocculation and sedimentation).
5- Control chamber .
6- Rapid gravity filters (RGF).
7- Sludge pit and site drainage.
8- Aluminum sulphate dosing.
9- Chlorine storage and dosing .
10- Back wash water tank .

3.2.1.1 Low Lift Station and Raw Water Intakes

The low pumping station is located on the Euphrates River in Al-Musayyib, Babil Governorate, from which raw water is pumped to the water treatment facility in Imam Aun, Kerbala Governorate, around 18 kilometers west of Al-Musayyib.

3.2.1.2 Raw Water Reservoir (Receiving Well)

The receiving well is a concrete structure that measures 13.7 x 10.7 meters and stands at a height of 9.2 meters. It is the point of entry for all water entering the Kerbala water treatment facility.

3.2.1.3 Flash Mixing Tank

This is one inlet structure for each of two treatment lines, each of which is made up by five clariflocculators. The circular intake construction includes a flash mixing chamber, a coagulated water distribution chamber, a sludge collecting launder, and a clarified water collector.

3.2.1.4 Clarified Water Collector

The lower portion of the annular collector receives return water from five clarity units, which is then transported through the DN 700 ductile iron pipes that arrive from the bottom of the collector. A 1400 DN ductile iron pipe placed upstream of the filter building collects the cleared water flows and transports them to the control room.

3.2.1.5 Sludge Collection Launder

Above the clarified water collector is a room called the sludge launder, which receives the clarified sludge returned from the five purification units forming a unified treatment line. Below this chamber is a v-shaped chamber called the sludge launder. The sludge is transported to the launder through five ductile iron pipes. Another function of sludge launder is to receive excess water in the event of an overflow from the distribution chamber .The DN 700 ductile iron pipe will transport the collected bleeder sludge out of the launder ,leading to sludge pit ,then the sludge pumping station.

3.2.1.6 Clariflocculator(Flocculation and Sedimentation)

Clarification tank consists of two concentric basins, the inner basin is used for flocculator , while the outer basin surrounding the flocculator basin is used as a sedimentation basin, the function of the Clarifloccultor is to remove suspended matter from the water. Total No. of clariflocculators : 10.

There are two pipelines that connect the rapid mixing tank with the Clarifloccultor, each lone contain of five Clarifloccultor to one tank of rapid mixing. The clariflocculator contains of the following main components :scraper blades, steel bridge , electrical peripheral drive with overload switch, handrails , access ladders and V-notch weirs are adjustable along the perimeter of the tank.

3.2.2 Main Component and Instrumentation (Clariflocculator)

3.2.2.1 Clarifier Tanks.

Table (3-1) shows the features and dimensions of the clarifier tank .

Table (3-1): Details and dimension of clarifier tank

Clarifier Tank Details	Dimension
Diameter of tank (inside)	38m
Flocculation zone Diameter (outside)	14m
Ground slope	1:10
Min. effective depth of water at perimeter	4.19 m

3.2.2.2 Flocculation Zone

In each flocculator tank and at the top of the flocculation area, four vertical agitators, which rotate slowly (paddle mixers), are distributed , with equal dimensions inside the flocculation zone, around the inlet well, where the coagulated raw water from the rapid mixer tank enters the clarifier tank.

In the center of the flocculation zone, the coagulated water enters the top and exits through four equal-dimensional openings that direct the water towards the edges of the paddle. In turn, the paddle turns the water slowly in order to promote formation of macroscopic flocs.

3.2.2.3 Control Chamber

The control chamber is a concrete tank located downstream of the clariflocculators and upstream of the filter building, and its function is to distribute the clarified water flow between filter gallery I and filter gallery II , and through two inlet tubes (D N 1400,Ductile iron).

3.2.2.4 Rapid Gravity Filters (RGF)

The filtration building consists of two (2 NO.) filter galleries. Each gallery are sub-divided into two parallel lines as shown in Figure (3-2). The location of filtration building at downstream of the control chamber where the flow is evenly distributed to all filters .

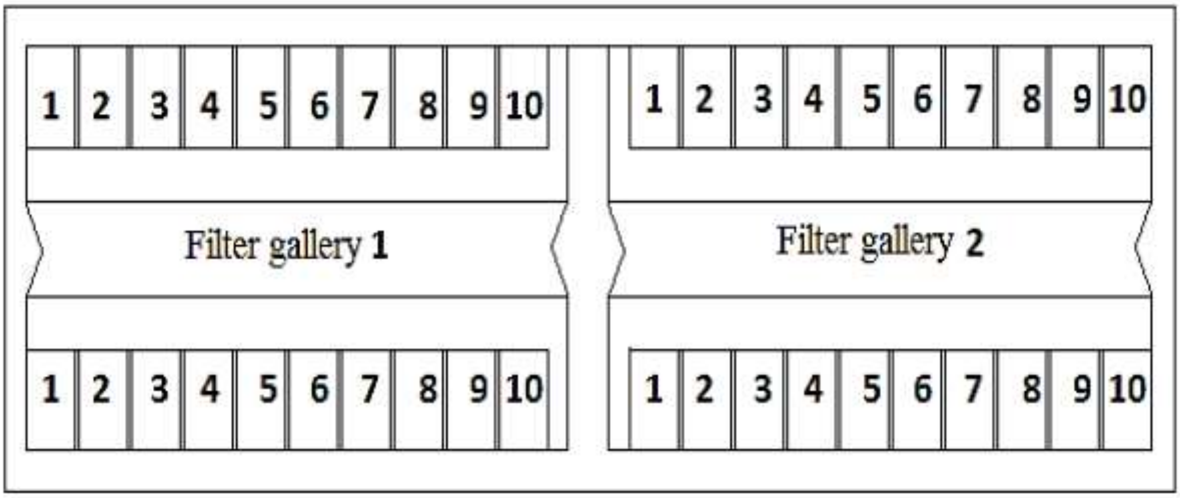


Figure (3-2): Filtration building plan in kerbala water treatment plant .

The type of filters is gravity rapid sand . The configuration is as follows:

Total NO. of Filter’s Cells: 40 cells , dimension of each cell (inside) is (9*5.6) m

Area of each Filter’s Cell (inside) : $50m^2$

Resulting normal filtration velocity : 5m/h

3.2.3 Main Component and Instrumentation of Rapid Filter

3.2.3.1 Clarified Water Distribution

After the flow coming from the clarifiers is divided evenly in the upstream control chamber , the clarified water is conveyed from the control chamber to the filter gallery via a tube (DN 1500) in the separate inlet ducts of each filter gallery.

3.2.4 Filter Media

High quality silica sand is used (clean and washed with acid) the main properties are listed below:

1. Media type : silica sand
2. size of sand: 0.6 to 0.65 mm
3. Effective Uniformity coefficient : ≤1.5
4. Bed depth : 700mm.

3.2.5 Supported by Gravel Layers

The bed consists of five layers of round gravel, each layer has a specific gradient and thickness, the larger gravel is placed at the bottom of the bed, and then the gradient less than the first is placed on the lower layer and so on.

Table (3-2) shows grading and depth of gravel layer**.**

Table (3-2): Size of gravel and depth of each layer in kerbala water treatment plant .

Media Type:Rounded Gravel		
layer Number	**Grading (mm)**	**Depth of layer(mm)**
Layer1(Top layer)	**2.5-6.5**	**150**
Layer2	**6.5-9.5**	**150**
Layer3	**9.5-13**	**100**
Layer4	**13-38**	**100**
Layer5 (Bottom layer)	**38-50**	**100**
Total Depth of Support Gravel		**600**

3.2.6 Aluminum Sulphate Dosing.

Dosing building consists of a storage room for alum as well as three alum tanks for stirring it. The alum tanks are used to prepare alum solution and as feeding tanks for the dosing station.

3.3 General Equations of Hydraulic Scaling of the Pilot Plant(Model)

The indication of the force of gravity and the force of inertia control the fluid motion by means of Froude's number which is the dimensionless parameter (William ,1957). The general expression for this this parameter is:

Fr = $\frac{V^2}{g.l}$ (for rectangular tank) = $\frac{Q^2}{4\pi^2 r^2 H^3 g}$ (for circular rank)(Casey,1997).

where:

V= fluid velocity, m./sec.

L = linear dimension; for example, the equivalent diameter, m.

g = gravitational constant, m./s^2.

Q = flow rate m^3/sec..

H=depth of basin ,m.

r= radius of basin ,m

Normally, a dynamic similitude between the pilot plant and prototype operation is considered to be the criterion for designing a successful pilot plant ,Froude number should be the same for both models and prototype systems.

For circular ,radial-surface over flow rate :

$(Fr)_{Prototype} = (Fr)_{model}$.

Also,

$$(\mathrm{Fr})_p = \frac{Q^2}{4\pi^2 r^2 H^3 g} = (\mathrm{Fr})_m = \frac{Q^2}{4\pi^2 r^2 H^3 g}$$

$$\left(\frac{Q^2}{r^2 H^3}\right)_P = \left(\frac{Q^2}{r^2 H^3}\right)_m$$

$$\frac{Q^2{}_m}{Q^2{}_P} = \frac{(r^2 H^3)_m}{(r^2 H^3)_P}$$

$\lambda Q^2 = \lambda r^2 * \lambda H^3$

$\lambda Q = \lambda r * \lambda H^{3/2}$ --(3-1)

For circular ,radial-surface over flow rate :

$V_s = \frac{Q}{\pi(R^2{}_{out.} - R^2{}_{in})}$ --------------------------- -------------------------------------(3-2)

For circular ,radial-flow settling basin :

$V_r = \frac{Q}{2\pi * r * h}$ ---(3-3)

Where:

V_{sp} , V_{sm} are the settling velocity of prototype units, and settling velocity of pilot plant units, respectively .

λV_S= Scaling factor of settling velocity (S.O.R).

R_P, R_m are the radius of tank of prototype of units, and radius of tank of pilot plant of units, respectively.

λ_R = Scaling factor of radius of tank.

A_p, A_m are the area in prototype, and area in pilot plant, respectively.

λ_A= Scaling factor of area.

h_p, h_m are the depth of tank in the prototype of units, and depth of tank in the pilot plant of units, respectively.

λ_h= Scaling factor of depth.

Q_p, Q_m are the flow rate in the prototype, and flow rate in the pilot plant, respectively.

λ_Q = Scaling factor of flow rate.

t_p, t_m are the detention time in the prototype, and detention time in the pilot plant ,respectively.

λ_t= Scaling factor of detention time .

$(V_r)_p, (V_r)_m$are the rotational velocity in the prototype, and rotational velocity in the pilot plant of units, respectively .

λV_r = Scaling factor of rotational velocity .

For the purpose of obtaining correct results and applying the improvements and modifications made to the original units, a pilot plant for drinking water treatment plant units was built in the same form as the original units for the drinking water treatment plant in the holy Kerbala, which leads to improving and raising the efficiency of the treatment process.

Villemonte et al.,1969 made important contributions in similarity problems in circular settling basin. They used similar model proportional to the scale ratio =4 ,in order to studying the scale - up criteria by flow -through experiment done by dye tracing.

The experiment involving various surface loads and the ratio of basin depth to diameter was evaluated for typical points on flow through a hydrograph plotted in a dimensionless coordinate system.

In modeling hydraulic conditions and since previous sources had assigned equally important roles to the Fr and Re criteria, (Villemonte and Rohlich,1969) investigated both experimentally at specific Ts =Q/F surface load ranges . The scale factor of Ts are (λ =4).

Where:

Ts : value of the surface load.

F: Surface area of tank .

After analyzing the experiment results, the following conclusions regarding the hydraulic similarity of radial flow sedimentation tanks were made.

(a) Conversions based on Reynolds number criteria was unsatisfactory approved .

(b)While the conversions that relied on Froude's number achieved acceptable results.

(c) From the relationship which determining the surface load ,the conversions criterion was derived and obtained results even more accurate than from applying the Fr number.

This helped to obtain the successful study and applying the most successful modification scheme to a prototype unit could result in an improved treatment operation.

For the practical modeling ,the use of ratios [length scale ($\lambda L=\lambda r$) /depth scale ($\lambda D = \lambda h$) $\leq$ 5.0] reference is made to (Horvath,1984) on the similarity criteria related to settling of suspended solids.

According to Rouse,1945 length scale $(\lambda_L) = (\lambda r)$ =1/50, and ratio used $[(\lambda_r)/(\lambda_h)]$ =1/50.

The scaling factors could possibly be obtained by application of the previous equations, by using three types of scale ratios were taken to choose the best one from the following four Tables: (A-1),(A-2),(A-3),(A-4) as a results of scale ratio as shown in Appendix (A).

The best result for scale ratio selected from table (A- 2) as shown in the appendix (A),the lowest value of rotational velocity , and lowest value of SOR

, it can be good with value of ($9.89*10^{-3}$) ,(1.24) respectively because the lower rotational velocity and lower SOR it means good settling velocity, and according to (Rouse,1945) length scale (λ_l) = (λ_r) = (1/50),and ratio used [($\lambda_r/\lambda h$) = 1/4].

∴ The best value of scale factor ($\lambda = 4$) , $\lambda r = \frac{1}{50}$, $\lambda h = \frac{1}{12.5}$

It can be possible to be obtained the scaling factors by application of the previous equations ,using:

$\lambda r = \frac{1}{50}$, $\frac{\lambda r}{\lambda h} = \frac{1}{4}$, ∴ $\lambda h = \frac{1}{12.5}$

$\lambda_Q = \lambda r \text{ x } (\lambda h)^{1.5}$

$\lambda_Q = \frac{1}{50} \text{ x } (\frac{1}{12.5})^{1.5} = 4.525*10^{-4}$

The water treatment plant is composed of 10 clariflocculator tanks ,each with a capacity of 1050 m^3/hr, total flow rate is 10500 m^3 /hr.

For the design of the pilot plant, the flow rate of one clariflocculator was taken in this experimental study which is 1050m^3/hr as flow rate.

For the purpose of increasing the flow rate in one- clariflocculator tank has been doubled it by three stages to satisfied the maximum value of flow rate can be used to increase the quantity of water requires within criteria:

1. (1.5X = 1.5 X 1050m^3/hr = 1575m^3/hr).
2. (2.0X = 2.0X1050m^3/hr = 2100m^3/hr).
3. (2.5 X= 2.5X1050m^3/hr = 2625m^3/hr) .

Table (3-3a) and (3-3b) shows demonstrated the design parameters and design dimensions at of flow rates which is used in treatment stages of Kerbala (W.T.P).

Table (3-3a): Design dimensions and some of prototype design parameters for rapid mixing at different flow rates were used during the experimental work. In prototype units of Kerbala (W.T.P).

Units	area (m^2)	Water Depth(m)	Flow rate(m^3/h)	Flow rate for one clariflocculator basin(X) = 1050m^3/hr	1.5 X=1575	2 X=2100	2.5 X=2625
Rapid mixing	23.74	4.73	Detention time(10-120)sec According to(Kawamura,1976),(Peavey et, al.,1985)and (Smethurst,1997).	1050*(5tank)=5250 m^3/hr $T=\frac{v}{Q}=\frac{112.29}{5250}$=0.021h=1.28 min =77sec	51.33	38.5	30.8
	G> 750 S^{-1} According to(Kawamura,1976),(Peavey et, al.,1985)and (Smethurst,1997).			760S^{-1}	760S^{-1}	800	1000 S^{-1}
	Gt $(3*10^4-6*10^4)$According to (Syed R.Qasim,2002)			58520	39011	30800	30800

Table (3-3b): Design dimensions and some of prototype design parameters for flocculation basin and sedimentation tank at different flow rates were used during the experimental work. In prototype units of Kerbala (W.T.P).

Flocculation basin	151.85	3.91	Detention time(20-60)min $t = \frac{V}{Q} = \frac{151.85m^2\ x\ 3.91m}{1050m^3/hr}$	33.92	22.61	17	13.57
			G=(15-60)s^{-1} According to(Syed R.Qasim,2002)	29s^{-1}	30s^{-1}	30s^{-1}	30s^{-1}
			Gt=$2*10^4-6*10^4$) (Kawamura,1976).	59020	40712	30534	24426
Sedimentation tank	957	4.19	Detention time(2-8)h	3.82h	2.54h	1.9h	1.52h
SOR(Vo)=Q/A m/hr (1.2-4.5)m^3/m^2h				1.09	1.645	2.194	2.74
Horizontal velocity $(V_h)=Q/2\pi r H \le 0.3m/min$				0.035m/min	0.052m/min	0.07m/min	0.078m/min
Fr= $\frac{Q_h}{4\pi^2 r^2 H^3 g}$				$8.280*10^{-9}$	$1.86*10^{-8}$	$3.312*10^{-8}$	$5.175*10^{-8}$

Where Table (3-4) shows some experimental design parameters and design dimension at different types of flow rates used (during experimental works) in treatment stages of pilot plant of Kerbala (W.T.P).

By using scaling factor illustrated in table (A-2) when $\lambda = 4$, $\lambda_r = \frac{1}{50}$, $\lambda_{h} = \frac{1}{12.5}$ the value of flow rate in pilot plant is equal to $0.475 m^3/hr$ which equivalent of flow rate in real plant ($1050 m^3/hr$).

For the purpose of increasing the flow rate in one- clariflocculator tank at pilot plant has been doubled it by three stages to satisfied the maximum value of flow rate can be used to increase the quantity of water requires within criteria:

1. ($1.5X = 1.5 X 0.475 m^3/hr = 0.712 m^3/hr$).
2. ($2.0X = 2.0X 0.475 m^3/hr = 0.95 m^3/hr$).
3. ($2.5 X= 2.5X 0.475 m^3/hr = 1.18 m^3/hr$) .

Table (3-4) :Some experimental design parameters and design dimension at different types of flow rates used (during experimental works) in treatment stages of pilot plant of Kerbala (W.T.P).

Units	area (m^2)	Water Depth (m)	Flow rate(m^3/h)	0.475	0.712	0.95	1.18
Rapid mixing	0.047	0.21	Detention time (10-120)Sec According to(Kawamura,1976) ,(Peavey et, al.,1985)and (Smethurst,1997).	78	52.02	39	31.39
	G> 750 S^{-1} According to(Kawamura,1976),(Peavey et, al.,1985)and (Smethurst,1997).			760^{-1}	760^{-1}	800 s^{-1}	1000s^{-1}
	Gt $(3*10^4 - 6*10^4)$			59280	39520	31200	31390
Floccula- -tion basin	0.06	0.3128	Detention .time (min)	2.37	1.58	1.185	0.954
G=15-60 s^{-1}				29 s^{-1}	30 s^{-1}	30 s^{-1}	30 s^{-1}
Gt=$(10^4 - 15*10^4)$ (Gt flocc) :($2*10^4$-$6*10^4$) (Kawamura,1976)				4123.8	2844	2133	1717.2
Sedime- -ntation tank	0.382	0.3352	Detention time (min)	16.17	10.79	8.08	6.51
SOR(Vo) m/hr (1.2-4.5)m^3/m^2h				1.243	1.86	2.486	3.08
Horizontal velocity≤0.3m/min Vh=Q/2πrH				$9.89*10^{-3}$	0.0148	0.0197	0.024
Fr= $\frac{Q_2}{4\pi^2 r^2 H^3 g}$				$8.273*10^{-9}$	$1.86*10^{-8}$	$3.31*10^{-8}$	$5.105*10^{-8}$

3.4 The Experimental Equipment

An experimental integrated water station was designed, constructed and equipped for research purposes, and is illustrated in Figure (3-3), (3-4) and Plate(3-2).The physical pilot plant for this work represents a conventional water treatment station in Kerbala governorate, which is composed of the following parts, according to its function:

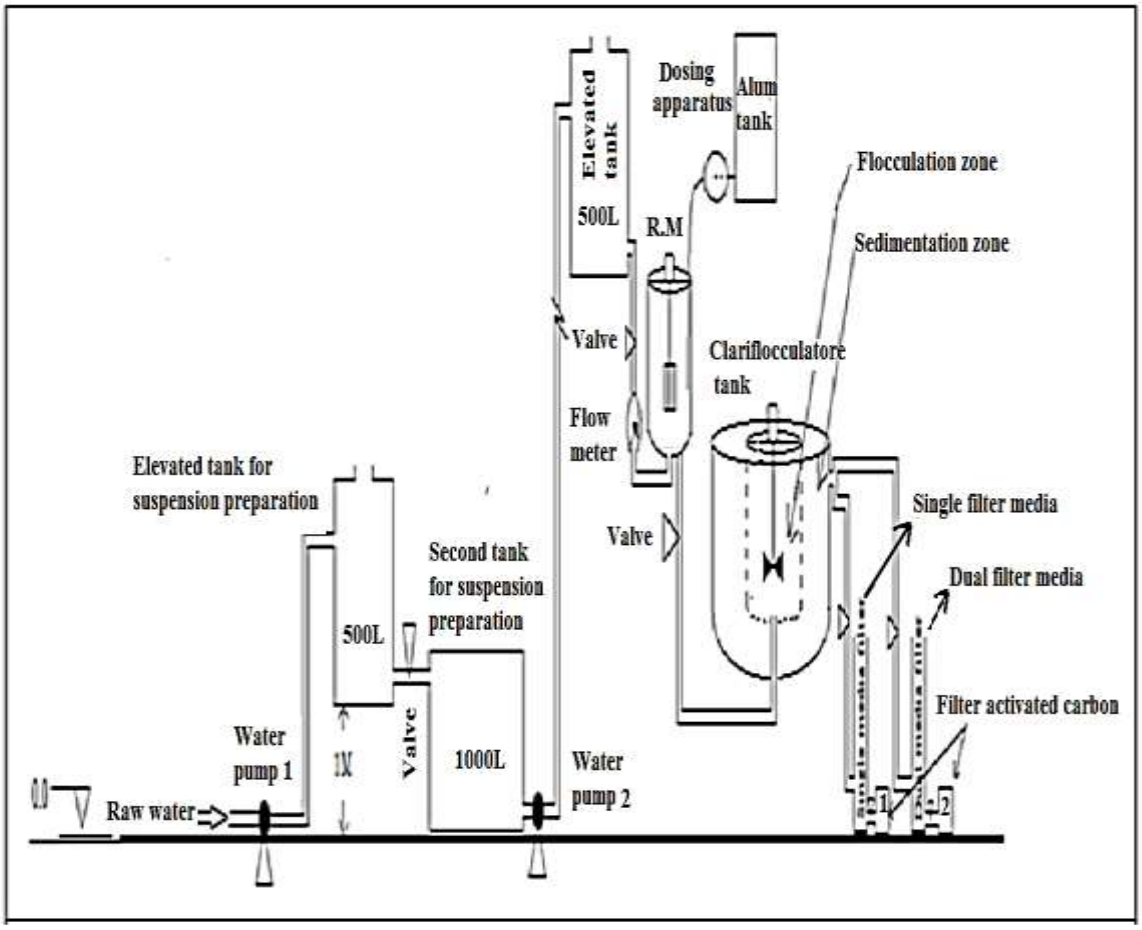


Figure (3-3): Schematic drawing of the pilot plant.

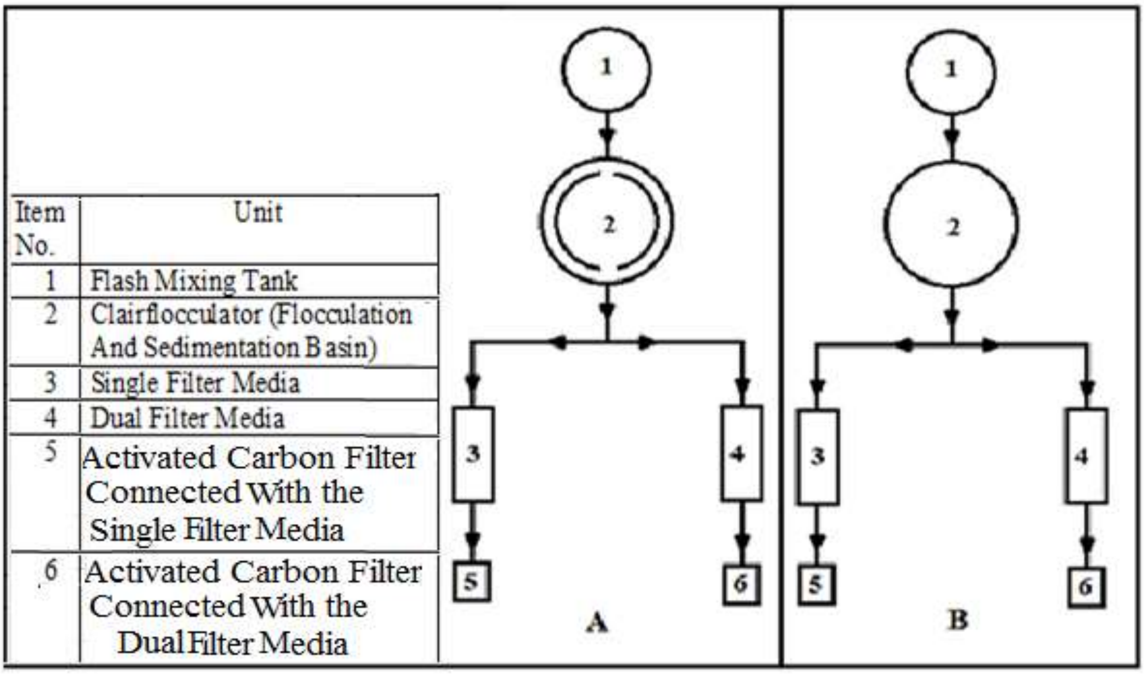


Figure (3-4):Flowchart for treatment water process in pilot plant(A: with plate settler &B: without plate settler).

Plate:(3-2) Ground level tank, Elevated tank ,Rapid mixing tank, Claroflocculator tank and Single filter media arrangement in the pilot plant .

3.4.1 Raw Water Collection Tank and Suspension Preparation Tank.

Two plastic tanks, one of them with a capacity of 500 liters located at a height of 1 m from the natural ground (in operation), were placed near the experimental station that draws raw water from the main tank (the receiving well) at the main station by connecting it to a water pump (0.75 kw ,Ø 250 mm out let pipe) at a distance of about 60 Meters, the required turbidity is prepared in this tank where the suspended clay is added in calculated weight ratios with a good mixing by a mechanical mixer . The rotating motors was of 400 volts, 0.67 Kw ,980 Rpm., GAMK-Turkey.

The other plastic tank with a capacity of 1000 liters was placed on a ground level , which collects the turbid water that was prepared in the first tank as shown in Plate (3-2).

3.4.2 The Elevated Tank

A plastic tank capacity of 500 liters has been installed at a height of (4m) from the ground level to facilitate the distribution of raw and suspended water to the rest of the subsequent experimental units, as shown in Plate (3-2).

For the purpose of keeping the suspension in this tank mixed during work, by means of continuous raw water circulation. To ensure mixing and circulation of water, a return tube of diameter (20 mm) was connected to the ground tank.

3.4.3 The Main Flow Meter

A standard (ZYIA INSTRUMENT COMPANY) Flow meter is device used to measure the main flow ,the range of (10-70 LPM,2-20 GPM), Plate (3-3) shows the flow meter fixed beside rapid mixing tank .

Before connect the flow –meter with the flow pipe ,it was calibrated by using graded cylinder and stopwatch.

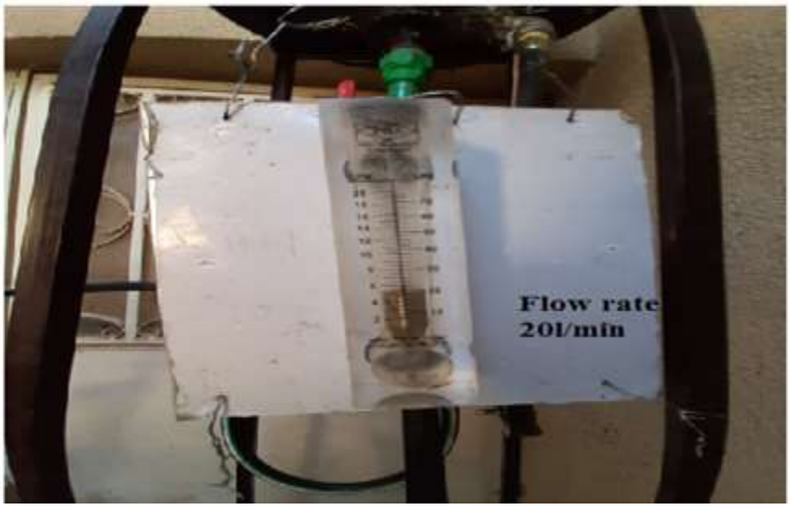


Plate (3-3) Main flow meter

3.4.4 Rapid Mixing Tank

A steel round basin with a diameter of 0.245 m and a height of 0.21 m as shown in Plate (3-4) ,and Figure (3-5) was used as coagulation tank.

A mixing motor of (400 volts, 0.67 Kw ,1500 Rpm) was fixed above flash tank ,equipped with mechanical propeller which installed inside the basin . The coagulant was introduced at the influent point of the tank .The propeller's rotation frequency (N) was measured at the maximum system flow rate and in operating conditions with a "Digital tachometer", range (2.5 to 99.999 rpm). Plate (3-5) shows a "mechanical tachometer, the stirring speed (N) (at maximum flow rate) was always greater than (697.41) rpm. This frequency can result in a G-value of $> 750\ s^{-1}$.

The relationship of G-N was calculated as $G = 0.041N^{(1.5)}$ rpm, as stated in Appendix (B), which is always greater than G value of the min. requirement of ($750\bar{s}^{1}$) according to (Smethurst,1997) and (Jiang et al.,1996).

Plate (3-4): Flash mixer tank.

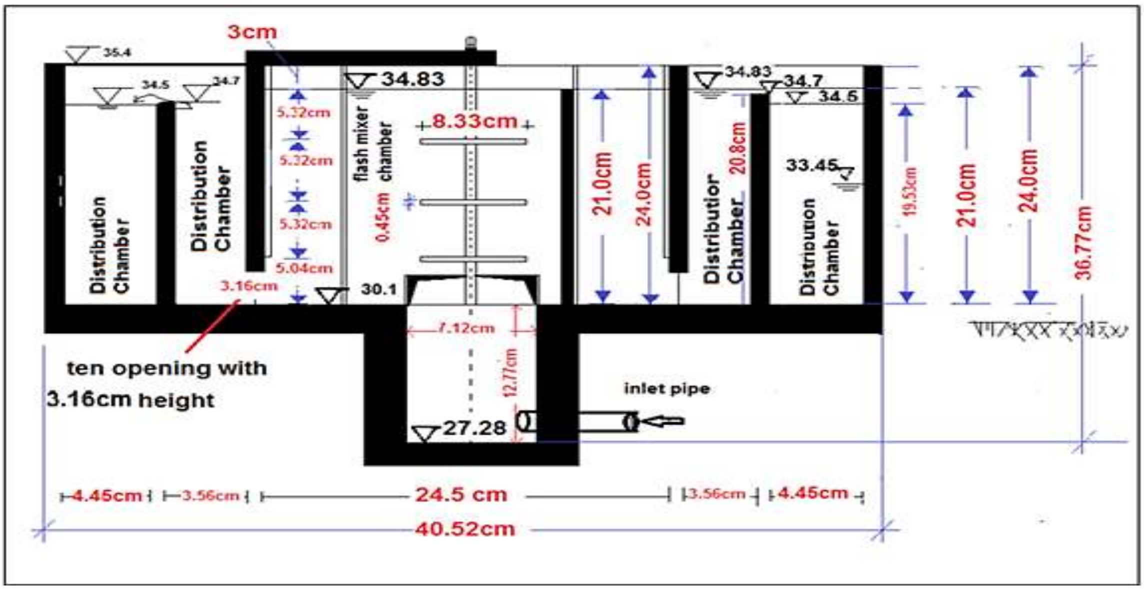


Figure (3-5): Section of flash mixer tank.

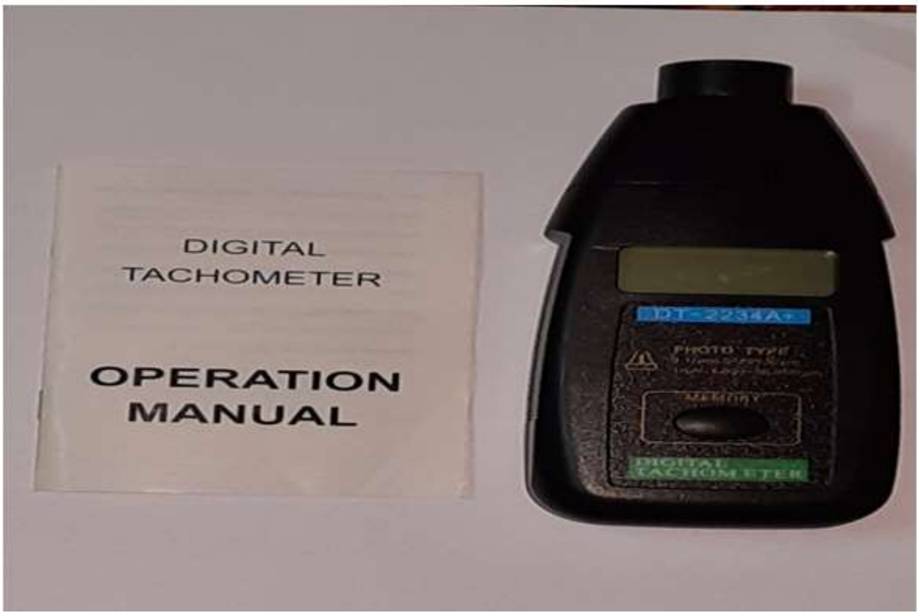


Plate (3-5) Digital Tachometer

3.4.5 The Conventional Clarifloculator Tank

The scale model of conventional clariflocculator was designed for a flow rate of (0.475 -1.18 m^3/hr). Clariflocculator consists of two concentric tanks, the inner tank acts as a flocculation basin and the external tank acts as a sedimentation basin.

In this basin, the water flowing from flocculation tank enters the sedimentation tank through many opening at the bottom of the tank ,on the wall that separating the flocculation zone and the sedimentation zone, and thus the water goes to the top of the sedimentation tank distributed radially on all sides of the basin and uniformly ,the settled water flows across the radial launder weir which located at the top circular tank (V-notch) ,thus the settled water goes to filters. Plate(3-6) shows the working model for clariflocculator tank. The size of pipe that connected between rapid and clariflocculator of 2.5 cm diameter was provided to transfer the coagulated water from the rapid mixing unit to the clariflocculator .

The diameters of clariflocculator and clarifier were 72.8 cm and 26.8 cm respectively. The sludge drainage line is provided with a valve in the lower middle part of the filter basin to remove sludge at regular intervals. The settled water is collected through a circular channel beside the tank from the inside and at the top of the clariflocculator basin it has two water exit holes for the purpose of transferring it to the filters.

In the flocculation zone, a rotating mechanical fan was inserted into the basin. The mixing motor was [400 V, 0.67 kW, 1500 rpm]. The speed of the mixer is controlled by an electrical convertor device Plate (3-7) illustrated the convertor device.

The frequency range of this converter of (0-50) Hz, which can be adjusted when a specific number of revolutions per minute is required. According to (McGhee,1991), the G value of flocculation at its range of (20-60)s^{-1}.At any flow rate value ,the (G-N) relationship was calculated as $G_{floc=}0.05N^{3/2}$ rpm as given in Appendix(B) .Table(3-4) shows the detention time in this tank in model of Karbala (W.T.P.) at different flow rates which should be used in this study .

Plate(3-6):Clariflocculater tank model(Flocculater and Sedimentation tank).

Plate (3-7):Electrical speed convertor device type(freqrol-S500) Mitsubishi, for slow mixer .(Also used in rapid mixer).

3.4.6 Alum Solution Tank

A round plastic tank of (20) liter capacity is placed over the rapid mixing tank as shown in the Plate (3-2), used for alum solution at a concentration of 1.0% that is controlled by a giving apparatus.. The purpose of this tank is to inject the alum solution into the rapid mixing tank as coagulant. The giving apparatus was connected to a small plastic tube to deliver the coagulant into the rapid mixing tank, in an amount measured by volume to time (number of coagulant drops per unit time).

3.4.7 The Settling Plates Settler

A semicircular shape was used for the plate settling module unit in the sedimentation basin. Regular plate sedimentation units holder arranged nicely to hold two plates of different diameter to be easily placed in the sump and removed .The inclination of the two panels is arranged at an angle of (60̊) with the horizontal , Plate (3-8) and Figure (3-6) illustrates this frame. The plats were made of aluminum sheet of (1.0 mm) thickness, the dimensions of each plate is (68 cm diameter in top ,50 cm at bottom, height of plate 16cm) and (58cm diameter in top ,42cm at bottom, height of plate 16cm).

The purpose of the parallel plates settler is to increase the settling capacity of the circular clarifier sedimentation basins by reducing the vertical distance a floc particle must settle before agglomerating to form larger particles ,also increase the surface area with decrease the surface overflow rate ,then increase the efficiency of removal .

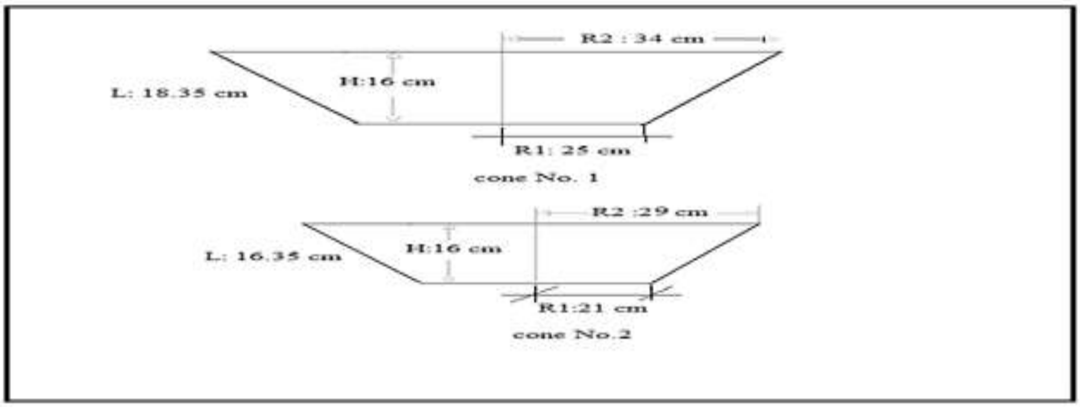


Figure (3-6):Aluminum plate settler

Plate (3-8) :Plate settler

3.4.8 The Common Filters Distribution Unit

After sedimentation ,the settled water is distributed equally to the two filters. Each filter was connected with the sedimentation tank using hose- pipe of Ø 20 mm one to each filter.

3.4.9 Filter Units and Accessories.

Two filters were used in this study ,the size of each one (0.34m length X0.17m width X2.2m height), made of galvanize plate with thickness 1.5mm.

The first filter was filled with four graded layers of gravel in the bottom ,the total depth of support gravel layer is 500mm . The media of this filter is silica sand with bed depth 700mm placed above the support gravel ,this is called single media.

The second filter is also filled with four support gravel layer of total thickness 500 mm on the bottom , above the support gravel placed silica sand 350 mm thickness and above silica sand was put anthracite of 350 mm .this is called dual media. Each filter container contains a Partition inside it ,separate the media and water flow ,this illustrated in Plate (3-2) .

Also, a network of plastic pipes of specific diameters was designed and installed under the support layer of gravel, whose function is to withdraw the filtered water and transfer it to the activated carbon filter basin, in addition its used in backwashing.

3.4.10 Optimization of Water Treatment

3.4.10.1 Optimization of Water Treatment Using Dual Media

Generally dual – media filters usually consist of a layer of silica sand with a depth ranging from 0.15 to 0.4 m, and above it a layer of anthracite coal with a depth of 0.3 to 0.6 m. (Peavy et al.,1985).

To produce a good separation of the particles or obtaining the degree of mixing after backwashing depends on the selection of both the size and the uniformity coefficients of the two media. (Cleasby ,1972).

Due to the nature of large pores of anthracite, it removes large particles and flocs., while most of the smaller particles penetrate the large pores in the anthracite layer and pass to the bottom where the sand layer exist before it is removed.

Therefore, dual media filters have the advantage of more efficient use of pore space for storage. So the filter operation period is longer and the output rate is higher due to the reduce head losses.

The filter material in dual media filters is loosely fixed in the anthracite layer, and this is one of the disadvantages of these filters.

Any sudden increase in hydraulic loading leads to the destruction of the layer and the displacement of its material and its transfer to the bottom at the surface of the sand layer, which leads to the rapid bonding at this level. (Peavy et al.,1985).

3.4.10.2 Optimization of Water Treatment Using Activated Carbon

Activated carbon is a material with multiple uses and has many applications in many fields, being it has many features, such as :high surface area, internal pore structure consisting of big, middle and macro pores, large porosity ,In addition to a wide range of functional groups present on the surface of activated carbon ,but especially in the field of an environment.

Activated carbon has more efficiency in removing organic compounds than inorganic minerals and contaminants, and has a granular shape with various sizes or fine grains, where has a distinctive property of adsorption due to its high porosity and large surface area that allows it to remove and retain many of the impurities present in water.(Hoboken,2003).

Activated carbon has become the most important odor-removing material available.

Organic matter is an important component of water that affects treatment performance in drinking water operations and drinking water quality. As a result, it requires extensive use of coagulants, disinfectants ,oxidizers , and in addition to being a formation of disinfection byproducts (Zouboulisa et al.,2007).

3.5 Test Run Materials

3.5.1 The Coagulant

Alum solution used in the experimental work was prepared by dissolving a certain weight of alum in a known volume of distilled water to give the desired (1%) strength . Alum is used in almost all water treatment plants. The alum used in this study is of Turkish origin.

The optimal doses of alum solution were experimentally made in the laboratory of the water treatment plant in Karbala using the jar test device for the different turbidities used in this study. Alkalinity determination for the raw water showed that it was sufficient for this alum to react with water without needing pH -adjustment .The optimum doses of alum solution were made experimentally in laboratory using the jar- test apparatus as (35mg/l) for the range of turbidities of (150-200 NTU) used in this work ,and for turbidities (20-30-40-50 NTU),the doses of alum solution were made experimentally in laboratory using jar-test apparatus it were (3mg/l ,6mg/l, 7mg/l ,8mg/l) respectively.

3.5.2 Suspension Preparation

For the purposes of preparing the suspended material required to be used in the experimental work, fine clay taken from the Euphrates River in the vicinity of the low lift station, where the unwanted suspended coarse materials were removed from it and dried by an electric oven and the mixing suspension was prepared from a certain weight of dried slurry and mixed with water taken from the river after measuring its turbidity and leaving the mixture for at least 24 hours for the purposes of homogeneity and obtaining a ready suspension.

Another material that has been used to produce a suspension with high turbidity is bentonite, which is a naturally occurring aluminum silicate clay.

In this study, bentonite clay that is commercially available and manufactured in Saudi Arabia was used.

One liter of raw water (river water) was prepared and its turbidity examined, after that a certain weight of bentonite was added to that liter of raw water ,and the degree of its turbidity was measured, it must continue to add the bentonite until the required turbidity is reached provided that the readings of the bentonite addition are recorded each time with turbidity for each addition, and thus the relationship between the amount of added bentonite and the turbidity value is drawn, for the purpose of finding the required turbidity it should be determine the turbidity value and dropped it on the curve to get the amount of bentonite that request.

Table (3-5) gives the equivalent mg/l of bentonite for each value of NTU of turbidity run in this study (the turbidity of raw water that used in this experimental was (12.0 NTU).

Table (3-5): The different doses of bentonite used according to the value of each turbidity.

Turbidity(NTU)	15.4	21	21.7	29.6	41.3	53.6	62.7
Bentonite in mg/l	15	30	45	60	75	90	100

3.6 The Materials and Devices

3.6.1 The Turbid Meter

For turbidity measuring of water ,turbid meter type (HANNAHI88703- -Tubidi--meter) was used . it's easy to use and measure. Plate (3-9) shows the Turbid meter used in this study .

Testing mechanism: The basic of this method depends on comparing the intensity of light dispersed by the water sample at specific conditions with the intensity of light dispersed by a standard water sample as reference suspension at the same conditions.

The value of turbidity depends on the intensity of the scattered light.(APHA,1999).

The turbidity device specifications are shown in the Table (3-6).

Table (3-6): Instrument specifications

Model Number	Digital Nephelometer Model –(HANNAHI88703)
Range	0-4000NTU 0-26800Nephelos 0-980 EBS

Plate (3-9) :Nephelometric turbid meter (HANNA-HI88703).

3.6.2 The Balance

Electronic device used to weighing materials such alum ,dried clay ,and bentonite used in experimental work , type (Kern) with max. capacity 220 g and minimum capacity is 10mg and sensitivity of (±0.1mg), Plate (3-10) shows this electronic balance.

Plate (3-10) : Kern Digital balance.

3.6.3 Jar –Test Equipment

A (FLOCUMATIC) sedimentation jar tester ,was used to extract the optimal dose coagulase material that should be used in the coagulation process.

The working and measurement procedures as prescribed by the manufacturer have been strictly followed. Plate(3-11) shows the jar-test device.

Plate (3-11) :Jar-tester used in the study

3.6.4 Magnetic Stirrer

A digital magnetic stirrer type (AGIMATIC-ED) was used for mixing alum with distilled water to prepare alum solution in the experimental work to give the desired (1%) strength. Plate (3-12) shows this apparatus.

Plate (3-12):Magnetic Stirrer.

3.6.5 Electrical Oven

An electric furnace type (J.P.SELECTA,s.a.230VAC,50/60Hz) manufactured in Spain was used to dry the soil used in the preparation of turbidity. Plate(3-13) illustrate the electrical oven.

Plate (3-13): Electrical oven.

3.6.6 Electrical Conductivity

Electric Conductivity Meter type (HANNA - EC215) is the measure of solution's ability to allow the transport of an electric charge Plate (3-14) shows Conductivity Meter .the measurement units are :

- S/cm (Siemens/cm).
- $\mu s/cm$(microsimens/cm).

Plate (3-14) Electric Conductivity Meter.

3.7 Experiments Design

In order to achieve the aims of this study , the parameters considered were divided into two major parts:

A: The pre-filtration and sedimentation works.

B: Filtration works.

Several groups of experiments were designed for each part using a specially controlled conceptual scheme as described below.

3.7.A The Pre–filtration-Sedimentation Research Work (Experimental Control Consideration).

The design parameters must be satisfied the coagulation/flocculation experimental conditions and requirements.

At different flow rates, the minimum retention time value and the optimum velocity gradient value were controlled as shown below.

1.The velocity gradient G and min. Gt values in rapid mixing and coagulation process were applied to be within the requirements, G > 750 S^{-1}according to (Kawamura,1976),(Peavy et al.,1985), and Gt values of ($3X10^4$<Gt >$6X10^4$) according to (Qasim , 2002).

2. The values of the flocculation unit ,G_f of range (15-60) s^{-1} and Gt values of ($10^4 - 15X10^4$) were well controlled during the tests by adjustable frequency of the flocculation motor. Table (3-7),showing this part of experimental works for each value of raw water, according to(Qasim. , 2002).

Table (3-7): Different values of the flocculator speed and the ($G_{floc.}$) were used in this study .

(N) RPM	44.8	45.2	63.6	71.1	86.89	93.2	100.5	106.8	112.9
$G_{floc.}$(S^{-1})	15	15.2	25.4	30	40.5	45	50.4	55.2	60

3. The dosage of alum solution obtained as per Jar –test, for each value of turbidity . The alum solution is prepared according to the required dosage in the mixing tank.

4.The rate of discharge controlled through a calibrated flow meter of (0.475-1.18)m^3/h, rate range.

5. The value of pH range (7.4-8) ,water temperature usually within 30 C̊.

6. In this study four types of prepared raw water turbidity were used from mixing river soil with river water used in the treatment plant ,namely (20,30,40, and 50)NTU, in addition to using another four types of turbidity that prepared from mixing bentonite with river water which is required to treated it in the treatment plant, namely (20,50,120 and 200) NTU.

3.7.B The High Rate Sedimentation Research Works

3.7.B.1 The Purpose of The High Rate Sedimentation Works

The main purpose of this part of the experiment work is to improve both of the carrying capacity and the quality of the water produced at the Kerbala water treatment plant by using the inclined plates settler at the best inclination angle.

3.7.B.2 Experimental Control Consideration

The measured parameters for each experiment (the dependent variables) are the stabilized water effluent turbidity at:

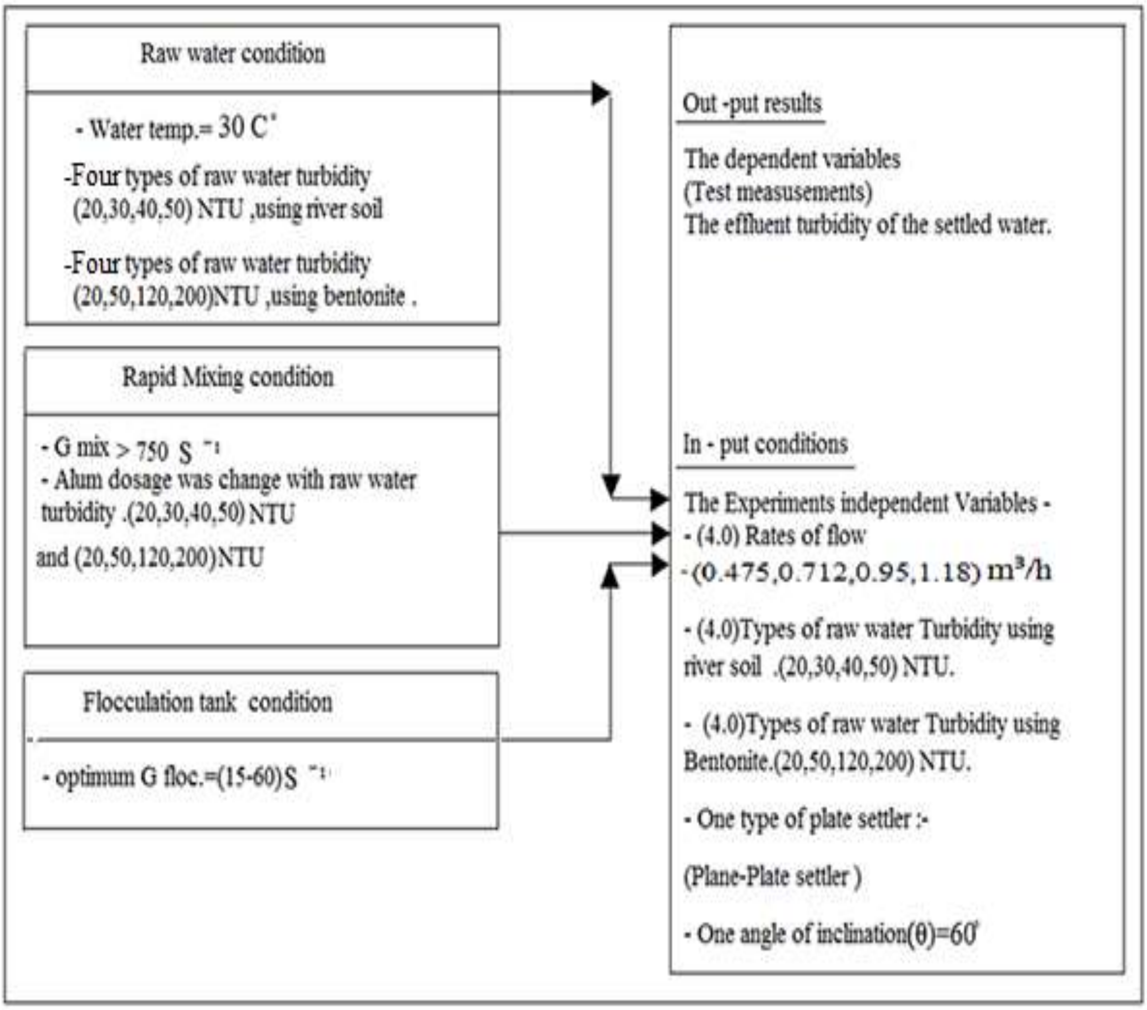
A. Angle of inclination ($\theta = 60^\circ$ (JA Salvato, 2003), (Cata ,1995).

B. Different flow rate.

C. Different raw water turbidity .

D. One type of plate settlers,[plane-plate settler].

The experimental testing space and the research works range are shown in the Figure(3-7).



Figure(3-7):The experimental testing space, and shows the range of research works.

3.8 The Sets of Total Experimental Works

3.8.1The Sets of Flocculation Experiments

For the purpose of obtaining the optimum values for $G_{floc.}$ and estimating the range of optimal values for $G_{floc.}$, eight experimental groups were designed for four types of raw water turbidity (using river soil to prepare the required turbidity), and four other types of turbidity (using bentonite to prepare the required turbidity) .

1- Set (1S. to 4S.) in conventional settling tanks at the range of ($G_{floc.}$) values of (15-60) S^{-1}, as shown in the table (3-4) for each of four different flow rate with raw water turbidity =(20 ,30,40,50) NTU respectively,(using river soil).

2- Set (5S. to 8S.) in conventional settling tanks at the range of ($G_{floc.}$) values of (15-60) S^{-1},as shown in the table(3-4) for each of four different flow rate with raw water turbidity = (20,50,120,200) NTU, respectively ,(using Bentonite).

3.8.2 The Sets of Sedimentation Experiment

Three sets of experiments were design for sedimentation test work. They are as follows:

1.Set (1S. to 4S.) for conventional settling basin (without –plate settler) at four different flow rate at raw water turbidity of (20,30,40,50) NTU respectively.

2.Set (5S. to 8S.) for conventional settling basin (without –plate settler) at four different flow rate at raw water turbidity of (20,50,120,200) NTU respectively.

3.8.3 The Sets of High Rate Sedimentation Experiments

To perform the high-speed sedimentation test, three sets of experiments were designed, each group conducted as follows:

1-Set (1S. to 4S.) for high rate settling tanks (Inclined plate settler),2 plates at angle 60° of inclination and at four different flow rate at raw water turbidity of(20,30,40,50) NTU respectively.

2-Set (5S. to 8S.) for high rate settling tanks (Inclined plate settler),2 plates at angle 60° of inclination and at four different flow rate at raw water turbidity of(20,50,120,200) NTU respectively.

3.9 The Filtration Research Work Experiments

3.9.1 Experimental Control Considerations

In order to achieve full confidence in the results, some important points have been addressed and taken into consideration throughout the work, including:

3.9.1.1 Influent Turbidity Control .

To unify the experiment condition on the work undertaken, sets of two parallel filter gallery were made to provide the possibility of better and clear comparison between their performance .first filter with sand filter bed (single filter), and the other one is dual filter (sand+ anthracite bed).

3.9.1.2 Constant Head /Constant Rate of Flow .

The steady flow rate of each type of filter was performed by continuous inspection of the calibrated filter flow meter. The condition of constant head was covered throughout the tests at the level of the over- flow drain pipe in the filter distribution unit.

3.9.1.3 Filter Depth

The effective working water head on the top of the filter media was (200±2 cm) .The total media depth was designed in the single filter as 65 cm for all the test runs, while the dual media depth was designed as 35cm sand instead of 65cm sand and 30 cm anthracite in order to assimilate well the best actual working media depth condition to provide a good comparison between the experimental and the actual filters performance results.

3.9.2 Area of High Rate Filtration Research Works

The feasibility of developing the existing water treatment plant and making it workable through optimizing pre-filtration and filtration processes is the main requirement for this work.

As for the technique of high rate filtration, the search strategy included studying and comparing the results of the effects of some independent variables of filtration on some dependent variables, and the filtering processes (independent variables) under study were the filter media materials such as sand, anthracite, and activated carbon, filtering techniques (single and double media) and filtration rates (5-19)m/h.

3.10 Application of the Water quality index (WQI)

Calculating and formulating the water quality index using weighted arithmetic index , includes three steps with three equations, which play a very important role in determining the indicator, as shown in following steps (Brown ,1972 and Joshi ,2009).

Step 1:- To obtained the value of (qn) ,which is the quality rating or sub-index, using the following formula :

qn= $\{ [\frac{V_{n-V_0})}{(S_{n-V_0})}] * 100\}$---(3-4)

Where:

V_n = Estimated value of each parameter from the water analysis.

V_i or V_0 = The ideal value of each parameter counted as zero, except the value of pH parameter =7 and D_0= 14.6 mg/L

S_n= The standard parameter recommended of the water quality i.e. the (IQS-417,2001).

Step 2:-In this step ,the relative unit weight of the parameter (Wn) can be calculated by using the following equation :

$$Wn = \frac{K}{S_n} \text{ --- (3-5)}$$

Where:

K is the proportionality constant ,it can be found by the formula:

$$K=\frac{1}{\Sigma(\frac{1}{V_n})} \text{ ---(3-6)}$$

Step 3- In this step it can be found the total Arithmetic Water Quality Index (WQI) using the equation:

$$WQI = \frac{\sum q_n * W_n}{\sum W_n} \text{ --(3-7)}$$

Table(3-8): shows the categories of water classification based on the weighted Arithmetic index value .(Chaturvedi , 2009. and Mishra, 2001).

Table (3-8):The categories of water classification based on the weighted Arithmetic index value.(Chaturvedi,2009. and Mishra,2001).

The Value of Water Quality Index	Category of Water Quality	Grading
0-25	Excellent	A
26-50	Good	B
51-75	Poor	C
76-100	Very Poor	D
> 100	Unsuitable for drinking	E

3.11 Physiochemical Parameters Were Used In (WQI)

There are many physiochemical parameters were used in this study such as: Turbidity (Turb.), pH, Electrical Conductivity (EC), Total Dissolved Solids (TDS), Alkaline (Alk.) Calcium (Ca^{+2}), Magnesium (Mg^{+2}), Sulfates ($SO4^{2}$), Chlorides (Cl), and Total Hardness (T.H). The Iraqi standard for drinking water

No. 417 of 2001 is shown in Table (3-9) and selected to progress the required index.

Table(3-9): The Iraqi standard for drinking water No.417 of 2001 for physiochemical parameters of water purification.

Parameter	Tur. NTU	pH	E.C $\mu S/cm$	Alk. mg/L	T.H mg/L	Ca mg/L	Mg mg/L	Cl mg/L	SO_4 mg/L	TDS mg/L
Standard Value	5	8.5	2000	125	500	75	50	250	250	1000

In pilot plant it was used four parameters to calculate the water quality index which are (turbidity value , PH, electrical conductivity , total dissolved solid). When compared efficiency removal based on water quality index for real plant with water quality index for pilot plant , four parameter were used for each plant.

3.12Calculation of Removal Efficiency of Water Treatment Plants.

The evaluation of the removal efficiency of water treatment project was calculated by defining a WQI for raw and treated water. Equation (3-8) illustrate the calculation of efficiency.

$\%E=\frac{WQI\ of\ raw\ water - WQI\ of\ treated\ water}{WQI\ of\ raw\ water}*100$------(3-8). (Alobaidy, A.M et al.,2010).

Chapter Four

RESULTS AND DISSCUSION

CHAPTER FOUR
RESULTS AND DISSCUSION

4.1 Euphrates River Turbidity

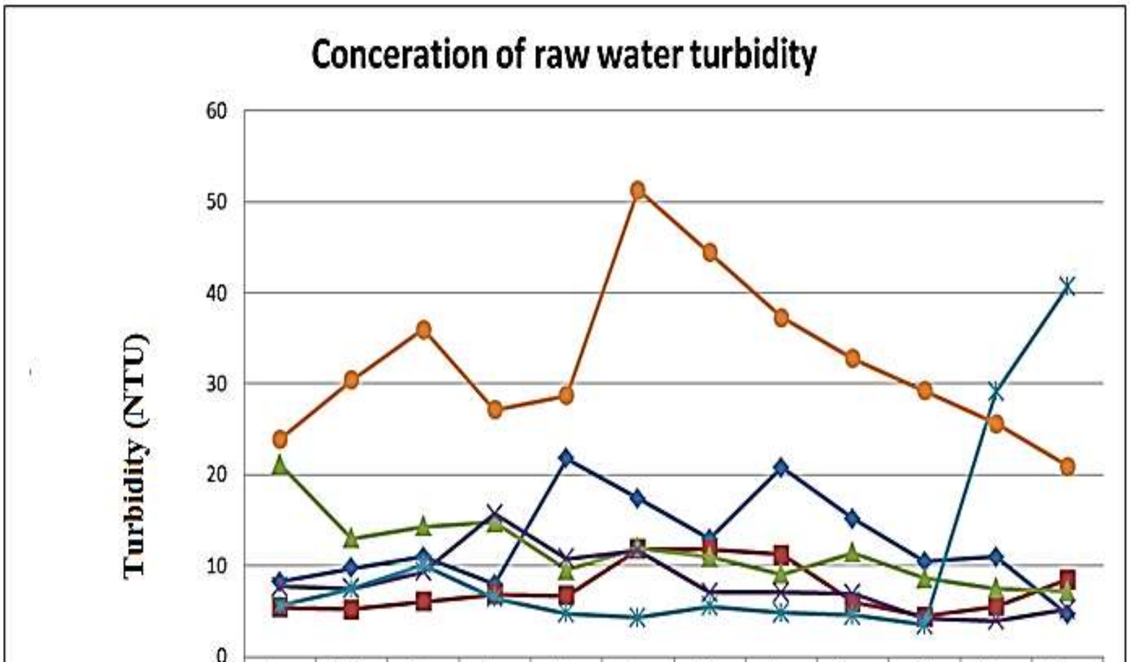
In Kerbala Governorate, the Euphrates River is the only supply of water for water treatment plants. The turbidity of the river water is low in all months of the year, according to data from tests done in the water treatment plant in Kerbala governorate from 2014 to 2019.Table (4-1) displays the monthly values of turbidity from 2014 to 2019.

Table (4-1) :The monthly mean values of raw turbidity over the years (2014 - 2019) in Kerbala water treatment plant.

month	Turb.(NTU) 2014	Turb.(NTU) 2015	Turb.(NTU) 2016	Turb.(NTU) 2017	Turb.(NTU) 2018	Turb.(NTU) 2019
Jan.	8.27	5.442	21.14	7.718	5.64	24.00
Feb.	9.77	5.254	13	7.53	7.48	30.41
Mar.	11	6.07	14.36	9.35	10.14	36.00
Apr.	8.00	6.91	14.81	15.73	6.47	27.13
May	21.79	6.77	9.54	10.83	4.77	28.72
June	17.44	11.82	11.97	11.75	4.34	51.27
July	12.99	11.76	10.98	7.09	5.58	44.42
Aug.	20.78	11.24	9.13	7.06	4.88	37.28
Sep.	15.17	6.03	11.48	6.97	4.57	32.80
Oct.	10.5	4.45	8.63	4.29	3.52	29.25
Nov.	11.03	5.58	7.56	4.00	29.22	25.64
Dec.	4.68	8.50	7.20	5.24	40.71	20.97

Figure (4-1): Depicts the monthly mean values of turbidity for the years 2014 to 2019 in kerbala water treatment plant .

Following the data in the graph below, there is evidence that the turbidity from year to year is between the limits (4-20) NTU approximately from (2014-2018).This low turbidity in raw water is due to the storage of water before the Al-Hindiya barrage on the Euphrates River, which causes stagnation of water before the dam, where the water intake is located. As well as the source of the water that feeds the river is from the water stored in the barrage , which stagnates in the reservoir.



	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
raw water tur.2014	8.27	9.769	11	8.005	21.79	17.44	12.99	20.78	15.17	10.5	11.03	4.681
raw water tur.2015	5.442	5.254	6.071	6.91	6.774	11.82	11.76	11.24	6.029	4.447	5.576	8.5
raw water tur.2016	21.135	13	14.36	14.805	9.54118	11.9722	10.98	9.12857	11.4786	8.63125	7.56	7.20526
raw water tur.2017	7.718	7.53	9.345	15.73	10.83	11.75	7.088	7.055	6.967	4.285	3.997	5.244
raw water tur.2018	5.64	7.478	10.14	6.474	4.767	4.335	5.576	4.878	4.565	3.517	29.12	40.71
raw water tur.2019	23.9557	30.4105	35.9889	27.13	28.725	51.2706	44.4182	37.2769	32.7933	29.25	25.635	20.971

Months

Figure (4-1) :The variation of turbidity for Euphrates River from 2014-2019.

Figure (4-2) Shows a frequency distribution of turbidity in kerbala water treatment plant. The values of turbidity in kerbala plant vary along years 2014-2019.

From the data of turbidity in kerbala water treatment plant it's found that the minimum turbidity was 4 NTU and maximum turbidity was 100NTU ,therefor the values of turbidity in the pilot plant have been selected match the water turbidity for years 2014-2019 , with an increase in preparation turbidity to 200 NTU using bentonite , for the purpose of knowing the effectiveness of the bentonite when used in the preparation of raw water.

For preparing raw water turbidity by using river soil, four different turbidity values were determined that corresponded to river water turbidity were : (20,30,40 and 50 NTU). In the same way it was used four different turbidity values of bentonite were: (20,50,120 and 200 NTU) for the purpose of knowing the effectiveness of the bentonite when used in the preparation of raw water, also for deterring the best value of turbidity which can be used.

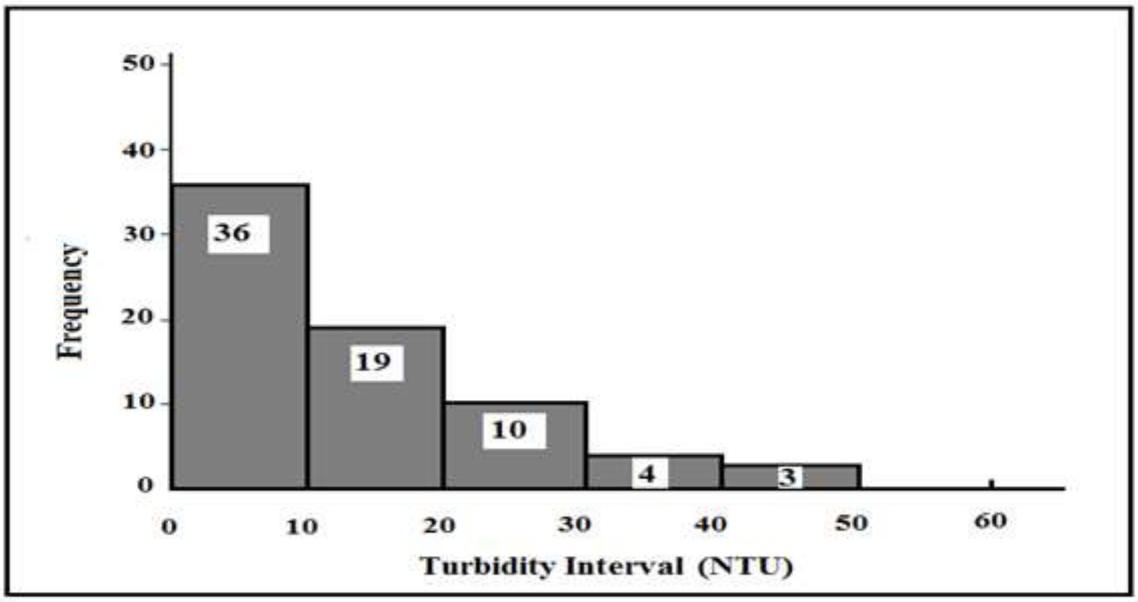


Figure (4-2): Distribution of the frequency of turbidity during the years 2014-2019 at the water treatment plant in Kerbala.

4.2 Water Quality Index and Efficiency Removal for Real Turbidity in Water Treatment Plant.

To assess the water quality index and Removal Efficiency for real turbidity in water treatment plant , and compare it with the water quality index and Removal Efficiency for pilot plant, it was selected the value of turbidity from the data available for the plant 2014-2019 is 20NTU , and extract its water quality index and to compare it to the water quality index of the pilot plant.Four parameters: (turbidity , total suspended solids, electrical conductivity, pH) were used to calculated the water quality index .

4.2.1 WQI for Real Turbidity in WTP at Conditions "Turb.= 20 NTU, Q = 0.475 m^3/hr".

From Table (C-65) page (C-32) to table (C-70) page (C-37) , the annual average of physio-chemical parameters in real plant at turb. 20 NTU , it can be calculated the results of the water quality indicator for the water treatment plant in Kerbala for the years (2014-2019) using the weighted mathematical indicator are shown in Table (4-2).WQI valuations are increased from 2014to 2017 and then reduced for 2018 till 2019.

Table (4-2) :WQI at "Turb.= 20NTU, and Q = 0.475 m^3/hr",for period 2014 to 2019.

Type of water	water quality index (WQI)					
	2014	2015	2016	2017	2018	2019
Raw water	273.34	261.55	264.33	263.71	263.69	263.72
Treated water	34.88	29.13	24.6	23.03	31.10	32.01
	Good	Good	Excellent	Excellent	Good	Good

4.2.2 Removal Efficiency for Real Turbidity at Conditions "Turb. = 20 NTU and Q= 0.475m³/hr"

The removal effectiveness for raw and processed water may be calculated using Weighted Arithmetic WQI based on the values in Table (4-2). The efficiency calculation could be seen using Equation (4-1).

$$\% E= \frac{WQI\ of\ raw\ water - WQI\ of\ treated\ water}{WQI\ of\ raw\ water} *100 \text{ ----------------------(4-1)}$$

Table (4-3) shows the removal Efficiency using Weighted Arithmetic WQI of Kerbala WTP from year 2014 to 2019. The removal efficiencies are increased from 2014 to 2017 and then reduced for 2018 till 2019.

Table (4-3): The removal efficiency from year 2014 to 2019

Year	2014	2015	2016	2017	2018	2019
Removal Efficiency %	87.24	88.86	90.69	91.26	88.20	87.86

4.3 WQI and Removal Efficiency in Pilot Plant

In this investigation, eight turbidity values were chosen, four of which were created using river soil (20, 30, 40, 50) NTU and the other four using bentonite (20, 50, 120, 200) NTU. For each turbidity value, four flow rates (0.475, 0.712, 0.95, and 1.18) m^3/hr were employed, with eight distinct filtering units produced as scenarios, as well as a sedimentation unit (plate settler vs. no plate settler), as indicated in Table (4-4) and Figure (4-3) below.

Table (4-4) :Eight units of filtration , two units of sedimentation process as operation Scenarios were used in the study.

Item No.	Unit of filtration	Scenario No.
1	Sedimentation unit (without using plate settler)	1
2	Sedimentation unit (using plate settler)	2
3	Single filter media (without using plate settler)	3
4	Single filter media (using plate settler)	4
5	Dual filter media (without using plate settler)	5
6	Dual filter media (using plate settler)	6
7	Activated Carbone in Single filter media (without using plate settler)	7
8	Activated Carbone in Single filter media (using plate settler)	8
9	Activated Carbone in Dual filter media (without using plate settler)	9
10	Activated Carbone in Dual filter media (using plate settler)	10

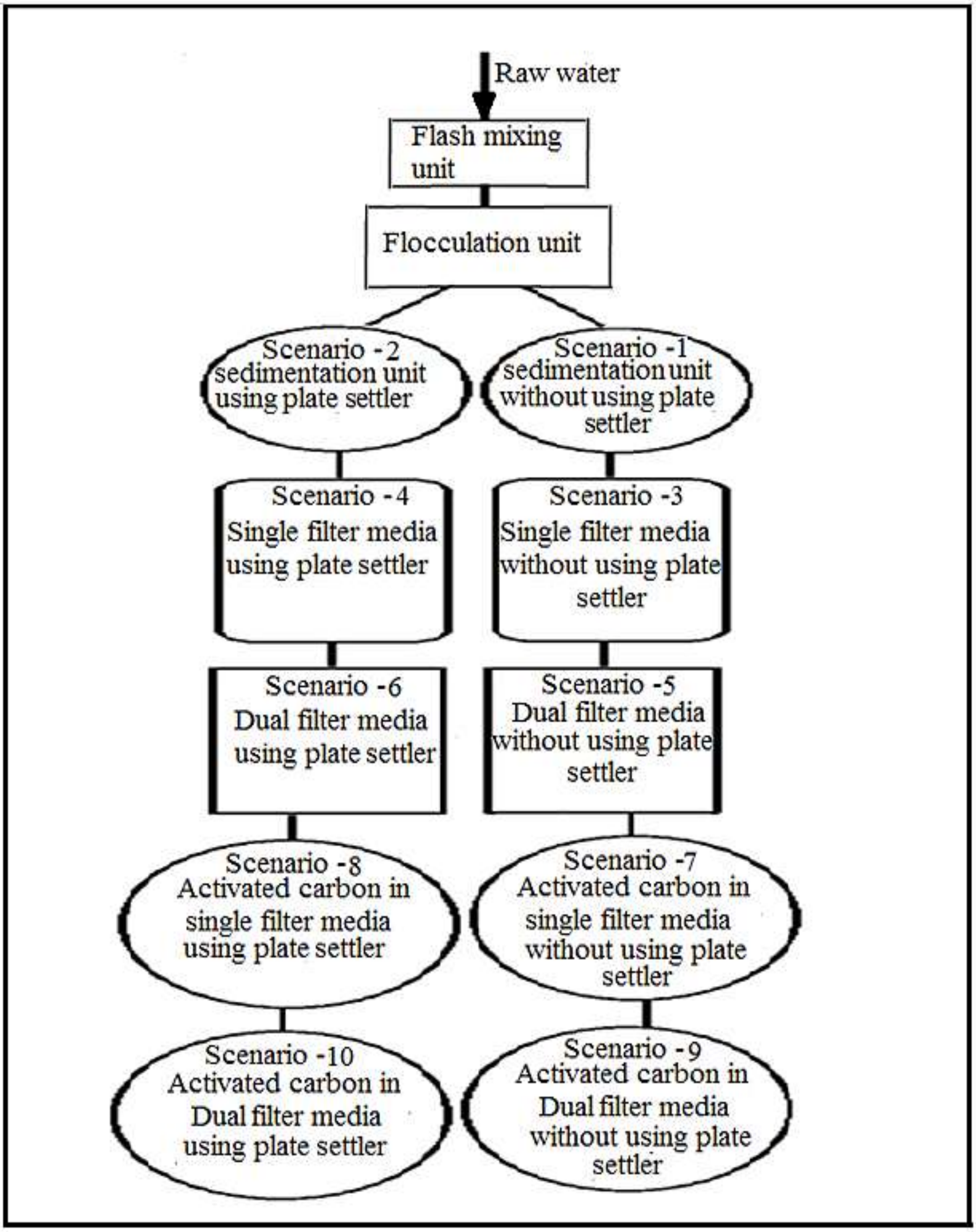


Figure (4-3): Stages of filtration processes in pilot plant

4.3.1 WQI in Pilot Plant at Conditions "Turb. = 20NTU,Q= 0.475m³/hr". Using River Soil

According to the section (3-10) listed in Chapter Three, the water quality index is calculated at turbidity of 20 NTU and flow rate of 0.475 m^3/hr.

Raw water has a WQI of 271.12, as seen in the Table (4-5).

Table (4-5): WQI at conditions "turb.= 20NTU,Q= 0.475m³/hr". using river soil.

Parameter	BIS Standard (Sn)	1/Sn	∑1/Sn	k=1/∑1/Sn	Wi=Wn= k/Sn	Ideal value (Vo)	Mean conc. Value (Vn)	Vn/Sn	Qn=(Vn/Sn)*100	Wn*Qn
Tur.	5	0.2000	0.3215	3.110	0.622	0	20.00	4.00	400	248.83
pH	8.5	0.1200	0.3215	3.110	0.366	7	7.90	0.60	60.00	21.96
EC	2000	0.0005	0.3215	3.110	0.002	0	1254.00	0.6270	62.70	0.10
TDS	1000	0.0010	0.3215	3.110	0.003	0	739.00	0.74	73.90	0.23
Sum		0.3215								271.12
WQI=271.12	The Raw water is unfit for consumption									

Likewise, the water quality index for turbidity 20NTU was computed for raw water at a flow rate of 0.475 m^3/hr. using the values in Tables (C-2) to (C-16).

Tables (4-6) to (4-9) show the total quality index calculations using soil at turbidities of 20, 30, 40, and 50 NTU, with four flow rates (0.475,0.712,0.95, and1.18) m^3/hr, for each turbidity.

Table 4-6 illustrates the overall quality index calculations utilizing soil and flow rate (0.475, 0.712, 0.95, 1.18) m^3/hr for all turbidities (20,30,40,50) NTU.

Table (4-6) :WQI at conditions "turb. =20NTU , Q = 0.475,0.712, 0.95 and 1.18 m^3/hr". using river soil.

Flow rate m^3/hr.	Grade of WQI at turbidity value = 20 NTU (Using river soil)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	271.12 Unfit	221.35 Unfit	195.16 Unfit	78.19 Very poor	59.69 Poor	57.23 Poor	54.69 Poor	58.68 Poor	50.64 Poor	51.83 Poor	44.49 Good
0.712	266.25 Unfit	189.18 Unfit	164.29 Unfit	34.87 Good	23.10 Exc-ellent	29.94 Good	18.17 Exc-ellent	33.08 Good	21.47 Exc-ellent	30.51 Good	19.24 Exc--ellent
0.95	271.12 Unfit	196.47 Unfit	164.16 Unfit	63.43 Poor	41.15 Good	34.44 Good	28.89 Good	45.06 Good	37.12 Good	27.01 Good	25.14 Good
1.18	271.11 Unfit	194.02 Unfit	157.99 Unfit	29.09 Good	25.35 Good	34.56 Good	24.11 Exc-ellent	25.23 Good	22.99 Exc-ellent	21.79 Exc-ellent	20.30 Exc-ellent

Table (4-7):WQI at conditions "turb.=30NTU, Q = 0.475, 0.712, 0.95 and 1.18 m^3/hr" using river soil.

Flow rate m^3/hr.	Grade of WQI at turbidity value = 30 NTU (Using river soil)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	393.1 Unfit	315.99 Unfit	269.33 Unfit	27.21 Good	25.10 Good	21.54 Exc.	20.67 Exc.	19.34 Exc.	18.22 Exc.	14.66 Exc.	13.79 Exc.
0.712	390.67 Unfit	267.5 Unfit	214.04 Unfit	32.39 Good	24.35 Exc.	23.59 Exc.	20.62 Exc.	21.29 Exc.	20.04 Exc.	18.92 Exc.	16.43 Exc.
0.95	393.11 Unfit	209.27 Unfit	186.68 Unfit	23.07 Exc.	18.76 Exc.	21.20 Exc.	17.39 Exc.	19.26 Exc.	18.01 Exc.	16.31 Exc.	14.69 Exc.
1.18	390.66 Unfit	205.33 Unfit	186.67 Unfit	27.45 Good	23.72 Exc.	19.99 Exc.	19.37 Exc.	20.16 Exc.	19.53 Exc.	17.30 Exc.	16.05 Exc.

Table (4-8):WQI at conditions "turb.=40NTU,Q = 0.475, 0.712, 0.95 and 1.18 m³/hr". using river soil.

Flow rate m³/hr.	Grade of WQI at turbidity value = 40 NTU (Using river soil)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	515.09 Unfit	370.77 Unfit	349.36 Unfit	36.12 Good	28.53 Good	24.23 Exc.	22.86 Exc.	29.95 Good	24.72 Exc.	19.42 Exc.	19.17 Exc.
0.712	522.39 Unfit	248.68 Unfit	206.47 Unfit	27.60 Good	24.62 Exc	22.25 Exc.	19.31 Exc.	19.24 Exc.	19.11 Exc.	17.12 Exc.	13.81 Exc.
0.95	522.40 Unfit	236.24 Unfit	176.12 Unfit	29.05 Good	25.0 Good	24.82 Exc.	18.70 Exc.	20.82 Exc.	20.32 Exc.	17.38 Exc.	15.26 Exc.
1.18	515.08 Unfit	213.99 Unfit	139.38 Unfit	21.24 Exc.	18.0 Exc.	19.99 Exc.	19.39 Exc.	17.55 Exc.	13.57 Exc.	19.24 Exc.	12.82 Exc.

Table (4-9):WQI at conditions "turb.= 50NTU ,Q = 0.475, 0.712, 0.95 and 1.18 m³/hr". using river soil.

Flow rate m³/hr.	Grade of WQI at turbidity value = 50 NTU(Using river soil)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	639.48 Unfit	316.0 Unfit	238.9 Unfit	26.9 Good	20.6 Exc.	24.66 Exc.	19.10 Exc.	22.96 Exc.	16.91 Exc.	16.84 Exc.	13.85 Exc.
0.712	641.92 Unfit	291.11 Unfit	228.91 Unfit	43.57 Good	34.79 Good	29.93 Good	19.98 Exc.	26.20 Good	17.04 Exc.	20.85 Exc.	18.23 Exc.
0.95	611.93 Unfit	278.68 Unfit	204.70 Unfit	54.82 Poor	33.42 Good	23.72 Exc.	15.68 Exc.	43.72 Good	26.30 Good	16.05 Exc.	11.37 Exc.
1.18	641.91 Unfit	216.46 Unfit	166.74 Unfit	20.97 Exc.	20.10 Exc.	22.09 Exc.	18.48 Exc.	16.29 Exc.	19.35 Exc.	14.92 Exc.	17.23 Exc.

4.3.2 Removal Efficiency in Pilot Plant at "Turb.=20 NTU. and four types of flow rate". Using River Soil.

The removal efficiency is determined using the water quality index for turbidity 20 NTU and four types of flow rate that were used in this study, as shown in table (4-10).

Table (4-10):The Removal Efficiency at "turb.=20NTU and four types of flow rate". Using river soil.

Flow rate m^3/hr.	% Removal Efficiency at turbidity value = 20 NTU and four types of flow rate (Using river soil)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	18.36	28.02	71.16	77.98	78.89	79.83	78.36	81.32	80.88	83.59
0.712	28.95	38.29	86.90	91.32	88.75	93.17	88.33	91.94	88.62	92.77
0.95	27.53	39.45	76.60	84.82	87.30	89.34	84.12	86.31	90.04	90.73
1.18	28.43	41.72	89.27	90.65	87.25	91.11	90.69	91.52	91.96	92.51

Similarly, the removal efficiency at turbidity 20 NTU was computed for WQI at a flow rate of 0.475 m^3/hr, using tables (4-7) to (4-9). The removal efficiency estimates for turbidities (30,40,50) NTU utilizing river soil are shown in Tables 4-11 to 4-13.

Table(4-11): Removal efficiency at "turb.=30 NTU and for four types of flow rate". Using river soil.

Flow rate m³/hr.	% Removal Efficiency at turbidity value = 30 NTU and four types of flow rate. (Using river soil)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	19.62	31.49	93.08	93.61	94.52	94.74	95 .08	95.37	96.27	96.49
0.712	31.53	45.21	91.71	93.77	93.96	94.72	94.55	94.87	95.16	95.79
0.95	46.77	52.51	94.13	95.23	94.61	95.58	95.10	95.42	95.85	96.26
1.18	47.44	52.22	92.97	93.39	94.88	95.04	94.84	95.0	95.57	95.89

Table(4-12): Removal efficiency at "turb.=40 NTU and for four types of flow rate". Using river soil.

Flow rate m³/hr	% Removal Efficiency at turbidity value = 40 NTU and four types of flow rate. (Using river soil)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	28.02	32.17	92.99	94.46	95.30	95.56	94.19	95.20	96.23	96.28
0.712	52.40	60.48	94.72	95.29	95.74	96.30	96.32	96.34	96.72	97.36
0.95	54.78	66.29	94.44	95.21	95.25	96.42	96.01	96.11	96.67	97.08
1.18	58.45	72.94	95.88	96.51	96.12	97.01	96.59	97.37	96.26	97.51

Table(4-13): Removal efficiency at "turb.=50 NTU and for four types of flow rate". Using river soil.

Flow rate m³/hr.	% Removal Efficiency at turbidity value = 50 NTU and four types of flow rate. (Using river soil)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	50.58	62.64	95.79	96.78	96.14	97.01	96.41	97.36	97.37	97.83
0.712	54.65	64.34	93.21	94.58	95.34	96.89	95.92	97.35	96.75	97.16
0.95	56.59	68.11	91.46	94.79	96.30	97.56	93.19	95.90	97.50	98.23
1.18	66.28	74.02	96.73	96.87	96.56	97.12	97.46	96.99	97.68	97.32

4.4 Optimal WQI and Removal Efficiency in Pilot Plant Using River Soil

The optimal values of WQI and removal efficiency for each turbidity level can be noted from Tables (4-14) to (4-17). It is rise in the value of the quality indicator, where this was seen that the quality indicator improved once the turbidity value was increased, also when the flow rate was increased, and from the first to the tenth scenario. The tenth scenario at the double filter with activated carbon yields the best quality index rating. Because the removal efficiency is related to the quality indicator, the removal efficiency has improved progressively when the turbidity value has increased from 20 NTU to 50 NTU and when the flow rate has increased from the first to the tenth scenario. This means high flow rate (using plate settler) and high turb. (using bentonite) leads high WQI , and high removal efficiency. (Schutte et al., 2007).

Table (4-14): The optimal WQI and Removal Efficincy at 20 NTU

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	18.17	10	0.475	93.17	6	0.712
2	19.24	10	0.712	92.77	10	0.712
3	20.30	10	1.18	92.51	10	1.18
4	21.47	8	0.712	91.96	9	1.18
5	21.79	9	1.18	91.94	8	0.712
6	22.99	8	1.18	91.52	8	1.18
7	23.1	4	0.712	91.32	4	0.712
8	24.11	6	1.18	91.11	6	1.18
9	25.14	10	0.712	90.73	10	0.95
10	25.23	7	1.18	90.69	7	1.18

Table (4-15): The optimal WQI and Removal Efficincy at 30 NTU

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	13.79	10	0.475	96.49	10	0.475
2	14.66	9	0.475	96.27	9	0.475
3	14.69	10	0.95	96.26	10	0.95
4	16.05	10	1.18	95.89	9	0.95
5	16.31	9	0.95	95.89	10	1.18
6	16.43	10	0.712	95.79	10	0.712
7	17.3	9	1.18	95.58	6	0.95
8	17.39	6	0.95	95.57	9	1.18
9	18.01	8	0.95	95.42	8	0.95
10	18.22	8	0.475	95.37	8	0.475

Table (4-16): The optimal WQI and Removal Efficincy at 40 NTU

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	12.82	10	1.18	97.51	10	1.18
2	13.57	8	1.18	97.37	8	1.18
3	13.81	10	0.712	97.36	10	0.712
4	15.26	10	0.95	97.08	10	0.95
5	15.39	6	1.18	97.01	6	1.18
6	17.12	9	0.712	96.72	9	0.712
7	17.38	9	0.95	96.67	9	0.95
8	17.55	7	1.18	96.59	7	1.18
9	18	4	1.18	96.51	4	1.18
10	18.7	6	0.95	96.42	6	0.95

Table (4-17): The optimal WQI and Removal Efficincy at 50 NTU

No.	Optimum WQI	Scenario	Flow rate m³/hr.	Optimum E%	Scenario	Flow rate m³/hr.
1	11.37	10	0.95	98.23	10	0.95
2	13.85	10	0.475	97.83	10	0.475
3	14.92	9	1.18	97.68	9	1.18
4	15.68	6	0.95	97.56	6	0.95
5	16.05	9	0.95	97.5	9	0.95
6	16.29	7	1.18	97.46	7	1.18
7	16.84	9	0.475	97.37	9	0.475
8	16.91	8	0.475	97.36	8	0.475
9	17.04	8	0.712	97.35	8	0.712
10	17.23	10	1.18	97.32	10	1.18

4.4.1 Best Results of WQI in Pilot Plant Using River Soil

Table (4-18) and Figure (4-4), Illustrate the selection of the best values results for water quality index in the pilot plant for four types of turbidities. The dgrees of water quality index were excelent between (11.37-15.26) ranged. At turbidity 50 NTU, flow rate 0.95m^3/hr, and activated carbone in dual filter media, the best and highest value of the water quality index was 11.37 (scenario 10). Dual filter media are effective in removing turbidity from effluents with turbidity less than 0.3 NTU, as well as removing organic matter precursors from disinfection products. However, with turbidity 40 NTU, flow rate 1.18 m^3/hr, and activated carbone in dual filter media, the better and second highest value of the water quality index was 12.82 (scenario 10). For water produced from the dual filter media with activated carbon, the percentage of water quality index decreased by about 8% from 11.37 to 12.36.

The reason for this is a decrease in the turbidity value from 50 NTU to 40 NTU, because when the turbidity value increases, it leads to better removal.

These findings are supported by Schutte's research (Schutte et al.,2007). at turbidity 40 NTU, flow rate 1.18m^3/hr, and activated carbon in single filter medium, the third level of water quality index was 13.57, down 16.21% (from 11.37 to 13.57) (scenario 8). The drop in the value of WQI is due to a fall in turbidity from 50 NTU to 40 NTU, as mentioned before, and the effectiveness of activated carbon removal in a single filter is smaller than that of an activated carbon removal in a dual filter. The lowest value of the water quality index was15.26 with turbidity 40 NTU, flow rate 0.95m^3/hr.,activated carbone in dual filter medium (scenario 10) with plate settler, indicating that raw water with low turbidity produces lower WQI. The results of the quality indicator in Table (4-18) show that the water produced in all scenarios is of excellent quality, particularly the water quality index at activated carbon filter, whether in dual filter or single filter, but the best was at dual filter media, as previously confirmed by (Hoboken,2003) that activated carbon has the best removal efficiency.

Furthermore, at high rates of flow, the majority of the values of the quality index of water generated from the sedimentation basin with plate settler were outstanding results (1, 2, 3, 8, and 10). The intent of the parallel plates settler is to increase the settling capacity of circular clarifier sedimentation basins by reducing the vertical distance a floc. particle must settle before agglomerating to form larger particles, also increase the surface area with decrease the surface overflow rate, increase capacity of conventional plant by 50-150 percent, reduce the settling area needed by one-fourth to one-sixth of what a conventional basin required (Schulz and Okun,1984).

Table (4-18): Best results of WQI in pilot plant for four types of turbidities. using river soil .

Item No.	Optimum WQI	Scenario	flow rate m^3/hr.in pilot plant	flow rate m^3/hr.in real plant	Turbidity value	velocity gradient (G) rapid mixing tank s^{-1}	velocity gradient (G) flocculation mixing tank s^{-1}
1	11.37	10	0.95	2100	50	800	30
2	12.82	10	1.18	2625	40	1000	30
3	13.57	8	1.18	2625	40	1000	30
4	13.79	10	0.475	1050	30	760	29
5	13.81	10	0.712	1575	40	760	30
6	13.85	9	0.475	1050	50	760	29
7	14.66	10	0.475	1050	30	760	29
8	14.69	10	0.95	2100	30	800	30
9	14.92	9	1.18	2625	50	1000	30
10	15.26	10	0.95	2100	40	800	30

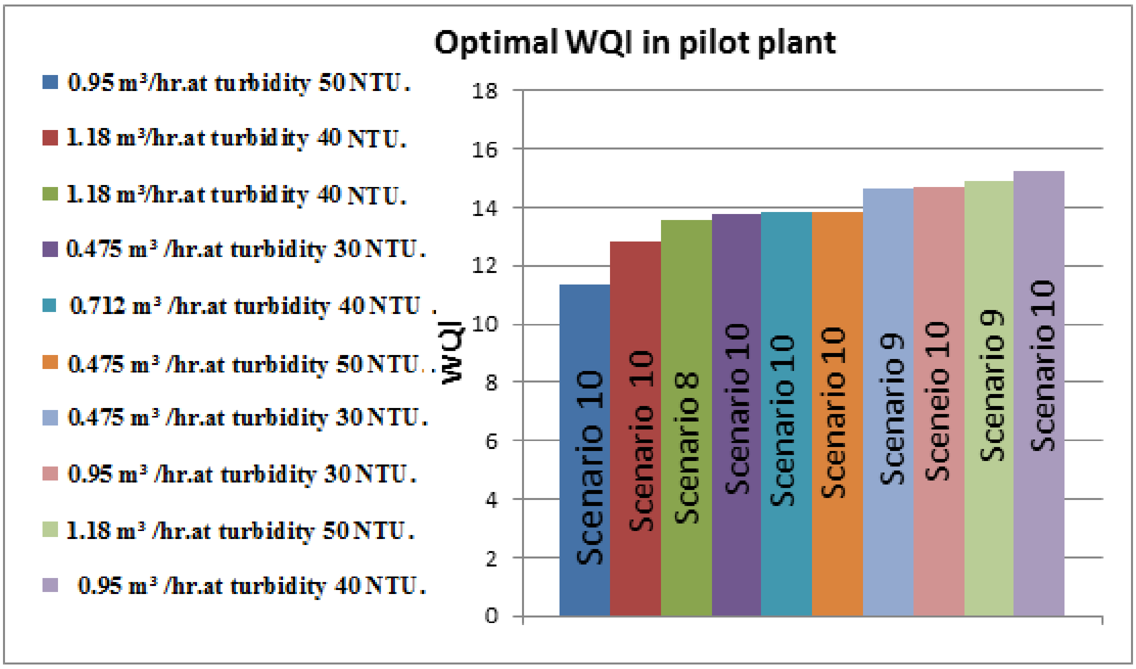


Figure (4-4):Best results of WQI in pilot plant for four types of turbidities.

4.4.2 Best Result of Removal Efficiency in the Pilot Plant Using River Soil.

Table (4-19) and Figure (4-5) show the optimal values for removal efficiency at the pilot plant for four different types of turbidities utilising river soil were chosen. The best removal efficiency using water quality index for the water produced in the experimental work using plate settler, at activated carbon in dual filter media (Scenario 10), at turbidity value of 50 NTU and flow rate 0.95m^3/hr, was 98.23 percent., which is what a previous study confirmed (Hoboken, 2003). The second best removal efficiency using the water quality index was 97.83 percent for the water generated in the experimental work by employing a plate settler, in dual filter medium using activated carbon (Scenario 10) with a turbidity value of 50 NTU and a flow rate of 0.475 m^3/hr.This efficiency reduces by around 0.4 percent, from 98.23 to 97.83.

The drop might be due to the high turbidity of raw water and huge disparities between WQI for raw water and WQI for scenario 10 at 0.475m^3/hr and 50 NTU turbidity.

The third best removal efficiency using the quality index for water generated in the experimental work without utilising a plate settler in dual filter media using activated carbon (Scenario 9) was 97.68 percent at turbidity of 50 NTU and flow rate of 1.18 m^3/hr. This efficiency reduction dropped by 1.56 percent from 98.23 to 97.68. The decline might be due to high turbidity raw water and substantial variances between WQI for raw water and WQI for scenario 9 at 1.18 m^3/hr and 50 NTU turbidity. The lack of a plate settler was another explanation for the treated water and activated carbon results in the double filter. These findings show that utilising activated carbon in dual filter media and a turbidity of 50 NTU resulted in a removal efficiency of more than 97 percent. Previous research has shown that employing activated carbon in dual filter media and a turbidity of 50 NTU results in the greatest removal efficiency (Hoboken et al., 2003). As in table (4-19), it can be seen that the optimum efficiency removal occurs at turbidity values between (40-50) NTU, whereas turbidity values between (20-30) NTU did not yield good removal results. This means that removing impurities in high turbidity water is easier than removing impurities in low turbidity water (Schutte et al., 2007). The removal efficiency when using the plate settler device was more than 97 percent at high flow rates (0.95 and 1.18m^3/hr.) as shown in item No. (1,4,5,9), because plates settler technology improves clarification performance by reducing detention time, resulting in an increased flow rate, because detention time is reduced. Researchers such as, (Hassan &Hassan,2011) demonstrated that the inclined plates settler may be employed to boost hydraulic capacity and improve water quality in a traditional settling tank. There are so many excellent values of activated carbon in single filter media using plate settler (scenario 8), indicating that the removal efficiency of this scenario is excellent.

This is what a previous study confirmed by (Hoboken ,2003), that this scenario has the best removal efficiency when using activated carbon.

When utilising traditional sand and anthracite filtration and secondary filtering with GAC (granular activated carbon),researchers (Bundy et al.,2007) obtained a removal efficiency of pharmaceutical compounds on the order of 95% and a turbidity reduction to less than 1 NTU.

The researcher (Thiel et al.,2006) demonstrated that sand:GAC filters are effective in removing turbidity from effluent with a turbidity of less than 0.3 NTU, as well as removing precursor organic matter from disinfection products. Even though single filter media produced a greater efficiency removal of turbidity, (Dyna et al., 2018) found that all units with activated carbon provided efficient removal that complied with the EPA and WHO's microbiological risk limit of 0.3 NTU.

Table (4-19): Best removal efficiency in pilot plant using river soil.

No.	Optimum E%	Scenario	Flow rate m^3/hr.in pilot plant	Flow rate m^3/hr.in real plant	Turbidity value	Velocity gradient (G)in rapid mixing tank s^1	Velocity gradient (G)in flocculation mixing tank s^1
1	98.23	10	0.95	2100	50	800	30
2	97.83	10	0.475	1050	50	760	29
3	97.68	9	1.18	2625	50	1000	30
4	97.56	6	0.95	2100	50	800	30
5	97.51	10	1.18	2625	40	1000	30
6	97.50	9	0.95	2100	50	800	30
7	97.46	7	1.18	2625	50	1000	30
8	97.37	9	0.475	1050	50	760	29
9	97.37	8	1.18	2625	40	1000	30
10	97.36	8	0.475	1050	50	760	29

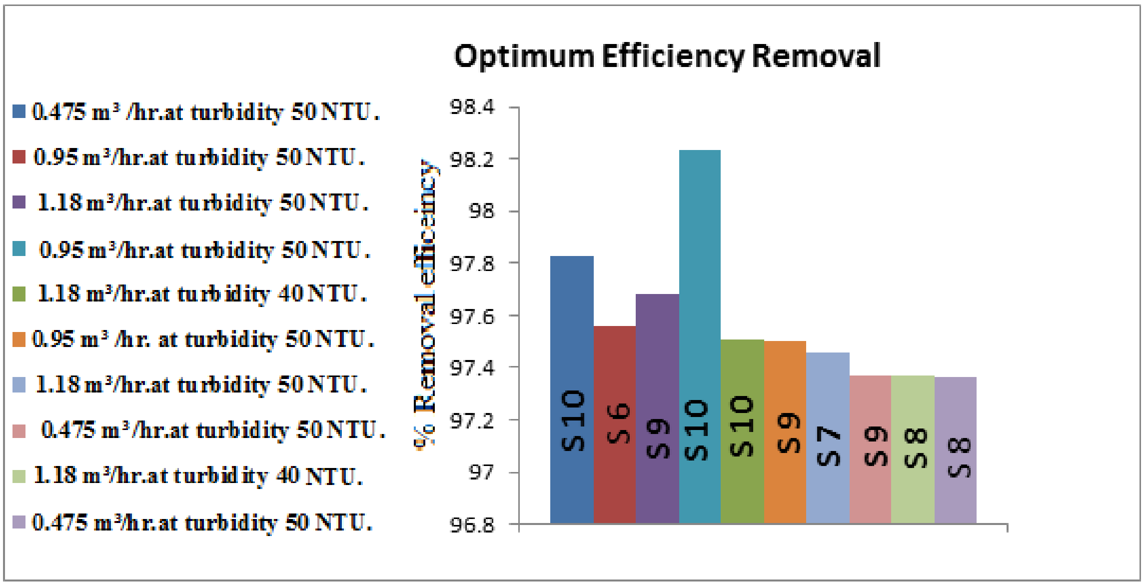


Figure (4-5):Best removal efficiency in the pilot plant for four types of turbidities.

4.5 WQI and Removal Efficiency in Pilot Plant Using Bentonite

Another four turbidity levels were chosen in this investigation, which was conducted using bentonite soil (20, 50, 120, and 200 NTU). For each turbidity value, four flow rates (0.475, 0.712, 0.95, and1.18) m^3/hr were employed, with eight distinct filtration units and sedimentation units (with plate settler, without plate settler) produced as scenarios, as indicated in Table (4-4) and Figure (4-3).

4.5.1 WQI in Pilot Plant Using Bentonite

The water quality index is computed for turbidity of 20 NTU and flow rate of 0.475 m^3/hr using the data in Table (C-17) and the four equations in Chapter Three. The Table (4-20) below shows the outcome of WQI =283.29 .

Table (4-20): The WQI of raw water at conditions "turb.=20NTU, Q = 0.475m^3/hr".

Parameter	BIS Standard (Sn)	1/Sn	∑1/Sn	k=1/∑1/Sn	Wi=Wn= k/Sn	Ideal value (Vo)	Mean conc. Value (Vn)	Vn/Sn	Qn=(Vn/Sn)* 100	Wn*Qn
Tur.	5	0.2000	0.3215	3.110	0.622	0	20.00	4.00	400	248.83
pH	8.5	0.12	0.3215	3.110	0.366	7	8.40	0.93	93.33	34.15
EC	2000	0.0005	0.3215	3.110	0.002	0	1167.00	0.5835	58.35	0.09
TDS	1000	0.0010	0.3215	3.110	0.003	0	670.00	0.67	67.00	0.21
Sum		0.3215			1.0					283.29
WQI=283.29	The Raw water is unfit for consumption				using bentonite					

Tables (4-21) to (4-24) show the total quality index calculations for turbidities (20,50,120,200) NTU using bentonite, with four flow rates (0.475, 0.712, 0.95, 1.18) m^3/hr for each turbidity, based on the data presented in tables (C-18) to (C-32).

Table (4-21) : WQI at conditions "turb.= 20 NTU , Q = 0.475, 0.712, 0.95 and 1.18 m^3 /hr".

Flow rate m^3/hr.	Grade of WQI at turbidity value = 20 NTU (Using bentonite)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	283.29 Unfit	182.60 Unfit	---	66.90 Poor	---	61.64 Poor	---	57.59 Poor	---	54.72 Poor	---
0.95	278.41 Unfit	188.88 Unfit	163.99 Unfit	61.39 Poor	42.85 Good	41.36 Good	36.63 Good	46.88 Good	34.56 Good	33.57 Good	27.59 Good

Table (4-22) : WQI at conditions "turb.= 50 NTU ,Q = 0.475, 0.712, 0.95 and 1.18 m³ /hr".

Flow rate m³/hr.	Grade of WQI at turbidity value = 50 NTU (Using bentonite)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	654.1 Unfit	253.53 Unfit	No Test	26.99 Good	No Test	30.86 Good	No Test	31.73 Good	No Test	28.99 Good	No Test
0.95	654.1 Unfit	228.64 Unfit	188.87 Unfit	38.07 Good	29.91 Good	30.91 Good	33.39 Good	33.37 Good	29.63 Good	30.25 Good	29.36 Good

Table (4-23) : WQI at conditions "turb.=120 NTU,Q=0.475, 0.712, 0.95 and 1.18 m³ /hr".

Flow rate m³/hr.	Grade of WQI at turbidity value = 120 NTU (Using bentonite)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.95	1525.03 Unfit	278.42 Unfit	213.76 Unfit	39.58 Good	31.66 Good	37.14 Good	27.23 Good	30.39 Good	25.41 Good	22.72 Excell.	21.72 Excell.

Table (4-24) : WQI at conditions "turb.=200 NTU,Q =0.475, 0.712, 0.95 and 1.18 m³ /hr".

Flow rate m³/hr.	Grade of WQI at turbidity value = 200 NTU (Using bentonite)										
	Raw water	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	2513.04 Unfit	179.12 Unfit	---	27.50 Good	---	22.27 Excellent	---	21.83 Excellent	---	18.25 Excellent	---
0.95	2515.50 Unfit	159.13 Unfit	149.22 Unfit	42.58 Good	36.03 Good	36.15 Good	29.48 Good	32.89 Good	25.79 Good	22.99 Excell.	20.11 Excell.

4.5.2 Removal Efficiency in Pilot Plant Using Bentonite

The removal efficiency was calculated using the water quality index for turbidities (20,50,120, and 200) NTU and four types of flow rate that were implemented in this study, from tables (4-21) to (4-24), and equation (4-1). Tables (4-25) to (4-28) show the removal efficiency at four types of turbidities, and four types of flow rate.

Table (4-25): Removal Efficiency at "turb.=20NTU ,and four types of flow rate".(using bentonite).

Flow rate $m^3/hr.$	% Removal Efficiency at turbidity value = 20 NTU and four types of flow rate (Using bentonite)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	35.68	---	76.44	---	78.29	---	79.71	---	80.73	---
0.95	32.16	41.10	77.95	84.61	85.14	86.84	83.16	87.59	87.94	90.09

Table (4-26): Removal Efficiency at "turb.= 50 NTU, and four types of flow rate". (using bentonite).

Flow rate $m^3/hr.$	% Removal Efficiency at turbidity value = 50 NTU and four types of flow rate (Using bentonite)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	61.24	---	95.87	---	95.28	---	95.15	---	95.57	---
0.95	65.05	71.13	94.18	95.43	95.27	94.90	94.90	95.47	95.38	95.51

Table (4-27): Removal Efficiency at" turb. =120 NTU ,and four types of flow rate". (using bentonite).

Flow rate m³/hr.	% Removal Efficiency at turbidity value = 120 NTU and four types of flow rate (Using bentonite)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.95	81.74	85.98	97.40	97.92	97.56	98.21	98.01	98.33	98.51	98.58

Table (4-28) : Removal Efficiency at turb. =200 NTU, and four types of flow rate".(using bentonite).

Flow rate m³/hr.	% Removal Efficiency at turbidity value = 200 NTU and four types of flow rate (Using bentonite)									
	Scenario.1	Sce.2	Sce.3	Sce.4	Sce.5	Sce.6	Sce.7	Sce.8	Sce.9	Sce.10
0.475	92.87	---	98.91	---	99.11	---	99.13	---	99.27	---
0.95	93.67	94.07	98.31	98.57	98.56	98.83	98.69	98.97	99.09	99.20

4.5.3 Optimal WQI and Removal Efficiency Using Bentonite

Tables (4-29) ,(4-30) ,(4-31) and (4-32) shows the optimal WQI and removal Efficiency for each turbidity value . Raise in the value of the quality indicator, with an improvement in the quality indicator noticed while raising the value of the turbidity, particularly when employing bentonite to increase the turbidity, as well as when the flow rate rises and from the first to the tenth scenario. The tenth scenario, utilizing a double filter with activated carbon, yields the best value for the quality index. Because of the relationship between removal efficiency and the quality indicator, removal efficiency has improved steadily when the turbidity value has increased from 20 to 50 NTU, at the same time ,the removal efficiency increased when the flow rate increased and from the first scenario to the tenth.

Table (4-29):The optimal WQI and Removal Efficincy at 20 NTU using bentonite.

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	27.59	10	0.95	90.09	10	0.95
2	33.57	9	0.95	87.94	9	0.95
3	34.56	8	0.95	87.59	8	0.95
4	36.63	6	0.95	86.84	6	0.95
5	41.36	5	0.95	85.14	5	0.95
6	42.85	4	0.95	84.61	4	0.95
7	46.88	7	0.95	83.16	7	0.95
8	54.72	9	0.475	80.73	9	0.475
9	57.59	7	0.475	79.71	7	0.475
10	61.39	3	0.95	78.29	5	0.475

Table (4-30):The optimal WQI and Removal Efficincy at 50 NTU using bentonite

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	26.99	3	0.475	95.87	3	0.475
2	28.99	9	0.475	95.57	9	0.475
3	29.36	10	0.95	95.51	10	0.95
4	29.63	8	0.95	95.47	8	0.95
5	29.91	4	0.95	95.43	4	0.95
6	30.25	9	0.95	95.38	9	0.95
7	30.86	5	0.475	95.28	5	0.475
8	30.91	5	0.95	95.27	5	0.95
9	31.73	7	0.475	95.15	7	0.475
10	33.33	7	0.95	94.90	7	0.95

Table (4-31): The optimal WQI and Removal Efficincy at 120 NTU using bentonite

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	21.72	10	0.95	98.58	10	0.95
2	22.72	9	0.95	98.51	9	0.95
3	25.41	8	0.95	98.33	8	0.95
4	27.23	6	0.95	98.21	6	0.95
5	30.39	7	0.95	98.01	7	0.95
6	31.66	4	0.95	97.92	4	0.95
7	37.14	5	0.95	97.56	5	0.95
8	39.58	3	0.95	97.40	3	0.95
9	213.76	2	0.95	85.98	2	0.95
10	278.42	1	0.95	81.74	1	0.95

Table (4-32):The optimal WQI and Removal Efficincy at 200 NTU .(using bentonite).

Item No.	Optimum WQI	Optimum Scenario	Flow rate(m^3/hr.)	Optimum E%	Optimum Scenario	Flow rate(m^3/hr.)
1	18.25	9	0.475	99.27	9	0.475
2	20.11	10	0.95	99.20	10	0.95
3	21.83	7	0.475	99.13	7	0.475
4	22.27	5	0.475	99.11	5	0.475
5	22.99	9	0.95	99.09	9	0.95
6	25.79	8	0.95	98.97	8	0.95
7	27.5	3	0.475	98.91	3	0.475
8	29.48	6	0.95	98.83	6	0.95
9	32.89	7	0.95	98.69	7	0.95
10	36.03	6	0.95	98.57	4	0.95

4.6 Best Results of WQI in Pilot Plant Using Bentonite

Table (4-33) and Figure (4-6) reveal the water quality index values in the pilot plant were excellent, ranging from 18.25 to 27.23 for four types of turbidities using bentonite. At turbidity 200 NTU, flow rate 0.475 m^3/hr, and activated carbone in dual filter media without employing a plate setter, the best and highest value of the water quality index was 18.25 (scenario 9).

According to a (Thiel,2006) dual filter media are effective in removing turbidity from effluent with turbidity less than 0.3 NTU, as well as removing precursor organic matter from disinfection products (Thiel et al.,2017). Likewise, with turbidity 200 NTU, flow rate 0.95 m^3/hr, and activated carbon in dual filter media, the better and second highest value of the water quality index was 20.11 (scenario 10).

Table (4-33): Best values for water quality index in the pilot plant using bentonite.

Item No.	Optimum WQI	Scenario	Flow rate m^3/hr.in pilot plant	Flow rate m^3/hr.in real plant	Turbidity value	Velocity gradient (G)in rapid mixing tank s^1	Velocity gradient (G)in flocculation mixing tank s^1
1	18.25	9	0.475	1050	200	760	29
2	20.11	10	0.95	2100	200	800	30
3	21.72	10	0.95	2100	120	800	30
4	21.83	7	0.475	1050	200	760	29
5	22.27	5	0.475	1050	200	760	29
6	22.72	9	0.95	2100	120	800	30
7	22.99	9	0.95	2100	200	800	30
8	25.79	8	0.95	2100	200	800	30
9	26.99	3	0.475	1050	50	760	29
10	27.23	6	0.95	2100	120	800	30

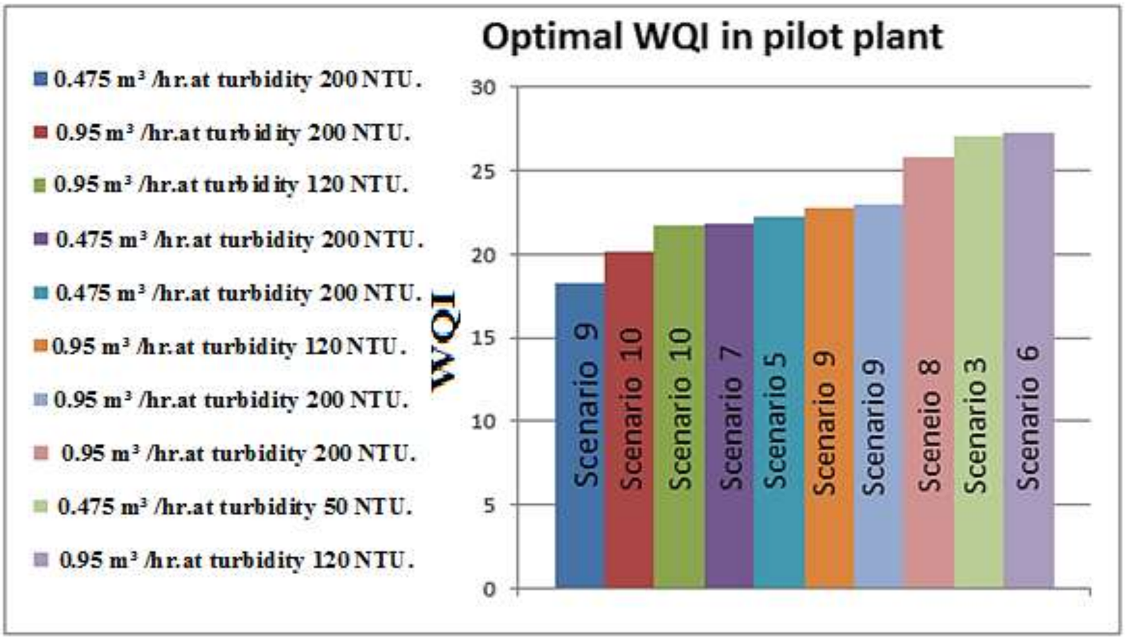


Figure (4- 6): Best WQI in pilot plant

For water produced from dual filter media with activated carbon (scenario 10), the percentage of water quality index decreased by about 9% from 18.25 to 20.11. This decline may be due to increases in pH, which increased from 7.6 at turbidity 200 NTU and flow rate 0.475 m^3/hr to 7.7 at turbidity 200 NTU and flow rate 0.95m^3/hr, while the remaining parameters have minor differences. At turbidity 120 NTU, flow rate 0.95m^3/hr., in dual filter media utilising activated carbon, the third level of water quality index was 21.72, down roughly 16 percent (from 18.25 to 21.72). (scenario 10). The drop in WQI value is due to a decrease in turbidity from 200 NTU to 120 NTU, since as the turbidity value of raw water increases, it leads to greater removal, as earlier indicated (Schutte et al., 2007).

The lowest value of the water quality index was 27.23 at turbidity 120 NTU, flow rate 0.95m^3/hr., in dual filter media (scenario 6) with plate settler, indicating that the water quality index generated by dual filter,without activated carbon is worse than that produced by dual filter with activated carbon. This relates to the effectiveness of activated carbon, which has a unique adsorption property due to its high porosity and vast surface area, allowing it to extract and retain many of the pollutants found in water, as demonstrated by (Steel et al.,1984).

The findings of the quality indicator in Table (4-33) show that the water generated in all scenarios is of very high quality, particularly the water quality index of the activated carbon filter, regardless of whether or not a plate settler is used. Previous research has shown that the optimum removal efficiency when utilising activated carbon (Hoboken,2003). Furthermore, the majority of the findings of the outstanding quality index of water created when high-turbid values (120, 200), and a varied flow rate, while the naked NTU (20,50) has no good WQI.This suggests that removing pollutants in high turbidity water is easier than removing impurities in low turbidity water (Schutte et al., 2007).

4.7 Best Results of Removal Efficiency in Pilot Plant Using Bentonite

Table (4-34) and Figure (4-7) show the optimal values for removal efficiency at the pilot plant for four different types of turbidities using bentonite were chosen. At a turbidity of 200 NTU and a flow rate of 0.475 m^3/hr, the best removal efficiency utilising water quality index for the water generated in the experimental work without employing plate settler, in dual filter medium with activated carbon (Scenario 9), was 99.27 percent. Because when there is a lot of turbidity, the removal efficiency is greater (Schutte et al., 2007).

The second greatest removal efficiency using the water quality index was 99.20 percent for the water generated in the experimental work utilising a plate settler, in dual filter medium employing activated carbon (Scenario 10) with a turbidity value of 200 NTU and a flow rate of 0.95 m^3/hr. The explanation for the loss in efficiency is that the value of pH(7.7) at turbidity 200 NTU, and flow rate 0.95m^3/hr., in scenario (10) is more than pH (7.6) at turbidity 200NTU, and flow rate 0.475m^3/hr in scenario (10) is greater than pH (7.6) at turbidity 200NTU, and flow rate 0.475m^3/hr. The other reason is that the water produced from dual filter media with activated carbon employing plate settler (scenario 10) was not tested at turbidity 200 NTU and flow rate 0.475 m^3/hr.

Table (4-34): Best results of removal efficiency in pilot plant Using bentonite.

No.	Optimum E%	Scenario	Flow rate m^3/hr. in pilot plant	Flow rate m^3/hr.in real plant	Turbidity value	Velocity gradient (s^1) (G) in rapid mixing tank	Velocity gradient (s^1) (G)in flocculation mixing tank
1	99.27	9	0.475	1050	200	760	29
2	99.20	10	0.95	2100	200	800	30
3	99.13	7	0.475	1050	200	760	29
4	99.11	5	0.475	1050	200	760	29
5	99.09	9	0.95	2100	200	800	30
6	98.97	8	0.95	2100	200	800	30
7	98.91	3	0.475	1050	200	760	29
8	98.83	6	0.95	2100	200	800	30
9	98.69	7	0.95	2100	200	800	30
10	98.58	10	0.95	2100	120	800	30

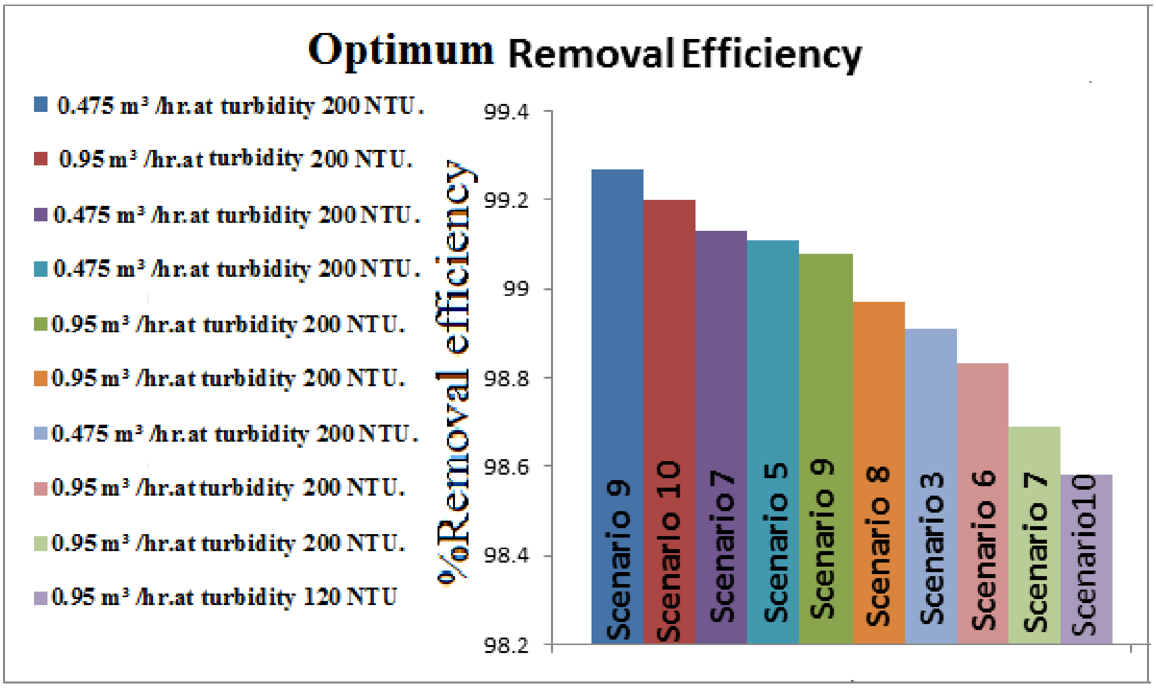


Figure (4-7): Best removal efficiency in pilot plant using bentonit.

As a result, the removal efficiency in scenario (9) at 200 NTU turbidity and 0.475 m^3/hr. was larger than the efficiency in scenario (10) at 200 NTU turbidity and 0.95m^3/hr .The greatest removal efficiency using the quality index for water generated in the experimental work without utilizing a plate settler in single filter medium using activated carbon (Scenario 7) was 99.13 percent at turbidity of 200 NTU and flow rate of 0.475 m^3/hr . This efficiency elimination lowered by 0.14 percent from 99.27 to 99.13, which is a little difference. The reduction may be due to the fact that the turbidity value and pH in scenario (7) are (0.36,7.7) NTU, respectively, while the turbidity value and pH in scenario (9) are (0.27,7.6) NTU, respectively. This means that when turbidity and pH values are large, the WQI rises, and at the same flow rate, the turbidity value for raw water rises, reducing the efficiency of removal. The results show that when using activated carbon in dual filter media at turbidity 200 NTU, the efficiency removal was greater than 99 percent.

Dual filtration is a treatment technology that consists of two stages: first, the clarified water passes through a granular shape with high porosity and large surface area that allows it to remove and retain many of the impurities present in water; and second, the clarified water passes through a granular shape with high porosity and large surface area that allows it following the reduction of turbidity in the first step, a second stage of filtration is used to polish the water (Sandobal- Paz et al. ,2015).

The GAC was utilized as a water purification filter because of its features, which allow it to boost effective removal by eliminating turbidity and dissolved organic contaminants (Gupta, and Ali, 2013). In the table (4-34), it can be seen that the optimum removal efficiency was achieved at a turbidity of 200 NTU (except for sequence 10, where the turbidity was 120 NTU), whereas turbidities of 20 and 50 NTU did not yield good removal results, implying that impurities are removed more easily in high turbidity water than in low turbidity water (Schutte et al., 2007). It appears that the plate settler device's work was insufficient at the low flow rate (0.475 m^3/hr.), because the plate settler requires a high flow rate to increase the quaintly of water treated, lowering the detention period while the basin volume is fixed.

4.8 WQI and Removal Efficiency in Single and Dual Filter Media Without Using Activated Carbon

Table (4-35) shows the WQI and Efficiency of four types of turbidity removal in single and dual filter media without the use of activated carbon.

Table (4-35) :WQI and removal efficiency in single and dual filter media (without using activated carbon).

Flow rate (m^3/hr.)		Turbidity value (NTU)											
		20		30		40		50		120		200	
0.475	Sce.	WQI	%E	WQI	%E	WQI	%E	WQI	%E	WQI	%E	WQI	%E
	S3	78.19	71.16	27.21	93.08	36.12	92.99	26.9	95.79	-	-	27.5	98.9
	S4	59.69	77.98	25.1	93.61	28.53	94.46	20.6	96.78	-	-	-	-
	S5	57.23	78.89	21.54	94.52	24.23	95.3	24.66	96.14	-	-	22.27	99.1
	S6	54.69	79.83	20.67	94.74	22.86	95.56	19.1	97.01	-	-	-	-
0.712	S3	34.78	86.9	32.39	91.71	27.6	94.72	43.57	93.21	-	-	-	-
	S4	23.1	91.32	24.35	93.77	24.62	95.29	34.79	9458	-	-	-	-
	S5	29.94	88.75	23.59	93.96	22.25	95.74	29.93	95.34	-	-	-	-
	S6	18.17	93.17	20.62	94.72	19.31	96.3	19.98	96.89	-	-	-	-
0.95	S3	63.43	76.6	23.07	94.13	29.05	94.44	54.82	91.46	39.56	97.4	42.58	98.3
	S4	41.15	84.82	18.76	95.23	25	95.21	33.42	94.79	31.66	97.92	36.03	98.57
	S5	34.44	87.3	21.2	94.61	24.82	95.25	23.72	96.3	37.14	97.56	36.15	98.56
	S6	28.89	89.34	17.39	95.58	18.7	96.42	15.68	97.56	27.23	98.21	29.48	98.83
1.18	S3	29.09	89.27	27.45	92.97	21.24	95.88	20.97	96.73	-	-	-	-
	S4	25.35	90.65	23.72	93.39	18	96.51	20.1	96.56	-	-	-	-
	S5	34.56	87.25	19.99	94.88	19.99	96.12	22.09	97.12	-	-	-	-
	S6	24.11	91.11	19.37	95.04	19.39	97.01	18.48	97.46	-	-	-	-

The best WQI value was 15.68 (good grade) with flow rate 0.95m^3/hr and turbidity 50 NTU in dual filter utilizing plate settler (S6).The second best WQI value was at flow rate 0.95m^3/hr and at turbidity 30 NTU in dual filter ,using plate settler (S6) is 17.39 (it's an excellent grade).

The best efficiency removal value was 99.11percent at a flow rate of 0.475m^3/hr. and a turbidity of 200 NTU in a dual filter (without utilizing a plate settler) (S5). At a flow rate of 0.95 m^3 /hr and a turbidity of 200 NTU in a single filter (without utilizing a plate settler) (S3) , the second greatest efficiency removal value was 98.91percent .

4.9 WQI and Removal Efficiency in Single and Dual Filter Media Using Activated Carbon.

Table (4-36) shows the WQI and Efficiency of four types of turbidity removal in single and dual filter media without employing activated carbon. The optimum WQI value was 11.37 at flow rate 0.95m^3/hr and turbidity 50 NTU activated carbon in dual filter (S10) using plate settler. (It received the highest WQI score in all tests). The second best WQI value was 13.85 (good grade) with flow rate 0.475m^3/hr and turbidity 50 NTU in activated carbon in dual filter, utilizing plate settler (S10).

At a flow rate of 0.475m^3/hr and a turbidity of 200 NTU in activated carbon in a dual filter (without utilizing a plate settler) (S9),the best efficiency removal value was 99.27percent .

The second greatest removal efficiency value was 99.20 percent in a dual filter (using plate settler) (S10) at a flow rate of 0.95m^3/hr and a turbidity of 200 NTU.

Table (4-36) : WQI and removal efficiency in single and dual filter media (using activated carbon).

Flow rate (m^3/hr.)		Turbidity value (NTU)											
		20		30		40		50		120		200	
0.475	Sce.	WQI	%E	WQI	%E	WQI	%E	WQI	%E	WQI	%E	WQI	%E
	S7	58.68	78.36	19.34	95.08	29.95	94.19	22.96	96.41	-	-	21.83	99.13
	S8	50.64	81.32	18.22	95.37	24.72	95.2	16.91	97.36	-	-	-	-
	S9	51.83	80.88	14.66	96.27	19.42	96.23	16.84	97.37	-	-	18.25	99.27
	S10	44.49	83.59	13.79	96.49	19.17	96.28	13.85	97.83	-	-	-	-
0.712	S7	33.08	88.33	21.29	94.55	19.42	96.32	26.20	95.92	-	-	-	-
	S8	21.47	91.94	20.04	94.87	19.11	96.34	17.04	97.35	-	-	-	-
	S9	30.51	88.62	18.92	95.16	17.12	96.72	20.85	96.75	-	-	-	-
	S10	19.42	92.77	16.43	95.79	13.81	97.36	18.23	97.16	-	-	-	-
0.95	S7	45.06	84.12	19.26	95.10	20.82	96.01	43.72	93.19	30.39	98.01	32.89	98.69
	S8	37.12	86.31	18.01	95.42	20.32	96.11	26.30	95.90	25.41	98.33	25.79	98.97
	S9	27.01	90.04	16.31	95.85	17.38	96.67	16.05	97.50	22.72	98.51	22.99	99.09
	S10	25.14	90.73	14.69	96.26	15.26	97.08	11.37	98.23	21.72	98.58	20.11	99.20
1.18	S7	24.23	90.69	20.16	94.84	17.55	96.59	16.29	97.46	-	-	-	-
	S8	22.99	91.52	19.53	95	13.57	97.37	19.35	96.99	-	-	-	-
	S9	21.79	91.96	17.3	95.57	19.24	96.26	19.92	97.68	-	-	-	-
	S10	20.30	92.51	16.05	95.89	12.82	97.51	17.23	97.33	-	-	-	-

4.10 Advantage of Activated Carbon

The optimum value of the water quality index was 11.37 at a flow rate of 0.95m^3/hr and at turbidity of 50 NTU when activated carbon was used in the dual filter (S10), whereas the value without using activated carbon was 15.68 at flow rate 0.95m^3/hr.

It can be seen that the WQI value when activated carbon is used (11.37) is higher than the WQI value when no activated carbon is used (15.68 without using activated carbon).

The best removal efficiency was 99.27 utilizing activated carbon in a dual filter (S9) at a flow rate of 0.475 m^3/hr. and at turbidity of 200 NTU, whereas the removal efficiency without activated carbon was 99.11percent .

This refers to the effectiveness of activated carbon, which has a unique adsorption property due to its high porosity and large surface area, allowing it to remove and retain many of the impurities present in water, as demonstrated by(Steel,1984), as well as sand : GAC filters are effective at removing turbidity, generating effluent with turbidity less than 0.3NTU, and effective at removing precursor organic matter from disinfect

4.11Enhancement of WQI and Removal Efficiency in Pilot Plant Using Bentonite at Turbidity (20, 50, 200) NTU

For the optimum value of the water quality index and removal efficiency when using bentonite, refer to Tables (4-29),(4-30),(4-33) and (4-34) , and Figures (4-8) and (4-9) in a row, and Tables (4-6),(4-10),(4-18) and(4-19) for the best value of the quality indicator and removal efficiency when using river soil.

The WQI and removal efficiency of river soil were 18.17, 93.17 percent at 20 NTU and 11.37, 98.23 percent at 50 NTU, respectively. while utilizing bentonite, the WQI and removal efficiency were 27.59 and 90.09 percent at

20 NTU and 26.99 and 95.87 percent at 50 NTU, respectively. Figures (4-8) and (4-9) demonstrate that using bentonite with low turbidity has no influence on removal efficiency, whereas the WQI and removal efficiency at turbidity (200 NTU) were (18.25, 99.27 percent) respectively. This indicates that adding bentonite to low turbidity raw water to increase turbidity has a positive impact as a coagulant, and because bentonite is a clay material, it seeks to surround the suspended particles and besiege them by gravity.

As a result, raw water with a high turbidity may require less coagulant for a proper coagulation, whereas raw water with a low turbidity may require more coagulant. As a result, adding turbidity to reasonably clear water might be beneficial at times. This is often done with bentonite (Peavy et al.,1985)

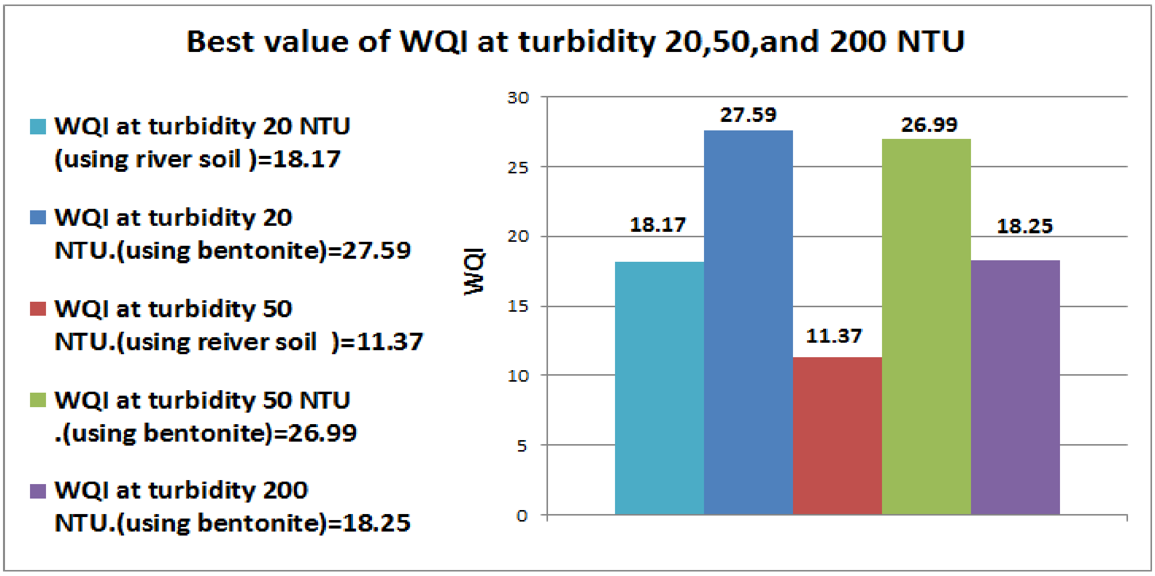


Figure (4 -8): WQI at turbidity 20, 50 and 200 NTU using river soil and bentonite

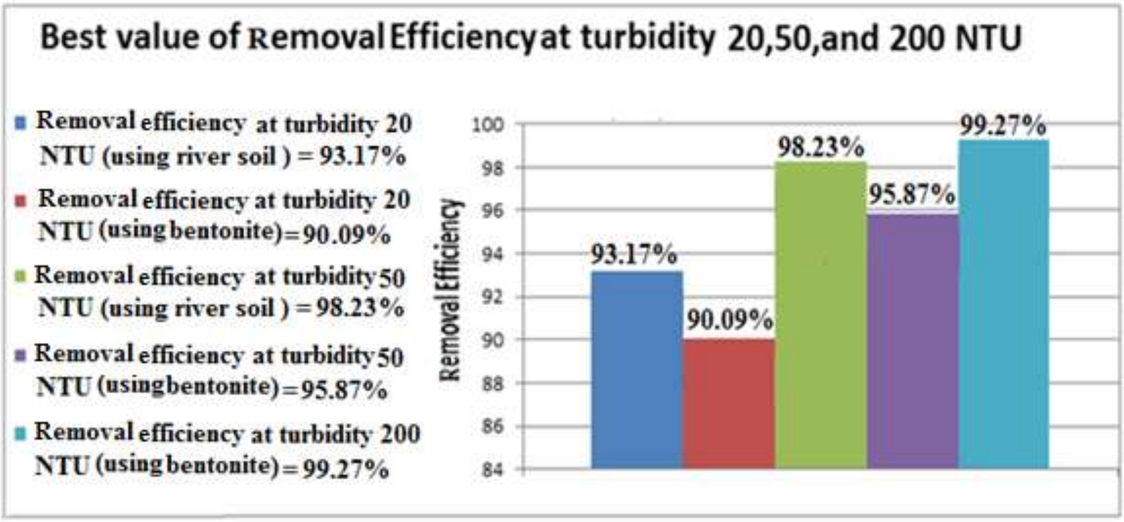


Figure (4 -9): Removal efficiency using river soil and bentonite

4.12 Comparison Between River Soil and Bentonite Using

Table (4-37) demonstrates the optimum WQI , and removal efficiency for 20 and 50 NTU when utilizing river soil and bentonite, respectively. In the case of river soil, the water quality index and removal efficiency at turbidity 20 and 50 are better than when bentonite is used. This is because bentonite was employed in the preparation of low turbidity (20, 50) NTU, therefore the turbidity removal technique is only efficient when the turbidity is high. Increasing the bentonite dose lowers the pH value more than using aluminum sulphate alone, making the pH optimal for coagulation and flocculation, as well as improving coagulation and flocculation operations to acquire excellent quality water and speed in the sedimentation of created flocs. When a bentonite dose of up to 0.8 g/L was added to raw water, the turbidity steadily rose (Rohana Abdullah et al., 2013). Visual examination also revealed that when the amount of bentonite in the supernatant rose, the supernatant grew clearer. The process of bentonite addition will reduce turbidity by reducing electrostatic forces and forming more flocs.

Table (4-37): WQI and removal efficiency using the river soil and bentonite

Turbidity value	Optimum status type	Using river soil	Flow rate (m^3/hr.)	Using bentonite	Flow rate (m^3/hr.)
20NTU	Scenario	6	0.712	10	0.95
	WQI	18.17		27.59	
	E%	93.17		90.09	
50NTU	Scenario	10	0.95	3	0.475
	WQI	11.37		26.99	
	E%	98.23		95.87	

4.13 WQI and Removal Efficiency in Sedimentation Basin

In the case of river soil and bentonite, Table (4-38) demonstrates the best WQI and removal efficiency in the sedimentation basin with or without the plates settler. When utilizing river soil, the best WQI was 189.18 without employing a plate settler at a flow rate of 0.712 m^3/hr. and a turbidity of 20 NTU, whereas in a sedimentation tank, the best WQI was 139.38 with a plate settler at a flow rate of 1.18 m^3/hr., and a turbidity of 40 NTU.

This indicates that the optimum WQI is achieved when employing a plate settler and a high flow rate (1.18m^3/hr.), because plate settler technology enhances clarifying performance by lowering detention time, resulting in a higher flow rate.

Furthermore, the inclined plates settler was shown to boost hydraulic capacity and improve water quality for existing traditional settling tanks (Hassan &Hassan,2011).

Whilst using river soil, the best removal efficiency was 66.28 percent without using a plate settler at a flow rate of 1.18 m^3/hr. and a turbidity of 50 NTU, while the best removal efficiency in a sedimentation tank was 74.02

percent using a plate settler at a flow rate of 1.18 m^3/hr. and a turbidity of 50 NTU.

This indicates that the highest removal efficiency occurs when employing a plate settler and a high flow rate (1.18 m^3/hr.), since plate settler technology increases clarifying performance by lowering detention time, resulting in a higher flow rate, because detention time is reduced.

Furthermore, the inclined plates settler was shown to increase hydraulic capacity and improve water quality for existing traditional settling tanks (Hassan &Hassan,2011).

Table (4-38): WQI and removal efficiency in the sedimentation basin

Item No	Turbidity value (NTU)	Flow rate (m^3/hr.)	WQI in sedimentation basin (without using plate settler) (S1)	WQI in sedimentation basin(using plate settler) (S2)	%Removal Efficiency (without using plate settler) (S1)	%Removal Efficiency (using plate settler) (S2)
1	20 With river soil	0.475	221.35	165.16	18.36	28.02
		0.712	189.18	164.29	28.95	38.29
		0.95	196.47	164.16	27.53	39.45
		1.18	194.02	157.99	28.46	41.72
2	30 With river soil	0.475	315.99	269.33	19.62	31.49
		0.712	267.5	214.04	31.53	45.21
		0.95	209.27	186.68	46.77	52.51
		1.18	205.33	186.67	47.44	52.22
3	40 With river soil	0.475	370.77	349.36	28.02	32.17
		0.712	248.68	206.47	52.40	60.48
		0.95	236.24	176.12	54.78	66.29
		1.18	213.99	139.38	58.45	72.94
4	50 With river soil	0.475	316	238.9	50.58	62.64
		0.712	291.11	228.91	54.65	64.34
		0.95	278.68	204.7	56.59	68.11
		1.18	216.46	166.74	66.28	74.02

5	20 with bentonite	0.475	182.6	---	35.68	---
		0.95	278.41	188.88	32.16	41.10
6	50 with bentonite	0.475	253.53	---	61.24	---
		0.95	228.64	188.78	65.05	71.13
7	120 with bentonite	0.95	278.42	213.76	81.74	85.93
8	200 with bentonite	0.475	179.12	---	92.87	---
		0.95	159.13	149.22	93.67	94.07

The best WQI in the sedimentation tank when using bentonite was 149.22 with plate settler at flow rate 0.95m^3/hr. and turbidity value 200 NTU, whereas the best WQI in the sedimentation tank when using bentonite was 159.13 without plate settler at flow rate 0.95 m^3/hr. and turbidity value 200 NTU. The best WQI was obtained while utilizing a plate settler with a high flow rate (0.95 m^3/hr) . According to (Gurjar. A .,et al., 2017), utilizing a plate settler module in a sedimentation basin improves particle settling efficiency. When compared to traditional treatment, the tube settler system achieves a turbidity reduction effectiveness of 70-80%. The best removal efficiency while using bentonite was 93.67percent without using a plate settler at a flow rate of 0.95 m^3/hr and a turbidity value of 200 NTU, while the best efficiency removal in a sedimentation tank was 94.07percent when using a plate settler at a flow rate of 0.95 m^3/hr. and a turbidity value of 200 NTU. This implies that the turbidity removal effectiveness of a tube settler unit is higher than the turbidity removal efficiency of a traditional sedimentation tank. Increase particle settling efficiency by using a tube settler module in a sedimentation basin. When compared to traditional treatment, the tube settler system has a turbidity reduction efficacy of 70-80% (Gurjar. A .,et al., 2017).

When using bentonite to prepare the turbidity, the flow rate was 0.95m^3/hr with the plate settler, and the turbidity value was 200 NTU, the best removal efficiency was (94.07percent). This suggests that bentonite is a useful material to employ in water treatment, as demonstrated by (M'hamed Ahari et al.,2019), who found that adding 20 mg/L of bentonite to water can remove 96.72 percent of turbidity and 60 percent of oxidizing article.

Increasing the bentonite dose lowers the pH value more than using aluminum sulphate alone, making the pH optimal for coagulation and flocculation, as well as improving coagulation and flocculation operations to acquire excellent quality water and speed in the sedimentation of created flocs. When a bentonite dose of up to 0.8 g/L was added to raw water, the turbidity steadily rose (Rohana Abdullah et al., 2013). Visual examination also revealed that when the amount of bentonite in the supernatant rose, the supernatant grew clearer. The process of bentonite addition will reduce turbidity by reducing electrostatic forces and forming more flocs.

Furthermore, prior research have shown that by adding 2g of bentonite, 95 percent of Fe2 removal may be achieved. As a result, the dose of bentonite must be raised above 1.2 g/L in order to increase Fe2 elimination, furthermore, using a plate settler inside a sedimentation tank with a high flow rate (0.95 m^3/hr.) and a high turbidity (200 NTU) will improve sedimentation, as demonstrated by researchers (Gurjar. A .,et al., 2017) who found that using a tube settler module in a sedimentation basin increased particle settling efficiency. When compared to traditional treatment, the tube settler device has a turbidity reduction effectiveness of 70-80 percent.

4.14 Comparison of WQI and Removal Efficiency in Real and Pilot Plant

As shown in Tables (4- 2) and (4-3), the best WQI and removal efficiency in real water treatment plants in 2017 were 23.03 and 91.26, respectively, at turbidity of 20 NTU and flow rate of 0.475 m^3/hr , while utilizing river soil. WQI and efficiency removal in the pilot plant at the same turbidity and flow rate but with bentonite were 66.90 and 76.44, respectively, as shown in Tables (4-21) and(4-25). The reason why the quality indicator and efficiency removal at the real station is better than the quality indicator and efficiency removal in the pilot plant at single filter medium without plate settler while employing bentonite and the same turbidity and flow rate (scenario 3). This occurred because the use of bentonite in the preparation of low turbidity has limited efficacy in the removal efficiency or the quality indicator. Using bentonite turbidity of 200 NTU and a flow rate of 0.475 m^3/hr., the WQI and efficiency removal at (scenario 3) were 27.30 and 98.91 percent, respectively. The best results were attained by adding bentonite to low-turbid raw water to enhance turbidity, since the supernatant grew clearer as the amount of bentonite increased. The process of bentonite addition will reduce turbidity by reducing electrostatic forces and forming more flocs. This bentonite mechanism has been proven (Rohana Abdullah et al., 2013). This indicates that raw water with low turbidity should not be treated directly. To enhance turbidity, bentonite should be added, and then the water should be passed through processing units. There are several reasons for using bentonite in treatment:

- Raw water with low turbidity requires a higher coagulant dose, such as aluminum sulphate, in order to be cleared; nevertheless, too much alum might induce Alzheimer's disease.
- When added to water, bentonite is a natural ingredient that has no detrimental effects.
- A number of treatment plants in the area do not add alum to low-turbid water; instead, water is passed directly from sedimentation basin sediment to filters without treatment, and this process puts pressure on the filters, which are the only ones that reduce turbidities, requiring them to be washed frequently.

4.15 Removal Efficiency of Physio-Chemical Parameters in Pilot Plant

From table (C-33) to (C-54), the lowest and maximum values of each parameter's removal efficiency at each flow rate and turbidity value have been discovered, as shown in Table (4-39) and figure (4-10).

Table (4-39): Removal efficiency for physical and chemical parameters in pilot plant.

Parameter	Maximum Removal Efficiency			Minimum Removal Efficiency		
	Ratio (%)	Turbidity	Flow rate	Ratio(%)	Turbidity	Flow rate
EC	9.6	30 soil	0.475	1.41	50 soil	1.18
	12.25	20 bentonite	0.475	3.05	50bentonite	0.475
TDS	9.61	30 soil	0.475	1.41	50 soil	1.18
	24.15	50 bentonite	0.95	6.02	50bentonite	0.475
pH	7.5	40 soil	0.712	1.26	20 soil	0.475
	7.23	120 bentonite	0.95	2.41	50bentonite	0.475

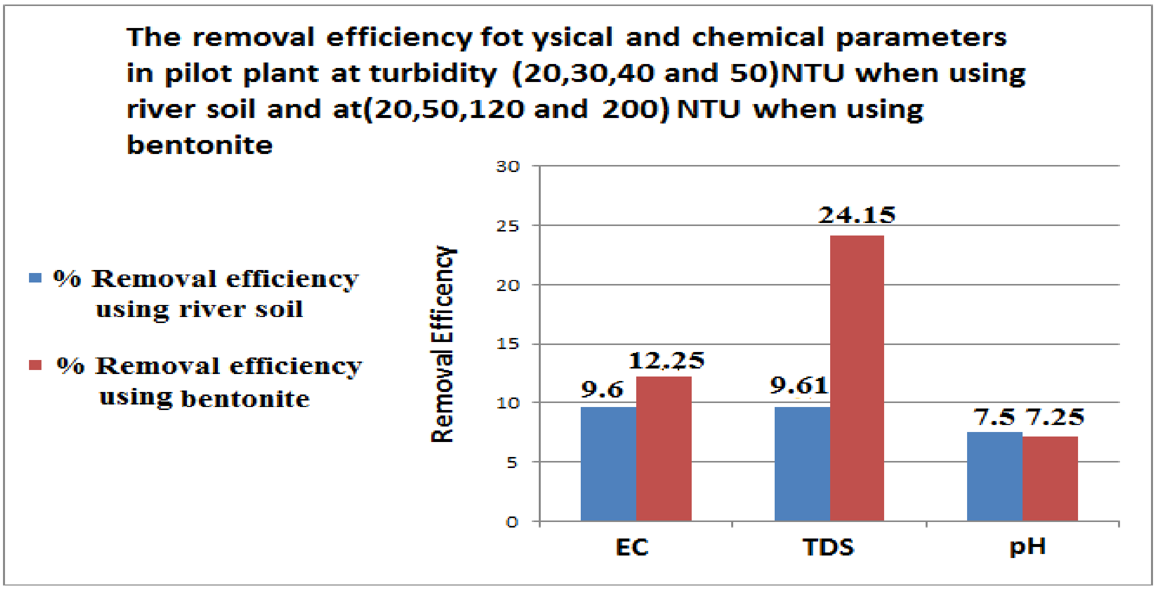


Figure (4-10): Removal efficiency for physical and chemical parameters in pilot plant.

The removal effectiveness of each parameter was determined using table (C-33) (using river dirt) and table (C-49) (using bentonite) in a pilot plant with a flow rate of 0.475 m^3/hr. and turbidity of 20 NTU, as shown in Table (C-33) and Figures (C-49), (4-11).

Table (4-40): Removal efficiency at "Q= 0.475 m³/hr, and turb.= 20 NTU in pilot plant" .

Parameter	Removal % (using river soil)	Removal % (using bentonite)
turbidity	90.1	88.9
EC	2.87	12.25
TDS	3.2	12.23
pH	1.26	3.57

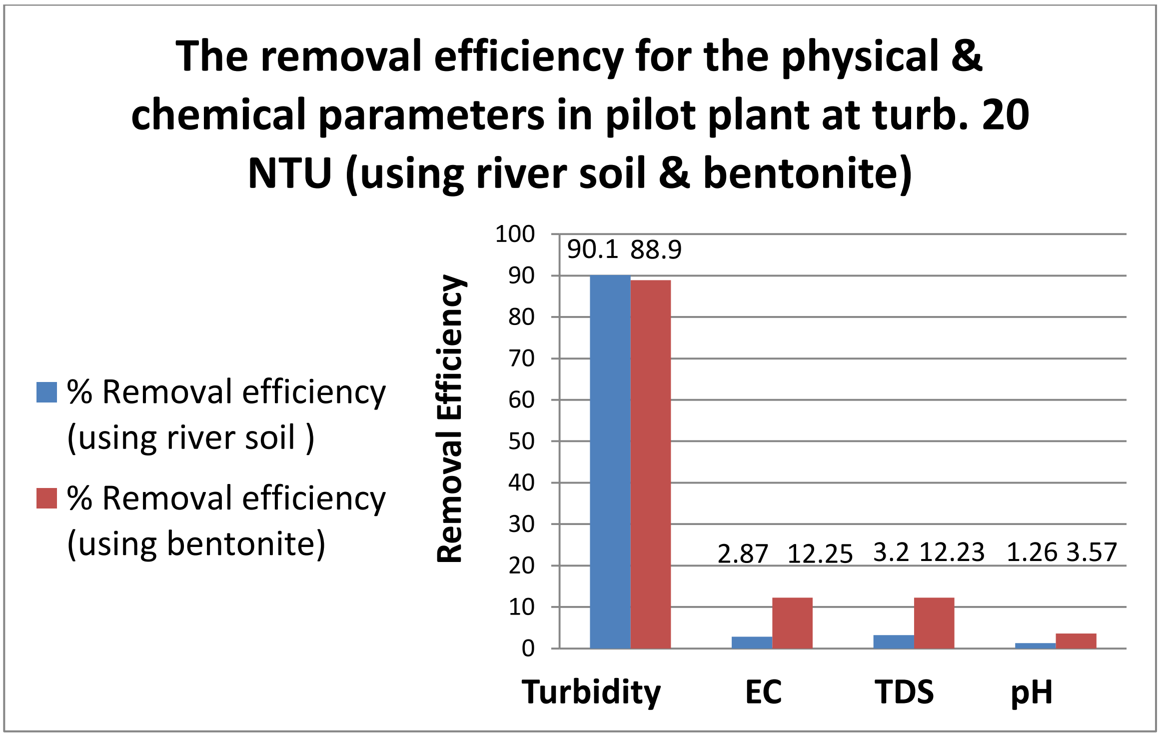


Figure (4-11): Physio-Chemical parameters removal in pilot plant at 20NTU

4.16 Removal Efficiency of Physio-Chemical Parameters in Real Plant.

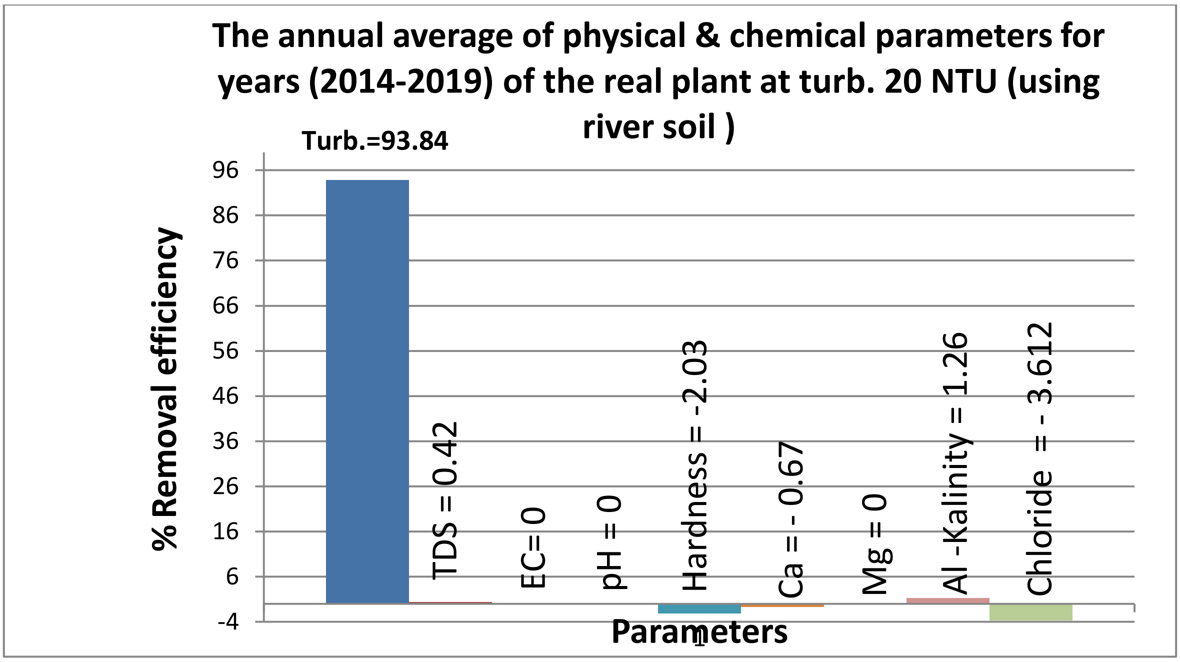
The yearly rate of parameters from 2014 to 2019 is depicted in Table (4-41) and Figure (4-12). It was taken and computed from tables (C-65) to (C-70), which show the qualitative properties of water treated at the real plant over a period of time (2014-2019). The qualitative parameters of raw water have not altered, indicating that the plant units are ineffective in removing pollutants. Except for the elimination of turbidity, the Kerbala treatment facility comprised a sequence of water traffic with no change in water quality:

1.The removal effectiveness of chloride exhibits a higher rise in chloride content in treated water than raw water ,reaching a ratio of -3.612 percent ,this change is due to adding chlorine in treated water for disinfection.

Table (4-41): Annual average of physio-chemical parameters in real plant at T=20 NTU

Year	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
Average -2014	20.1	1.3	536.7	548.5	1147.2	1150.9	7.8	7.7
Average-2015	18	1.3	663	634	1390	1397	7.43	7.51
Average-2016	19.0	1.0	534.7	552.9	1122.5	1127.4	7.5	7.5
Average-2017	17	0.83	561	556	1073	1060.5	No Test	No Test
Average-2018	21.7	1.5	510.8	502.7	1059.8	1059.1	No Test	No Test
Average-2019	21.0	1.5	605.8	603.9	1064.2	1061.8	No Test	No Test
Total Average	19.5	1.2	568.7	566.3	1142.8	1142.8	7.6	7.6

Year	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
Average-2014	382.3	385.9	93.1	91.9	36.4	37.1	132.7	127.8	105.1	109.6
Average-2015	419	423	121	125	28	26	120	122	127	135
Average-2016	428.7	429.4	107.3	108.6	39.0	39.3	102.4	103.6	127.1	132.1
Average-2017	430	475	111.5	108.5	37	36.5	111	109	128.5	133
Average-2018	403.6	401.8	102.1	97.9	36.4	38.1	90.5	88.0	133.4	133.8
Average-2019	362.3	360.1	89.0	87.7	33.8	33.8	109.0	106.4	109.9	113.6
Total Average	404.3	412.5	104.0	103.3	35.1	35.1	110.9	109.5	121.8	126.2



Figure(4-12): Annual average of physio-chemical parameters in real plant at T=20 NTU.

2. The removal efficiency of alkalinity demonstrates a reduction in alkalinity content in treated water, reaching a ratio of (1.26 percent).

3. There is no difference in Magnesium concentration between treated and untreated water, implying that the removal efficiency is zero.

4. Calcium removal efficiency demonstrates a reduction in calcium content in treated water, reaching a ratio of (0.67 percent).

5. Hardness removal efficiency demonstrates that treated water has a higher concentration of hardness than raw water, reaching a ratio of (-2.03 percent).

6. There is no difference in PH content between treated and untreated water, implying that the removal effectiveness is zero.

7. There is no difference in EC concentration between treated and untreated water, implying that the removal efficiency is zero.

8. The TDS removal efficiency demonstrates a reduction in TDS content in treated water, reaching a ratio of (0.42 percent).

9. The turbidity removal efficiency demonstrates in treated water, reaching a ratio of (93.84 percent).

Chapter Five

CONCLSIONS AND RECOMMENDATIONS

CHAPTER FIVE

CONCLSIONS AND RECOMMENDATIONS

5.1 Conclusions

The following conclusions are the summary of the findings from this study for the development of plant units:

1. In experimental work, the optimal dose of the alum in coagulation process obtained, were (3,6,7,and 8) mg /l, when using four types of turbidity (20,30,40 and 50 NTU) respectively (using river soil) , while the optimal dose of the alum in coagulation process obtained ,were (3,8,35,and 55) mg /l, when using four types of turbidity value (20,50,120,200 NTU) .
2. It's found that the optimum flocculation value utilizing for four types of turbidity (20, 30, 40, and 50 NTU) using river soil, and at four types of turbidity (20,50,120 and 200 NTU) using bentonite, were 29,30,30, and 30 s^{-1}, respectively .
3. It was found that the removal efficiency when using river soil was 98.23 percent by using the water quality index of the water generated in the experimental operation in dual filter media and activated carbon filter at a turbidity of 50 NTU ,at flow rate of 0.95 m^3/hr , using plate settler.
4. Also it was found that the removal efficiency when using bentonite was 99.27 percent by using the quality index of the water generated in the experimental operation in dual filter media and activated carbon filter at a turbidity of 200 and a flow rate of 0.475m^3/hr, without using plate settler.
5. Increasing water turbidity has a positive impact in removal efficiency .
6. In comparison to the removal effectiveness in the sedimentation basin of the water treatment plant in Kerbala and for the years 2014-2019, which did not surpass 33% efficiency rate, the sedimentation process may remove up to 90% of suspended particles.

7. In this study , the bentonite has been used as turbidity material added to water in order to increase the turbidity, In the other side when increase of the dose of bentonite it will decreases the value of pH more than the use of alum alone, which makes the pH of water optimal for coagulation–flocculation, but also to enhance the coagulation and flocculation processes to get good quality effluent and the rapid sedimentation of the flocs formed (M’hamed Aharia et al., 2019).

8. The usage of activated carbon in this study as a filter ,enhanced the removal efficiency .

9.The usage of dual filter media in this study, which included a sandy layer and an anthracite layer, enhanced removal efficiency.

10. It's found that the maximum flow rate gives the best results in removal efficiency especially at using plate settler.

5.2Recommendations

1.With the absence of disinfection, this research covers all water treatment facilities; thus, future studies must include cleansing and improvements in this entity to complete the construction of the water treatment plant.
2.This work should be extended by an economic feasibility analysis and compared to parallel returns to determine the efficacy of these development methods, as well as a cost resulting from energy consumption, adding materials used in the processing process, maintenance, and operation.

3. Suggest extensive research to consider direct removal for some pollutants , minerals and enhancing an electrical conductivity and the total dissolved solid that can be content , such as dangerous organic compounds, viruses, arsenic, sulphate and developing diseases such as giardia bacteria**.**

4.Part of the improvement performance in this research of the water station is an increasing the flow rate, which requires additional pumps in the low pumping station.

6.Its recommended to use sand with anthracite layers as a dual media filter in existing water treatment plant in kerbala governorate .

7. when using a bentonite as a coagulant material or to increase the raw water turbidity, prefer to test it to ensure that the safety of its material without causing any side effect.

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APPENDIX (A)

Calculations of Hydraulic Scaling of the Pilot Plant

APPENDIX (A)

CLCULATIONS OF HYDRAULIC SCALING OF PILOT PLANT

A.1 General Equations of Hydraulic Scaling of the pilot plant

In this appendix, the scaling factors were obtained by means of the dynamic similarity equations for three types of scale ratios that were taken, and therefore the best scale ratio is chosen from these three ratios for the purpose of calculating the design and dimensions of the model as shown below.

1- When λ = 4, $\lambda_r = \frac{1}{50}$, ratio used $\frac{\lambda r}{\lambda h} = \frac{1}{4}$, $\therefore \lambda_r = \frac{1}{50}$, $\frac{\lambda_r}{\lambda_h} = \frac{1}{4}$
(1/50)/λh = 1/4 , λh= 1/(12.5).

2-When λ = 4, $\lambda r = \frac{1}{40}$, ratio used $\frac{\lambda r}{\lambda h} = \frac{1}{4}$, $\therefore \lambda r = \frac{1}{40}$, $\frac{\lambda r}{\lambda h} = \frac{1}{4}$
(1/40)/λh = 1/4 , λh= 1/(10)

3-When λ = 4, $\lambda r = \frac{1}{30}$, ratio used $\frac{\lambda r}{\lambda h} = \frac{1}{4}$, $\therefore \lambda r = \frac{1}{30}$, $\frac{\lambda r}{\lambda h} = \frac{1}{4}$
(1/30)/λh = 1/4 , λh= 1/7.5 , Table (A-1) shows three types of scale ratios when λ=4 and Table (A-2) shows different values of λ_Q , Q_m, SOR, V_r and Froude number at scale factor (λ)=4.

Table (A-1) : Different values of λ_h and λ_r at scale factor (λ)=4

Pilot plant unit	λ = 4					
	$\lambda_r = \frac{1}{50}$ Radius (m)	$\lambda_h = \frac{1}{12.5}$ Water depth (m)	$\lambda_r = \frac{1}{40}$ Radius (m)	$\lambda_{rh} = \frac{1}{10}$ Water depth (m)	$\lambda_r = \frac{1}{30}$ Radius (m)	$\lambda_r = \frac{1}{7.5}$ Water depth (m)
rapid mixing	(5.5÷ 2)/50 = 0.055	(34.83-30.1)/ 12.5 = 0.3784	(5.5÷2) /40 = 0.068	(34.83-30.1) /10 = 0.473	(5.5÷2) /30 = 0.091	(34.83-30.1) /7.5 = 0.630
flocculation	6.2 / 50 = 0.124	(34.01-30.1)/ 12.5 = 0.3128	6.2/40 = 0.155	(34.01-30.1) /10 = 0.391	6.2/30 = 0.206	(34.01-30.1) /7.5 = 0.521
clarifier (settling basin)	11.5 / 50 = 0.23	(34.29-30.1) /12.5 = 0.3352	11.5/40 = 0.287	(34.29-30.1) /10 = 0.419	11.5/30 = 0.383	(34.29-30.1) /7.5 = 0.558

Table (A-2) : Different values of λ_Q , Q_m, SOR, V_r and Froude number at scale factor (λ)=4

parameter	$\lambda = 4$, Q_p=1050 m³/hr.		
	$\lambda_r = \frac{1}{50}, \lambda_h = \frac{1}{12.5}$	$\lambda_r = \frac{1}{40}, \lambda_h = \frac{1}{10}$	$\lambda_r = \frac{1}{30}, \lambda_h = \frac{1}{7.5}$
$\lambda_Q = \lambda_r * (\lambda_h)^{1.5}$	λQ = $4.525*10^{-4}$	λQ = $7.905*10^{-4}$	λQ = $1.62*10^{-3}$
$\lambda_Q = Q_m / Q_p$, (m^3/hr.) $Q_m = \lambda_Q * Q_p$	0.475m³/hr.	0.829 m³/hr.	1.701m³/hr.
SOR = Q_m / A_m , (m/hr.)	1.24 m/hr.	1.386 m/hr.	1.599 m/hr.
Rotational velocity (m/min) = Q/(2πrh) = Q/(2*3.14*0.38*0.3352)	0.00989	0.01105	0.0127
Re = Q/(2πRυ)	61.9	86.44	133
Fr = $Q^2/(4\pi^2R^2H^3g)$ = Q^2 / 2.10196	$8.28*10^{-9}$	$8.26*10^{-9}$	$8.2975*10^{-9}$

1-When λ = 5 , λr = $\frac{1}{50}$, ratio used $\frac{\lambda r}{\lambda h} = \frac{1}{5}$, ∴λr= $\frac{1}{50}$, $\frac{\lambda r}{\lambda h} = \frac{1}{5}$

(1/50)/λh = 1/5 , λh = 1/10

2-When λ = 5 , λr = $\frac{1}{40}$, ratio used $\frac{\lambda r}{\lambda h} = \frac{1}{5}$, ∴λr= $\frac{1}{40}$, $\frac{\lambda r}{\lambda h} = \frac{1}{5}$

(1/40)/λh = 1/5 , λh= 1/8.

3-When λ = 5 , λr = $\frac{1}{30}$, ratio used $\frac{\lambda r}{\lambda h} = \frac{1}{5}$, ∴λr= $\frac{1}{30}$, $\frac{\lambda r}{\lambda h} = \frac{1}{5}$

(1/30)/λh = 1/5 , λh= 1/6. Table (A-3) shows three types of scale ratios when λ=5 and Table (A-4) shows different values of λ_Q , Q_m, SOR, V_r and Froude number at scale factor (λ)=5.

Table (A-3): Different values of λ_h and λ_r at scale factor (λ)=5

Pilot plant unit	λ = 5					
	$\lambda_r = \frac{1}{50}$ Radius (m)	$\lambda_h = \frac{1}{10}$ Water depth (m)	$\lambda_r = \frac{1}{40}$ Radius (m)	$\lambda_h = \frac{1}{8}$ Water depth (m)	$\lambda_r = \frac{1}{30}$ Radius (m)	$\lambda_h = \frac{1}{6}$ Water depth (m)
rapid mixing	(5.5÷ 2) /50 = 0.055	(34.83-30.1) / 10 = 0.473	(5.5÷2) /40 = 0.068	(34.83-30.1) /8 = 0.591	(5.5÷2) /30 = 0.091	(34.83-30.1) /6 = 0.788
flocculation	6.2 / 50 = 0.124	(34.01-30.1) / 10 = 0.391	6.2/40 = 0.155	(34.01-30.1) /8 = 0.448	6.2/30 = 0.206	(34.01-30.1) /6 = 0.651
clarifier (settling basin)	11.5 / 50 = 0.23	(34.29-30.1) /10 = 0.419	11.5/40 = 0.287	(34.29-30.1) /8 = 0.523	11.5/30 = 0.383	(34.29-30.1) /6 = 0.968

Table (A-4) : Different values of λ_Q , Q_m,SOR, V_r and Froude number at scale factor(λ) =5.0.

parameter	λ = 5 , Q_p=1050 m³/hr.		
	$\lambda_r=\frac{1}{50}, \lambda_h=\frac{1}{10}$	$\lambda_r=\frac{1}{40}, \lambda_h=\frac{1}{8}$	$\lambda_r=\frac{1}{30}, \lambda_h=\frac{1}{6}$
$\lambda_Q=\lambda_r * (\lambda_h)^{1.5}$	$\lambda Q = 6.324*10^{-4}$	$\lambda Q = 1.104*10^{-3}$	$\lambda Q = 2.268*10^{-3}$
$\lambda_Q = Q_m / Q_p$, (m³/hr.) $Q_m=\lambda_Q * Q_p$	0.664m³/hr.	1.159m³/hr.	2.3814m³/hr.
SOR = Q_m / A_m , (m/hr.)	1.74 m/hr.	1.937 m/hr.	2.239 m/hr.
Rotational velocity (m/min) = Q/(2πrh)=Q/(2*3.14*0.38*0.3352)	0.01106	0.01236	0.0134
Re = Q/(2πRυ)	86.55	120.85	175.1
Fr=Q²/(4π²R²H³g) = Q²/ 2.10196	$8.28*10^{-9}$	$8.272*10^{-9}$	$8.306*10^{-9}$

In order to select the best result for scale ratio from Table (A- 2),the lowest value of rotational velocity , and lowest value of SOR , it can be good with value of ($9.88*10^{-3}$) ,(1.24) respectively because the lower rotational velocity and lower SOR it means good settling velocity, and according to (Rouse,1945) length scale (λ_l)= (λ_r = (1/50), and ratio used [($\lambda_r/\lambda h$) = 1/4].

∴ The best value of scale factor (λ = 4) , $\lambda r = \frac{1}{50}$, $\lambda h = \frac{1}{12.5}$.

Also Table (A-5) shows the values of area, depth of water of each unit of treatment that should be use it in model .

Table (A-5): Area values and water depth for each unit of processing units that must be used in the form.

Unit mode	At λ = 4, $\lambda r = \frac{1}{50}$, $\lambda h = \frac{1}{12.5}$			
Rapid mixing	D= 24.5cm 24.5/100=0.245m Area of flash mixer = $\frac{(0.245)^2*\pi}{4}$ = 0.047m² Water depth = 0.21m			
Flocculation basin	R(outer) = 7/50 = 0.14	R(inner) = 0.8/50 = 0.016	R_{outer}-R_{inner} = 0.14-0.016 = 0.124	Area = $(0.14)^2 - (0.016)^2*\pi$ = 0.06 m^2 water depth = 0.3128m
Sedimentation basin	R(outer) = 19/50 = 0.38	R(inner) = 7.5/50 = 0.15	R_{outer}–R_{inner} = 0.38 - 0.15 = 0.23	Area = $(0.38)^2$- $(0.15)^2*\pi$ = 0.382 m² water depth = 0.3352m

APPENDIX (B)

Design models of Water Treatment Plant Units In Kerbala

APPENDIX (B)
DESIGN MODELS OF WATER TREATMENT PLANT UNITS IN KERBALA

With reference to Chapter three, Figure (3-3) shows the schematic diagram of the model that was designed for each unit of the water treatment plant in Karbala, which is the rapid mixing tank, the Clarifloccultor which is used for suspended matter removal ,this tank is divided into a central flocculation zone and an outer settling zone. The third facility of treatment plant is filtration unit. The design account for each component is presented in the following paragraphs:

B.1 Design of Rapid Mixing Tank

B.1.1 Design Condition (Pilot Plant):

A. Max. flow =1.18 m^3/hr.

B. Agitator details:

I-Impeller installation: vertical.

II-Impeller type: angle blade.

III-Blade angle to horizontal 28º.

IV-No. of blades per arm:2.

V-No. of arms per agitator (stage): 3

$$\therefore \frac{paddle\ area\ rotating\ in\ the\ crosse\ section\ in\ prototype}{(crosse\ section\ area\ of\ tank\ )_p} * 100 = \frac{1.9*0.1*3}{23.74} *100$$

= 2.4% <(15-25)% ok

Ratio of area of blades to Crosse section area of tank area(in model) =

$$\frac{area\ of\ blddes}{crosse\ section\ area\ of\ tank} *100$$

$$\frac{2.4}{100} = \frac{area\ of\ blddes}{0.04711}$$

$area\ of\ blddes$= $1.13*10^{-3}$m²

$length\ ratio\ of\ bladdes\ (in\ prototype) to\ diameter\ of\ tank\ = \frac{1.9}{5.5}*100$

= 34%

$$\frac{34}{100} = \frac{length\ of\ bladdes(\ in\ model)}{diameter\ of\ tank\ (in\ model)} = \frac{length\ of\ bladdes(\ in\ model)}{0.245}$$

$length\ of\ bladdes(\ in\ model)$= 0.34 * 0.245

= 0.0833m.

Width of blades(in model) = $\frac{area\ of\ blade(in\ model)}{length\ of\ blade(in\ model)} = \frac{\frac{1.13*10^{-3}}{3}}{0.0833} = 4.52*10^{-3}$

∴ The dimension of one blade= (0.0833*0.00452) m

The mechanical propeller of (3)blades of diameter (DT= 8.33 cm),with (W_B=0.45 cm),it is rotating by a mixing motor of (N=1371 can say 1400 rpm) at each flow rate , Figure (B-1) shows the mechanical propeller .

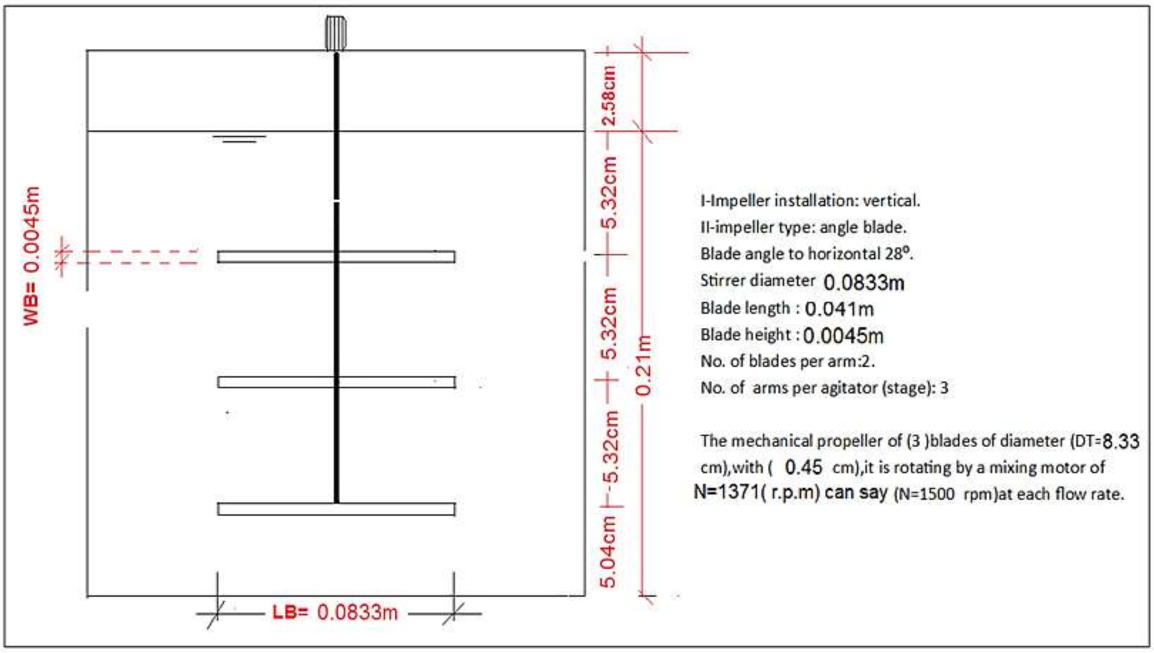


Figure (B-1): Mechanical propeller used in rapid mixing tank(pilot plant)

• To find Radius and depth of available tank (in pilot plant),the calculation are :

For λ= 4 , $\lambda r = \frac{1}{50}$, $\lambda h = \frac{1}{12.5}$

λ_Q= 4.525*10^{-4}

$\lambda_Q = \frac{Q_m}{Q_p}$

$Q_m = \lambda_Q * Q_p$

Q_m = 4.525*10^{-4}*1050m³/hr.

Q_m= 0.475m³/hr.

$Q_m = \frac{\forall_m}{t}$

$\frac{0.475}{3600} = \frac{\forall_m}{78sec}$

$\forall_m$=0.01m³

The ratio of water tank height to diameter in prototype = $\frac{H}{D} = \frac{4.73}{5.5}$ = 0.86

H= 0.86 D

$\forall_m = \frac{\pi D^2}{4}$ *H

$\forall_m = \frac{\pi D^2}{4} * 0.86D$

0.01= 0.675D³

D=0.245m≈ $24.5cm$

H=0.86 *24.5=0.21m≈ $21cm$

B.1.2 Design Criteria

According to(Kawamura,1976) , (Peavy et, al.,1985) and (Smethurst,1997).

1- Detention time =(10-120)sec .

2- $G_{mix} > 750\ s^{-1}$

B.1.3 Design Procedure

•Detention time in (model) :

-For rapid mixing tank in (model)

*At max. flow rate (1.18m³/hr.):

$t_{mix}=\frac{\forall}{Q}=\frac{0.01029}{1.18}$ = 8.72*10⁻³ hr. =31.39 sec

*At flow average rate (0.475m³/hr.) :

t_{mix}= 0.01029/0.475 hr. = 0.02166*3600=78 sec

*At flow rate (0.712m³/hr.):

$t_{mix}=\frac{0.01029}{0.712}$ = 0.01445 hr. = 52 sec.

*At flow rate (0.95 m³/hr.):

$=\frac{0.01029}{0.95}*3600$= 0.0105 hr. = 39 sec

• For G_{mix} values according to (peavey et, al., 1985), the power (p) dissipated for the given calculated is as follows :

Paddle area rotating in the Crosse section= (0.0833*0.0045*3)=1.124*10^{-3}

P = $\mu \forall G^2$

where:

p= power (watt)

$\mu = dynamic\ viscosity\ of\ fluid\ N.S/m^2$ =0.890*$10^{-3}$for water at 25c°.

G = Velocity gradient (s^{-1})

Let assume G=750 $\bar{s}^{1}$ G=(700-1000)S^{-1}

P= 0.890*10^{-3}* 0.01*$(750)^2$

P=5 watt

The linear velocity of paddle blades (V_p) = $\frac{2\bar{\pi}R\ N}{60}$

Velocity differential for paddle(Vd) =0.6 the linear velocity of paddle blades

Total power input (p) = $\frac{CD*A_p\ \rho_w*V^3}{2} = \frac{CD*A_p\ \rho_w*(2\bar{\pi}rN)^3*(1-K)^3}{2}$

$$P = \frac{1.47*997*(0.0833*0.0045*3)(2*3.14*0.04165N)^3}{2}*(\frac{0.6}{60})^3$$

P =1.474*$10^{-8}$$N^3$

$N^3 = \frac{5}{(1.474*10^{-8})}$

N^3=339213026

N= 697.414

P= $\mu\ \forall\ G^2$ = 0.01*0.890*10^{-3}* G^2

= 8.90*10^{-6} G^2

1.474*$10^{-8}$$N^3$ = 8.9*$10^{-6}$$G^2$

1.474*10^{-8}*$(697.414)^3$ = 8.9*10^{-6} G^2

$G^2 = \frac{1.474*10^{-8}*339213026}{8.9*10^{-6}} = \frac{5}{8.9*10^{-6}}$

G^2 = 561798

G = 749.531≈ 750 , G =(700-1000) s^{-1}

$G^2 = \frac{1.474*10^{-8}N^3}{8.9*10^{-6}} = 1.656*10^{-3}N^3$

$G = 0.04069*(N)^{\frac{3}{2}}$

When G = 750 , $N^{2/3} = \frac{750}{0.04069}$

$N = (18432)^{2/3}$

N= 697.775

Where:

CD = Drag coefficient, Table (B-1) shows drag coefficient of flat blade

Table (B-1) Drag coefficient (CD) of flat blade (AL.Nakeeb 2000)

Ratio $\frac{L_B}{W_B}$	CD
< 1	1.16
1 to 5	$1.16+[\frac{L_B}{W_B} -1]* 0.01$
5 to 20	$1.2+[\frac{L_B}{W_B} -5]* 0.02$
>20	$1.5+[\frac{L_B}{W_B} -20]* 0.04$

B.2 Design of Flocculation Tank

B.2.1 Design Condition (Pilot Plant)

- Max. flow =1.18 m^3/hr. and
- Radius of available tank in(pilot plant) = $(\frac{7}{50} - \frac{0.8}{50})$ = 0.124m , height = 0.3128m are calculated below :

Radius of flocculation basin in (pilot plant) = scale factor * Radius of flocculation basin (in prototype).

Radius of flocculation basin in(pilot plantl) = $\frac{1}{50}$*(6.2)=0.124m=12.4cm

Depth of flocculation basin in(pilot plant) = scale factor * Depth of flocculation basin(in prototype).

Depth of flocculation basin (in pilot plant) = $\frac{1}{12.5}$*3.91= 0.3128 m

- There are four (4 NO.) vertical ,slowly revolving agitators (paddle mixers)in each flocculator tank .Thy are evenly spaced inside the inner flocculation zone, around the inlet well, where the coagulated raw water from the flash mixer enters the clarifier tank.
- Agitator details:

Table (B.2) shows the paddle specification in flocculation basin in WTP.

Table (B-2): Paddle specification used in prototype flocculation basin.

NO. of paddle Arms		2x4
NO. of paddle Blades per Agitator		12
Paddle Diameter	3000mm	
Paddle Height	3500mm	
Paddle width	150mm	

B.2.2 Design Criteria

According to (Kawamura,1976),(Peavy el.at.,1985) and (Mc Ghee,1991) for flocculation basin in prototype unit. .

Detention time:(20-30)min

(Gt_{flocc}):(2*10^4- 6*10^4)

Total area of paddle :(10-24)% of the vertical Crosse sectional area of the tank Rotational speed of impeller greater than 100 r. p. m.

Velocity of tip of blades , v_i:(0.3-0.4m/s).

Velocity of water at tip of blades V=25% of above v_i in m/s .

G=(15-60)s^{-1}(Qasim,2002)

Peripheral speed of paddle =0.2-0.6 m/s (typical 0.4 m/s)

B.2.3 Design Procedure :

- Detention time (in model)

At max. flow :1.18m^3/hr.

$t_{floc.} = \frac{0.06*0.3128}{1.18}$ = 0.0159 hr.= 0.954 min =57.24sec.

At min. flow : 0.475m^3/hr.

$t_{floc.} = \frac{0.06*0.3128}{0.475}$ = 0.0395*60 = 2.37 min.= 142.24 sec.

P= $\mu\, G^2 \forall$

Let $G_{floc.}$ =50 S^{-1} , G=(15-60)S^{-1}, (Syed R. Qasim,)

P= 0.890*10^{-3}*50^2*(0.06*0.3128)

P= 0.0417 watt

Velocity of water of tip blades = 0.25X0.3 = 0.075.

P= $\frac{1}{2}$ CD* $\rho * AP *$(V-v_i)^3

P= $\frac{1}{2}$ * 1.8* 997 $* AP *$(0.3-0.075)^3, CD=1.8 for flat paddle

0.0417=10.22 AP

AP= 4.08*$10^{-3}$$m^2$

Ratio of area of paddles to vertical Crosse sectional area of flocculator

$=\frac{AP}{2\pi RH} = \frac{4.08*10^{-3}}{2*3.14*0.124*0.3128}$ *100= 1.67%< (10-25)%

Thus provide 6 No. of paddles of height 0.25 m

Area of one paddle = $\frac{AP(total)}{6} = \frac{4.08*10^{-3}}{6} = 6.8*10^{-4}$

Width of paddle = $\frac{AP}{length\ of\ paddle} = \frac{6.8*10^{-4}}{0.25} = 2.72\text{X } 10^{-3}\text{m} \approx 0.3cm$

<u>To find the value of N</u>

$P=\frac{CD\rho}{2} * AP*(2R\pi N)^3*(1-K)^3$

$P= \frac{\mathbf{1.8}}{\mathbf{2}} * 997*(4.08*10^{-3})(2*0.031*3.14*N)^3(\frac{0.7}{60})^3$

Where p = $\forall * \mu * G^2$

$P = (0.06*0.3128)*0.890*10^{-3}G^2$

$(0.06*0.3128)*0.890*10^{-3}G^2$
$= \frac{1.8}{2}*997*(4.08*10^{-3})*(2*0.031*3.14*N)^3 *(\frac{0.7}{60})^3$

$1.66608*10^{-5}G^2 = 3.660 *(7.3784*10^{-3}N^3)*(1.5879*10^{-6})$

$G^2=\frac{4.28810^{-8}N^3}{1.66608*10^{-5}} = 2.57388*10^{-3}N^3$

$G=\sqrt{2.57388 * 10^{-3}N^3} = 0.050*N^{3/2}$.

$G=0.050\ N^{3/2}$

If $G=50\ s^{-1}$, $N^{3/2}= \frac{50\ S^{-1}}{0.05}$,$N= (\frac{50\ S^{-1}}{0.05})^{2/3}$

N=100 rpm

If G = 30 S^{-1} , $N^{3/2}=\frac{30\ S^{-1}}{0.05}$

N= 71.13 rpm

Figure (B-2)illustrated the shape of mixer used in flocculation basin

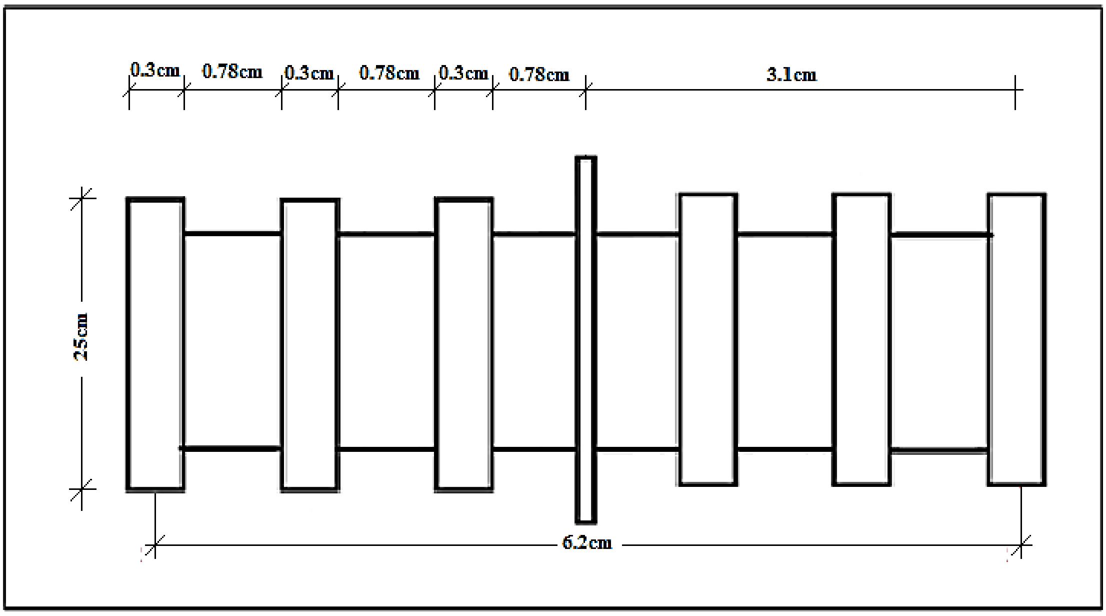


Figure (B-2):Type of mixer used in flocculation basin of pilot plant

B.3 Design Clarification System (Sedimentation Basin)

B.3.1 Design Condition

- Available tank dimension (r = 0.23m , h = 0.335)
- Area of basin=[$(\frac{19}{50})^2$-$(\frac{7.5}{50})^2$]*3.14= 0.382 m²
- $\forall = A * H = 0.382\text{m}^2 * \frac{4.19}{12.5}$ = 0.128m³
- One type of up flow plate settler model (plane-plate settler)
- Max. flow rate= 1.18 m³/hr..

B.3.2 Design Criteria

According to (Qasim ,2002), Water Works Engineering Planning, Design and Operation:

- SOR(V_o) =1.2 - 4.5 m³/m².hr.
- Detention time (2-3) hrs.
- Horizontal (radial) velocity , (Vr) ≤ 0.3 m/min.
- Weir loading rate ≤ 300m³/m/day (Syed R.Qasim ,2002)

B.3.3 Design Procedure :

At max. flow rate (1.18 m³/hr).

-Detention time(t_s) = (0.128)/(1.18) *60= 6.5 min.

-SOR(V_o) = (1.18) /(0.382) =3.08m³/m².hr.

-Horizontal flow velocity , (Vr $= \frac{Q}{A}$ =) = $\frac{1.18}{2*3.14*\frac{19}{50}*\frac{4.19}{12.5}}$ = 1.475 m/hr.

= 0.0245m/min < 0.3m/min.

- Plate settler dimension.

There are two cones can be used inside sedimentation tank as shown in the Figure (B-3). The total area of settler approximate net spacing 0.04 m

The distance between flocculation wall and sedimentation wall = 0.23m.

The curved surface area of frustum = πL (R_1+R_2)

Where :-

R_1= Radius of the lower base of frustum.

R_2= Radius of the upper base of frustum .

L= Oblique height.

H=Height of cone.

Cone no. 1:

R_1= 21cm.

R_2= 29 cm.

$H = 16$cm

curved surface area of frustum=3.14X 18.47(21+29) = 2900 cm²

Cone no.2:

R_1= 25cm.

R_2= 34 cm.

$L = 18.47$cm

$H = 16$cm

curved surface area of frustum=3.14X 18.47(25+34) = 3422Cm²

Total area of the two cones(total settling area) = 2900+3422= 6322 cm² = 0.6322 m²

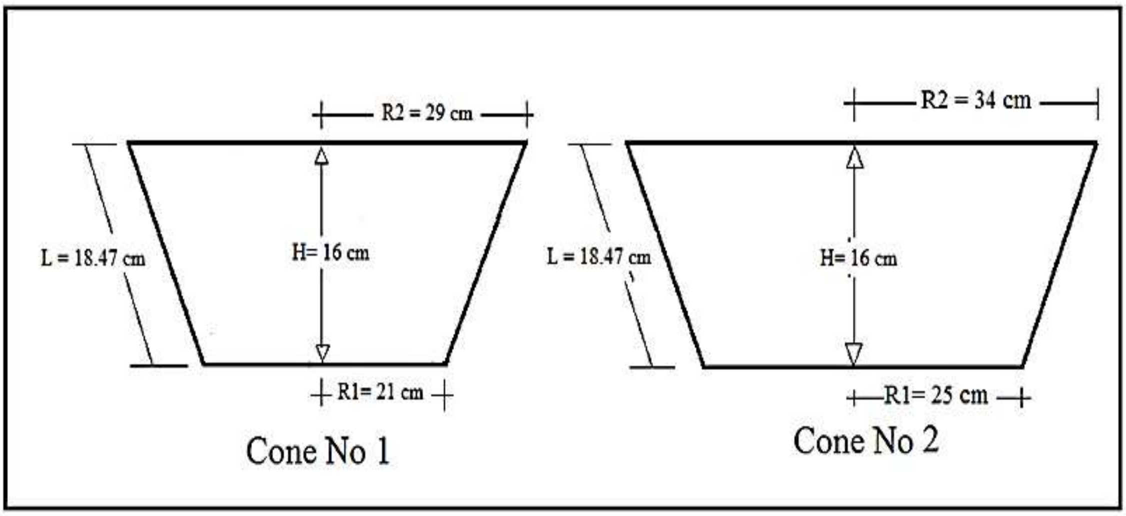


Figure (B-3) : Type of plate settler using in pilot plant

- Plates angle of inclination(θ)= 60°,(JA Salvato,2003), (Fadel and Baumann ,1990).

A- At Max. Flow Rate (1.18 m^3/hr.) :

-Detention time at max. flow rate(1.18m^3/hr.)= $\frac{0.128}{1.18}$ = 0.108 hr. = 6.51min.

-Horizontal velocity at max. flow rate(1.18m^3/hr.) = $\frac{1.18m^3/hr.}{2*3.14*\frac{19}{50}*\frac{4.19}{12.5}}$ =1.475m/hr. = 0.0245m/min.< 0.3 m/min.

- SOR at max. flow rate (1.18m^3/hr.), with plate settler = $\frac{1.18m^3/hr.}{0.6322\ m^2}$ = 1.86 m^3/m^2.hr……(1.2-4.5) m^3/m^2.hr.

B- At Flow Rate (0.95 m^3/hr.) :

-Detention time at flow rate (0.95m^3/hr.) = $\frac{0.128m^3}{0.95m^3/hr.}$ = 0.134 hr. = 8.08min.

-Horizontal velocity at flow rate(0.95m^3/hr.) = $\frac{0.95m^3/hr.}{2*3.14*\frac{19}{50}*\frac{4.19}{12.5}}$ = 1.187 m/hr.= 0.0197m/min.< 0.3 m/min.

- SOR at flow (0.95 m^3/ hr.),with plate settler = $\frac{0.95m^3/h}{0.6332m^2}$ = 1.5 m^3/m^2. hr.

C- At Flow Rate (0.712m^3/hr.):

-Detention time at flow rate (0.712m^3/hr.) = $\frac{0.128m^3}{0.712m^3/hr.}$ = 0.179 hr. = 10.79min.

-Horizontal velocity at flow rate(0.712m^3/hr.) = $\frac{0.712m^3/hr.}{2*3.14*\frac{19}{50}*\frac{4.19}{12.5}}$ = 0.890 m/hr.= 0.015 m / min.< 0.3 m/min.

- SOR at flow (0.712 m^3/ hr.),with plate settler = $\frac{0.712\ m^3/hr.}{0.6332m^2}$ = 1.124 m^3/m^2.hr. (No needs plate settler) (1.2-4.5)m^3/m^2. hr.

D - At Flow Rate (0.475 m³/hr.):

-Detention time at flow rate $(0.475\text{m}^3/\text{hr.}) = \frac{0.128}{0.475}$ *60= 16.17 min.

-Horizontal velocity at flow rate(0.475m³/hr.) = $\frac{0.475m^3/hr.}{2*3.14*\frac{19}{50}*\frac{4.19}{12.5}}$ = 0.593 m/hr.= 9.89*10⁻³ m/min.< 0.3 m/min.

- SOR at flow (0.475 m³/hr.),with plate settler = $\frac{0.475\ m^3/hr.}{0.6332m^2}$ = 0.75m³/m².hr. (no needs plate settler)(1.2-4.5)m³/m².hr .

B.4 Design Weir System (Sedimentation Basin)of pilot plant

B.4.1 Design Condition

According to (Qasim ,2002)

- Max. flow rate: 1.18m³/hr.

-Min. flow rate: 0.475m³/hr.

-Available tank dimension (r = 0.38 m)

B.4.2 Design Procedure

A - At max. Flow Rate (1.18 m³/hr.) :

- weir loading rate = $\frac{Q}{2\pi r} = \frac{1.18*24}{2*3.14*0.38}$ =11.86m³/m/day ≤300m³/m/day

B - At Flow Rate (0.95 m³/hr.) :

- weir loading rate = $\frac{Q}{2\pi r} = \frac{0.95*24}{2*3.14*0.38}$ = 9.55 m³/m/day ≤300m³/m/day

C - At Flow Rate (0.712 m³/hr.) :

- weir loading rate = $\frac{Q}{2\pi r} = \frac{0.712*24}{2*3.14*0.38}$ = 7.16m³/m/day≤300m³/m/day

D- At Flow Rate (0.475 m^3/hr.) :

- weir loading rate = $\frac{Q}{2\pi r} = \frac{0.475*24}{2*3.14*0.38}$ =4.777m^3/m/day≤300m^3/m/day

Table (B-3) shows the design dimension at different flow were used during experimental work with plate settler.

Table (B-3): Design dimension at different flow rate for experimental work with plate settler .

Flow rate (m^3/hr.)	0.475	0.712	0.95	1.18
Detention time (min)	16.17	10.79	8.08	6.51
Horizontal velocity m/min,(<0.3m/min)	0.0099	0.015	0.0197	0.0245
SOR(m^3/m^2/h),with plate settler (1.2-4.5) (m^3/m^2/hr.)	0.75	1.124	1.5	1.86
Weir loading rate $\leq \frac{300m^3}{m}/day$)	4.77	7.16	9.55	11.86

-Assume weir type ,V-notch type

- The top width of one V-notch:

Where the width of V-notch in prototype = 0.2m

$$\frac{(width\ of\ one\ V{-}notch\ )p}{circumerence\ of\ clarifire\ tank)p} = \frac{(width\ of\ one\ V{-}notch\ )m}{circumerence\ of\ clarifire\ tank)m}$$

$$= \frac{0.2m}{119.32} = \frac{(width\ of\ one\ V{-}notch\ )m}{2.28m}$$

$$(width\ of\ one\ V-notch\ ) = \frac{0.2*2.28}{119.32} = 3.82*10^{-3}m$$

$$width\ of\ one\ V-notch = 0.382cm.$$

No. of V-notch /unit= $\frac{circumerence\ of\ clarifire\ tank)m}{(width\ of\ one\ V-notch)*2}$

= $\frac{2.28}{(3.82*10^{-3})*2}$ = 298 V- notch

-At average flow rate (0.475)m³/hr.

$\frac{Flow(average\ )m}{one\ V-notch}$ = $\frac{total\ flow}{\text{No.of V}-\text{notch}}$ =$\frac{\mathbf{1.319*10^{-4}m^3/sec}}{\mathbf{298}}$ = 4.42*10^{-7}m³/sec =1.59*10^{-3}m³/hr.

At max. flow (1.18m³/hr.= 3.277*10^{-4}m³/sec.)

Where $\frac{\boldsymbol{max\ flow\ rate}}{\boldsymbol{one\ v-notch}}$ = $\frac{\mathbf{3.277*10^{-4}}}{\mathbf{298}}$ = 3.959*10^{-3}m³/hr.

Flow at 0.712m³/hr. = $\frac{1.978*10^{-4}}{298}$ = 2.39*10^{-3} m³/hr.

Flow at 0.95m³/hr. = 2.638*10^{-4}

$\frac{flow\ rate}{one\ v-notch}$= $\frac{2.638*10^{-4}}{298}$ =3.187*10^{-3} m³/hr.

B.5 Design of Filter

B.5.1 Design Condition

-Max. flow rate =1.18m³/hr.

-Min. flow rate = 0.475m³/hr.

-Available tank dimension

- Single Filter Area (inside): 50m²
- Filter Cell Area (inside): (L x W) = (9x5.5)m
- Filter Cell Depth (Floor – platform): 3.8m
- Filter Media Depth (Sand+ Gravel): 1.3m

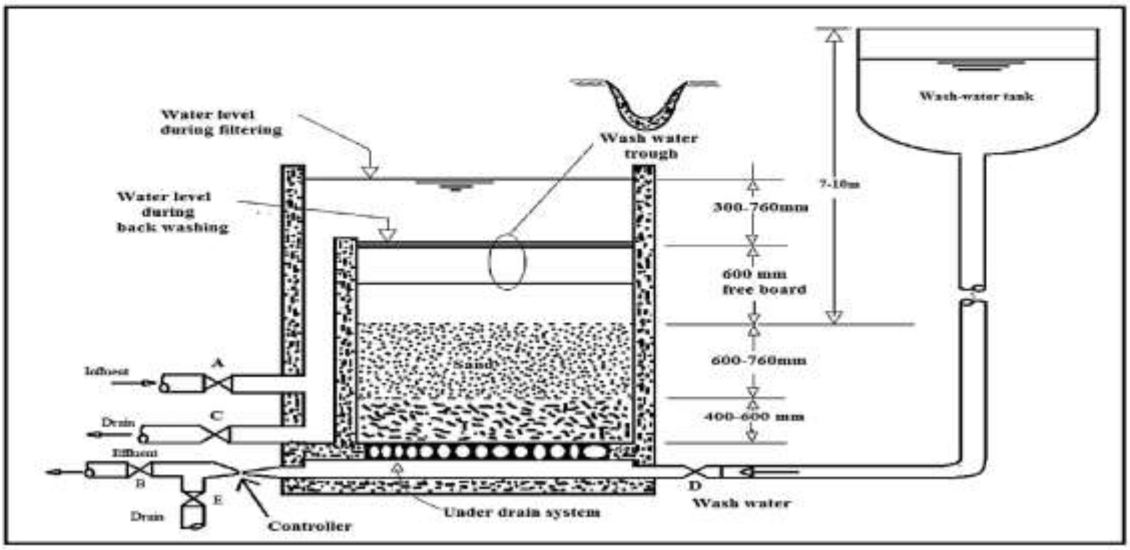
-Filter Media:

Table (B-4) shows the main characteristics of the high-quality sandy medium (clean and acid-washed), in addition to the characteristics of the supporting gravel layers used in the filters of the water treatment plant.

Table (B-4) : The characteristic of the media and support gravel layers used in real plant.

Media Type	Silica Sand
Effective Size	0.6-0.65 mm
Uniformity Coefficient	≤ 1.5
Bed Depth	700 mm
Media Type	Rounded gravel
Grading /Depth	
Layer 1(top)	2.5-6.5 mm/150mm
Layer 2	6.5-9.5mm/150mm
Layer 3	9.5-13mm/100
Layer 4	13-38mm/100mm
Layer 5 (Bottom)	38-50mm/100mm
Total Depth of support Gravel	600mm

Figure (B-4) Shows the typical gravity flow filter operation.



Figure(B-4):Typical gravity flow filter operation (Metcalf &Eddy.Inc., 1979).

Table (B-5) shows the main characteristics of the high-quality sandy medium (clean and acid-washed), in addition to the characteristics of the supporting gravel layers used in the model filters .

Table (B-5):The characteristic of the media and support gravel layers used at the pilot plant basin.

Media Type	Silica sand
Effective size	0.6 - 0.65 mm
Uniformity Coefficient	≤1.65
Bed Depth	700mm
Specific gravity	2.65
Media Type	Rounded gravel
Grading /Depth	
Layer 1(top)	2.5-6.5 mm/150mm
Layer 2	6.5-9.5 mm/150mm
Layer 3	9.5-13mm/100
Layer 4	13-38mm/100mm
Total Depth of support Gravel	500mm

B.5.2 Design Criteria

According to (.Steel ,1984),water supply and sewerage

- Filtration rate: (120-240) (m^3/m^2/day)=(5-10)m/hr.
- Effective size of sand:0.45-0.55mm
- Uniformity coefficient ≤ 1.7 and may be required be not less than 1.2
- If used anthracite alone or with sand in mixed filter the effective size of anthracite 0.7mm or more
- Uniformity coefficient of 1.75 or less .

B.5.3 Design Procedure

It is known that the water treatment rate for Karbala plant is 10,500 m^3/hr. , and since the number of filters in the station is 40 filters, this means that the flow rate that enters each filter is 262.5 m^3/hr. and that all filters operate simultaneously and there is no reserve.

Let the rate of filtration (ROF) in rapid Sand filter = 5m/hr. (120-240)(m^3/m^2/day) = (5-10) m/hr.

Filtration rate for dual –media filters range from 10 to 20m/hr

(peavey et al.,1985).

Flow rate in each filter cell in prototype = $\frac{Q_p}{4} = \frac{1050m^3/hr.}{4} = 262.5m^3/hr.$

∴ Flow rate in each filter cell in model $= \frac{Q_m}{4} = \frac{0.475m^3/hr.}{4} = 0.12m^3/hr.$

In this study, the flow rate was divided into two equal of flow, when the water exits from the sedimentation basin, the first half goes outside, and the other half is divided into two half ,first half goes to the single filter and the second half goes to the double filter, while adjusting the cross-sectional area of each filter according to the modified flow above.

- Cross section area of filtration at average flow rate = $\frac{Q_{ave.}}{ROF}$ --------(5-1)

= $\frac{0.24m^3/hr.}{5m/hr.}$ = 0.05 m^2 for two cells

From prototype the ratio of length :width =1:1 to 2:1

$\frac{L}{W}$ (prototype) = $\frac{9}{5.5}$ = 1.64

$say\ l =$ 1.64* W

A=L*W---(5-2)

A=1.64 W*W

0.05= 1.64 W^2

W=0.174m , L = 1.64 x0.174= 0.29+0.05 = 0.34

∴ area of filter (17cm*34cm).

ROF= $\frac{0.24}{(0.17*0.29)}$ = 4.86≈ $5m/hr.$

-At Flow Rate (0.712m^3/hr.).

ROF= $\frac{0.712/2}{(0.17*0.29)}$ = 7.22m/hr.

-At Flow Rate (0.95m^3/hr.).

ROF= $\frac{0.95/2}{(0.17*0.29)}$ = 9.63m/hr.

-At Flow Rate (1.18m^3/hr.).

ROF= $\frac{1.18/2}{(0.17*0.29)}$ = 12m/hr.

Where: A= Cross section area of filter model

L= length of Filter model

W= width of filter model

Dual media filters : According to (peavey et al.,1985),the thickness of silica sand range from (0.15-0.4)m, in this study it is used 0.35 m layer .

The thickness of anthracite coal layer may ranging from 0.3 to 0.6 m, also in this study it is used 0.35 m layer , with specific gravity 1.4-1.6 effective size 0.9-1.0 mm ,uniformity coefficient <1.8.

B.6 Design of Pipes

B.6.1 Design Criteria

In order to carrying flocc. without any problem ,velocity of flow should range between two limits, the lower that does not allow flocs. Settling ,and the upper does not cause flocculation shearing .

B.6.2 Design Procedure

1.Measure the tube that carries raw water from the source to the flash tank in diameter (2.54 cm) using a flexible tube. Also use 2.0 pipes of a diameter (1.25 in.,3.175 cm) to transfer water from the flash tank to the clarifier tank (flocculation tank).

2.To connect the clriflocculator with two filters, use (2.0 tubes of 1.875 cm each). drain .

3.Each of the filters is connected to each of the two activated carbon pools by a flexible tube of 1.25 cm in diameter.

It should be check its flow condition by the Reynolds number values as below:

Max. $Re \leq V*R/\nu = 2000$ [Upper limit of laminar flow].

Where :

Re = Reynolds number, dimensionless.

V= Velocity of flow in pipe (cm/sec.).

ν =Kinematic viscosity of fluid (cm^2/sec).

R=hydraulic radius of pipe(cm).

R=D/4

$$V=\frac{Q}{A}=\frac{Q}{\pi D^2/4}=\frac{4Q}{\pi D^2}$$

$$Re=\frac{\frac{4Q}{\pi D^2}*R}{\nu}=\frac{\frac{4Q}{\pi D^2}*\frac{D}{4}}{\nu}=\frac{Q/D\pi}{\nu}$$

Re= Q /π*D* ν , from which D = $\frac{Q}{\pi*Re*\nu}$

Where :

Q = flow rate passing through pipes(m^3/hr).

D = diameter of pipe used (m).

For max. flow rate :

$$(D\text{max.})_{req}=\frac{\left(\frac{1.18*10^6}{3600*2}\right)m/sec.}{\pi*2000*0.893*10^{-6}*10^4}=\frac{163.88}{56}=2.92\text{cm}>2.54\text{ cm.}$$

$\therefore$ 2.0 pipes use Ø =3.175cm pipe ,satisfied the requirement

B.7 Design Criteria for Under Drain System

Total required No. of perforated pipe line = $\frac{filter\ area}{area\ of\ one\ perforation\ (AP)}$

Space between lateral perforated pipeline (S)= (0.1-0.3)m

$$\frac{No.of\ lateral\ perforated\ pipe\ line}{filter\ area}=\frac{length\ of\ lateral\ pipe}{space\ between\ lateral\ perforated\ pipe}*2$$

No. of perforation per lateral pipe line = $\frac{total\ No.of\ perforated}{No.of\ lateral\ pipe\ line}$

$\frac{No.of\ perforation\ per\ lateral\ pipe\ line}{area\ of\ one\ perforation\ (AP)}$ = Total area of perforated in each pipe

Cross section area of lateral perforated pipeline =(2-4)(Total perforated area in each pipeline).

Total perforated area in each pipeline = $\frac{\pi d^2}{4}$

Dia. of lateral perforated pipeline = $\sqrt{\frac{Total\ perforated\ area\ in\ each\ pipeline*4}{\pi}}$

Area of manifold (collector of laterals flow) = (1.7-2) Cross section area of lateral perforated pipeline *No. of pipeline

Assume circular cross section of manifold

Area of manifold= $\frac{\pi d^2}{4}$

Dia. of manifold = $\sqrt{\frac{Area\ of\ manifold*4}{\pi}}$

Limiting velocity through manifold =1m/s[should be < 1.8-2.4m/s]

∴ discharge permitted through manifold = $\frac{\pi}{4} * dia. of\ mainfold * velocity$

Rate of back washing = $\frac{discharge\ permitted\ through\ manifold\ l/min}{area\ of\ filter}$

Limit of backwash discharge =(200-600) l/min

APPENDIX (C)

Test Result Calculation

Appendix C

C.1 First Test Result Using River Clay at :

C.1.1 Raw water 20 NTU and Flow Rate 0.475 m^3/hr.

Table (C-1) shows the result of testing at raw water =20 NTU and flow rate = 0.475m^3/hr.

Table (C-1): The result of testing at raw water =20NTU and flow rate = 0.475m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	16	13.7	4.69	3.4	3.2	2.8	3.12	2.67	2.57	1.98
E.C	1254	1250	1245	1238	1232	1240	1235	1225	1212	1226	1218
TDS	739	736	736	733	729	726	727	721	712	724	715
PH	7.9	7.9	8	7.8	7.7	7.7	7.8	7.8	7.7	7.8	7.8
Temp.	23	24	24	24	24	24	24	24	24	24	24

C.1.2 Raw Water 20 NTU and Flow Rate 0.712 m^3/hr.

Table (C-2) shows the result of testing when raw water =20 NTU and flow rate = 0.712m^3/hr.

Table (C-2): The result of testing at raw water =20 NTU and flow rate = 0.712m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	14	12	1.6	0.85	1.4	0.65	1.25	0.72	1.25	0.55
E.C	1295	1286	1270	1267	1262	1263	1254	1264	1258	1253	1248
TDS	764	757	750	747	744	745	740	746	741	738	738
PH	7.7	7.6	7.6	7.6	7.5	7.5	7.5	7.7	7.5	7.6	7.5
Temp.	25	25	25	25	25	25	25	24	24	24	24

C.1.3 Raw Water 20 NTU and Flow Rate 0.95 m³/hr.

Table (C-3) shows the result of testing when raw water =20 NTU and flow rate =0.95m³/hr.

Table (C-3): The result of testing at raw water = 20 NTU and flow rate = 0.95m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	14	11.6	3.7	1.9	1.37	1.12	2.42	1.78	0.97	0.82
E.C	1246	1238	1230	1212	1206	1209	1198	1187	1176	1170	1165
TDS	740	738	727	713	709	711	706	699	794	689	688
PH	7.9	7.9	7.8	7.7	7.7	7.7	7.6	7.6	7.6	7.6	7.6
Temp.	24	24	24	24	24	24	24	24	24	24	24

C.1.4 Raw Water 20 NTU and Flow Rate 1.18 m³/hr.

Table (C-4) shows the result of testing when raw water = 20 NTU and flow rate =1.18m³/hr.

Table (C-4): The result of testing at raw water =20NTU and flow rate =1.18m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	14	11.3	0.94	0.63	0.61	0.54	0.63	0.45	0.55	0.43
E.C	1230	1225	1222	1219	1215	1211	1206	1211	1209	1204	1201
TDS	725	721	719	716	714	711	707	711	709	706	704
PH	7.9	7.8	7.7	7.7	7.7	7.6	7.7	7.7	7.7	7.6	7.6
Temp.	23	23	23	23	23	23	23	23	23	23	23

C.1.5 Raw Water 30 NTU and Flow Rate 0.475 m³/hr.

Table (C-5) shows the result of testing when raw water =30 NTU and flow rate =0.475m³/hr.

Table (C-5): The result of testing at raw water =30 NTU and flow rate =0.475m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	30	24	20.25	0.79	0.62	0.53	0.46	0.55	0.46	0.37	0.3
E.C	1271	1203	1186	1180	1172	1174	1166	1168	1160	1156	1149
TDS	749	710	700	696	691	692	688	669	684	682	677
PH	7.8	7.7	7.7	7.7	7.7	7.6	7.6	7.5	7.5	7.4	7.4
Temp.	25	25	25	25	25	25	25	25	25	25	25

C.1.6 Raw Water 30 NTU and Flow Rate 0.712 m³/hr.

Table (C-6) shows the result of testing when raw water = 30 NTU and flow rate = 0.712m³/hr.

Table (C-6): The result of testing at raw water = 30 NTU and flow rate = 0.712m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	30	20.1	16	1.4	0.95	0.89	0.65	0.9	0.8	0.7	0.5
E.C	1321	1312	1288	1278	1257	1274	1270	1260	1254	1257	1244
TDS	780	775	758	752	744	754	749	744	740	744	729
PH	7.7	7.7	7.6	7.6	7.5	7.5	7.5	7.4	7.4	7.4	7.4
Temp.	26	26	26	26	26	26	26	26	26	26	26

C.1.7 Raw Water 30 NTU and Flow Rate 0.95 m³/hr.

Table (C-7) shows the result of testing when raw water =30 NTU and flow rate = 0.95m³/hr.

Table(C-7):The result of testing at raw water =30 NTU and flow rate = 0.95m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	30	15.42	13.8	0.65	0.5	0.52	0.39	0.54	0.44	0.5	0.37
E.C	1333	1327	1323	1311	1304	1297	1293	1298	1287	1285	1277
TDS	788	783	779	772	769	766	763	766	760	757	750
PH	7.8	7.7	7.6	7.6	7.5	7.6	7.5	7.5	7.5	7.4	7.4
Temp.	26	26	26	26	26	26	26	26	26	26	26

C.1.8 Raw Water 30 NTU and Flow Rate 1.18 m^3/hr.

Table (C-8) shows the result of testing when raw water = 30 NTU and flow rate =1.18m^3/hr.

Table (C-8):The result of testing at raw water = 30 NTU and flow rate =1.18m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	30	15.3	13.8	1.2	0.9	0.6	0.55	0.8	0.75	0.57	0.47
E.C	1290	1285	1280	1257	1244	1270	1258	1253	1240	1244	1230
TDS	759	755	750	736	728	745	737	734	726	728	720
PH	7.7	7.6	7.6	7.5	7.5	7.5	7.5	7.4	7.4	7.4	7.4
Temp.	27	27	27	27	27	27	27	27	27	27	27

C.1.9 Raw Water 40 NTU and Flow Rate 0.475 m^3/hr.

Table (C-9) shows the result of testing when raw water = 40NTU and flow rate = 0.475m^3/hr.

Table (C-9):The result of testing at raw water =40 NTU and flow rate = 0.475m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	40	28.4	26.68	1.7	1.09	0.94	0.83	1.4	0.98	0.75	0.73
E.C	1337	1331	1297	1289	1279	1284	1275	1286	1277	1270	1266
TDS	786	783	766	759	755	757	751	758	753	748	744
PH	7.7	7.7	7.7	7.6	7.6	7.5	7.5	7.5	7.5	7.4	7.4
Temp.	24	24	24	24	24	24	24	24	24	24	24

C.1.10 Raw water 40 NTU and Flow Rate 0.712 m^3/hr.

Table (C-10) shows the result of testing when raw water = 40NTU and flow rate = 0.712m^3/hr.

Table (C-10): The result of testing at raw water = 40 NTU and flow rate = 0.712m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	40	18	15	0.82	0.58	0.39	0.35	0.54	0.53	0.37	0.3
E.C	1266	1260	1246	1244	1238	1239	1235	1233	1220	1216	1208
TDS	747	744	737	736	732	733	729	725	718	714	710
PH	8	8	7.8	7.7	7.7	7.7	7.6	7.5	7.5	7.5	7.4
Temp.	27	26	26	26	26	26	26	27	26	27	26

C.1.11 Raw Water 40 NTU and Flow Rate 0.95 m^3/hr.

Table (C-11) shows the result of testing when raw water = 40NTU and flow rate = 0.95m^3/hr.

Table (C-11): The result of testing at raw water = 40 NTU and flow rate = 0.95m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	40	17	12.56	0.74	0.61	0.4	0.3	0.47	0.43	0.39	0.22
E.C	1276	1274	1271	1267	1265	1263	1256	1267	1265	1257	1252
TDS	754	753	751	749	748	747	743	749	748	743	740
PH	8	8	7.8	7.8	7.7	7.8	7.6	7.6	7.6	7.5	7.5
Temp.	26	26	26	26	26	26	26	26	26	26	26

C.1.12 Raw Water 40 NTU and 1.18 m^3/hr. Flow Rate

Table (C-12) shows the result of testing when raw water =40NTU and flow rate =1.18m^3/hr.

Table (C-12): The result of testing at raw water = 40 NTU and flow rate =1.18m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	40	15.8	10	0.7	0.44	0.6	0.23	0.6	0.28	0.54	0.22
E.C	1274	1270	1269	1267	1264	1266	1263	1259	1253	1254	1248
TDS	753	748	746	745	743	744	746	743	740	739	737
PH	7.7	7.7	7.6	7.5	7.5	7.5	7.5	7.4	7.4	7.5	7.4
Temp.	25	25	25	25	25	25	25	25	25	25	25

C.1.13 Raw Water 50 NTU and Flow Rate 0.475 m^3/hr.

Table C-13) shows the result of testing when raw water = 50NTU and flow rate = 0.475m^3/hr.

Table (C-13): The result of testing at raw water = 50 NTU and flow rate = 0.475m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	24	18	0.96	0.65	0.78	0.53	0.84	0.55	0.74	0.5
E.C	1226	1220	1210	1215	1206	1212	1198	1209	1200	1206	1193
TDS	720	717	711	714	708	712	703	710	705	708	701
PH	7.7	7.7	7.6	7.6	7.5	7.6	7.5	7.5	7.4	7.3	7.3
Temp.	28	27	27	27	27	27	27	27	27	27	27

C.1.14 Raw Water 50 NTU and Flow Rate 0.712 m^3/hr.

Table (C-14) shows the result of testing when raw water = 50NTU and flow rate = 0.712m^3/hr.

Table (C-14) : The result of testing at raw water = 50 NTU and flow rate = 0.712m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	22	17	2.3	1.79	1.4	0.6	1.1	0.56	0.67	0.46
E.C	1234	1230	1228	1226	1223	1222	1218	1223	1220	1214	1210
TDS	725	722	721	718	717	715	715	718	716	713	700
PH	7.8	7.7	7.7	7.6	7.5	7.5	7.5	7.5	7.4	7.5	7.5
Temp.	27	27	27	27	27	27	27	27	27	27	27

C.1.15 Raw Water 50 NTU and Flow Rate 0.95 m^3/hr.

Table (C-15) shows the result of testing when raw water = 50NTU and flow rate = 0.95 m^3/hr.

Table (C-15): The result of testing at raw water = 50 NTU and flow rate = 0.95m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	21	15.25	3.4	1.86	0.9	0.45	2.9	1.5	0.48	0.3
E.C	1254	1251	1246	1249	1241	1244	1239	1247	1238	1241	1233
TDS	736	735	732	733	729	730	728	732	727	729	724
PH	7.8	7.7	7.6	7.5	7.5	7.5	7.4	7.3	7.3	7.4	7.3
Temp.	26	26	26	26	26	26	26	26	26	26	26

C.1.16 Raw Water 50 NTU and Flow Rate 1.18 m³/hr.

Table (C-16) shows the result of testing when raw water =50NTU and flow rate =1.18m³/hr.

Table (C-16):The result of testing at raw water =50 NTU and flow rate =1.18m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	16	12.2	0.68	0.61	0.77	0.48	0.5	0.55	0.39	0.38
E.C	1205	1203	1199	1197	1194	1197	1191	1195	1191	1194	1188
TDS	707	707	704	703	701	703	699	702	699	701	697
PH	7.8	7.7	7.6	7.5	7.5	7.5	7.5	7.4	7.5	7.4	7.5
Temp.	27	27	27	27	27	27	27	27	27	27	27

Since river water is slightly turbid, coagulants are widely used as a suspending, stabilizing and binding agent and as an absorbent or clarifying agent in many applications. Sometimes bentonite and/or kaolin are added to the water especially when the water has low turbidity and the water is flocculated for effective flocculation (SCHUTTE,2007). Bentonite is added to raw water, especially with low turbidity, it increases the weight of the suspension and increases the density of particles in addition to providing a large surface for the adsorption of organic compounds. The dosage of bentonite clay ranges from 10 - 50 mg / liter. (Cohen and Hannah, 1971).

A new sweeping coagulation mechanism was explored, used to treat low turbidity water. This mechanism uses flocculants consisting of flocculation of bentonite dispersion with cationic polyelectrolyte instead of alum during the coagulation process. Bentonite clay was used to remove colloidal suspensions in wine, which had a positive charge, It binds and coagulates with negatively charged bentonite particles.(Murray , 2000).

By examining the data of the raw water entering the treatment plant, it is found that it has low turbidity resulting from particles of infinite size and it is necessary to remove it. Because the concentration of nanoparticles is low in water, the rate of attraction and contact between these particles limits the overall coagulation process. (Wiley & Sons,1972).

Water with low turbidity is treated by effective coagulation achieved by using alum (aluminum sulfate) which is called sweeping coagulation.(Amirtharajah and Mills,1982).

In this type of coagulation and because the dose of alum used is high, it will result in amorphous precipitation of aluminum hydroxide, which increases the incidence of collisions between particles and collides with suspended particles and thus is removed by sedimentation.

The sweeping coagulation process using alum produces a large amount of waste sludge, in addition to maintaining high levels of aluminum concentration in the treated water at both acidity and alkalinity, which raised public health problems. (Driscoll and Letterman ,1995).

For the above reasons, it was directed to the use of bentonite in this study and for another purpose, which is to increase the turbidity of the raw water.

C.2 Second Test Result Using Bentonite Clay at :

C.2.1 Raw Water 20 NTU and Flow Rate 0.475 m^3/hr.

Table (C-17) shows the result of testing at raw water = 20 NTU and flow rate = 0.475m^3/hr.

Table (C-17): The result of testing at raw water = 20NTU and flow rate = 0.475m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	12.3	11.2	3	2.55	2.58	2.52	2.45	2.44	2.22	1.1
E.C	1167	1134	No Test	1155	No Test	1056	No Test	1077	No Test	1024	No Test
TDS	670	647	No Test	662	No Test	604	No Test	615	No Test	588	No Test
PH	8.4	8.2	No Test	8.2	No Test	8.2	No Test	8.1	No Test	8.1	No Test
Temp.	17	17	No Test	17	No Test	17	No Test	17	No Test	17	No Test

C.2.2 Raw Water 20 NTU and Flow Rate 0.712 m^3/hr.

Table (C-18) shows the result of testing at raw water = 20 NTU and flow rate = 0.712m^3/hr.

Table (C-18): The result of testing at raw water =20NTU and flow rate = 0.712m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	11.9	10	3	2.9	2.87	2.72	2.66	2.33	2.12	0.97

C.2.3 Raw Water 20 NTU and Flow Rate 0.95 m^3/hr.

Table (C-19) shows the result of testing at raw water = 20 NTU and flow rate = 0.95m^3/hr.

Table (C-19) the result of testing at raw water = 20NTU and flow rate = 0.95m^3/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	13	11	2.95	1.46	1.34	0.96	1.98	0.99	0.91	0.43
E.C	1183	1174	1149	1156	1138	1144	1128	1127	1115	1126	1106
TDS	685	675	655	664	658	663	643	641	634	647	628
PH	8.2	8.1	8.1	8	8	8	8	7.9	7.9	7.9	7.9
Temp.	18	18	18	18	18	18	18	18	18	18	18

C.2.4 Raw Water 20 NTU and Flow Rate 1.18 m³/hr.

Table (C-20) shows the result of testing at raw water = 20 NTU and flow rate = 1.18m³/hr.

Table (C-20): The result of testing at raw water = 20NTU and flow rate = 1.18m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	20	12	10.2	2.67	1.44	2.46	1.33	2.21	0.98	0.86	0.33

C.2.5 Raw Water 50 NTU and 0.475 m³/hr. Flow Rate

Table (C-21) shows the result of testing at raw water = 50 NTU and flow rate = 0.475m³/hr.

Table (C-21):The result of testing at raw water = 50NTU and flow rate = 0.475m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	18	15.4	0.43	0.37	0.3	0.19	0.37	0.23	0.15	0.13
E.C	1180	1176	No Test	1165	No Test	1159	No Test	1167	No Test	1144	No Test
TDS	680	672	No Test	658	No Test	648	No Test	661	No Test	639	No Test
PH	8.3	8.2	No Test	8.2	No Test	8.1	No Test	8.1	No Test	8.1	No Test
Temp.	17	17	No Test	17	No Test	17	No Test	17	No Test	17	No Test

C.2.6 Raw Water 50 NTU and Flow Rate 0.712 m³/hr.

Table (C-22) shows the result of testing at raw water = 50 NTU and flow rate = 0.712m³/hr.

Table (C-22):The result of testing at raw water = 50NTU and flow rate = 0.712m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	17.4	14.6	0.56	0.44	0.4	0.23	0.54	0.4	0.33	0.25

C.2.7 Raw Water 50 NTU and Flow Rate 0.95 m³/hr.

Table (C-23) shows the result of testing at raw water =50 NTU and flow rate = 0.95m³/hr.

Table (C-23):The result of testing at raw water =50NTUand flow rate = 0.95m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	16	13	0.88	0.42	0.49	0.7	0.7	0.4	0.45	0.38
E.C	1190	1184	1174	1166	1164	1169	1155	1158	1146	1123	1065
TDS	683	677	650	635	628	644	642	550	538	541	518
PH	8.3	8.2	8.1	8.1	8	8	8	8	8	8	8
Temp.	17	17	17	17	17	17	17	17	17	17	17

C.2.8 Raw Water 50 NTU and 1.18 m³/hr. Flow Rate

Table (C-24) shows the result of testing at raw water =50 NTU and flow rate =1.18m³/hr.

Table (C-24):The result of testing at raw water = 50NTU and flow rate = 1.18m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	50	15.74	14.1	0.39	0.36	0.6	0.33	0.34	0.26	0.22	0.19

C.2.9 Raw Water 120 NTU and Flow Rate 0.475 m³/hr.

Table (C-25) shows the result of testing at raw water =120 NTU and flow rate =0.475m³/hr.

Table (C-25):The result of testing at raw water =120NTU and flow rate =0.475m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	120	25	21	1.18	0.9	1.15	0.97	1	0.85	0.44	0.4

C.2.10 Raw Water 120 NTU and Flow Rate 0.712 m³/hr.

Table (C-26) shows the result of testing at raw water = 120 NTU and flow rate = 0.712m³/hr.

Table (C-26):The result of testing at raw water =120NTU and flow rate = 0.712m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	120	23	20.1	1.23	0.94	1.19	1.1	1.12	0.86	0.7	0.38

C.2.11 Raw Water 120 NTU and . Flow Rate 0.95 m³/hr.

Table (C-27) shows the result of testing at raw water =120 NTU and flow rate = 0.95m³/hr.

Table (C-27):The result of testing at raw water =120NTU and flow rate = 0.95m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	120	20	15	1	0.56	1	0.4	0.85	0.45	0.43	0.35
E.C	1215	1208	1194	1198	1187	1175	1159	1161	1166	1149	1123
TDS	692	689	680	682	674	668	665	668	669	654	643
PH	8.3	8.2	8.1	8.1	8	8	7.9	7.8	7.8	7.7	7.7
Temp.	17	17	17	17	17	17	17	17	17	17	17

C.2.12 Raw Water 120 NTU and Flow Rate 1.18 m³/hr.

Table (C-28) shows the result of testing at raw water =120 NTU and flow rate =1.18m³/hr.

Table (C-28):The result of testing at raw water =120NTU and flow rate = 1.18m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	120	21	14.4	0.93	0.66	0.99	0.43	0.89	0.47	0.59	0.3

C.2.13 Raw Water 200 NTU and Flow Rate 0.475 m³/hr.

Table (C-29) shows the result of testing at raw water = 200 NTU and flow rate = 0.475m³/hr.

Table (C-29):The result of testing at raw water =200NTU and flow rate = 0.475m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	200	13	11	1.01	0.82	0.59	0.46	0.36	0.33	0.27	0.22
E.C	1274	1277	No Test	1265	No Test	1259	No Test	1260	No Test	1218	No Test
TDS	670	654	No Test	645	No Test	635	No Test	601	No Test	588	No Test
PH	8	7.7	No Test	7.6	No Test	7.6	No Test	7.7	No Test	7.6	No Test
Temp.	17	17	No Test	17	No Test	17	No Test	17	No Test	17	No Test

C.2.14 Raw Water 200 NTU and Flow Rate 0.712 m³/hr.

Table (C-30) shows the result of testing at raw water = 200 NTU and flow rate = 0.712m³/hr.

Table (C-30):The result of testing at raw water = 200NTU and flow rate = 0.712m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	200	13.6	11.4	1.22	0.77	2	0.88	0.99	0.84	0.76	0.58

C.2.15 Raw Water 200 NTU and Flow Rate 0.95 m³/hr.

Table (C-31) shows the result of testing at raw water = 200 NTU and flow rate = 0.95m³/hr.

Table (C-31) :The result of testing at raw water =200NTU and flow rate = 0.95m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	200	11	10.4	1.24	0.91	0.92	0.58	1.05	0.48	0.45	0.22
E.C	1265	1244	1225	1233	1216	1221	1201	1210	1197	1217	1176
TDS	730	713	698	707	694	701	684	692	683	696	672
PH	8.1	7.9	7.8	8.1	8	8	7.9	7.8	7.8	7.7	7.7
Temp.	17	17	17	17	17	17	17	17	17	17	17

C.2.16 Raw Water 200 NTU and Flow Rate 1.18 m³/hr.

Table (C-32) shows the result of testing at raw water =200 NTU and flow rate =1.18m³/hr.

Table (C-32):The result of testing at raw water = 200NTU and flow rate =1.18m³/hr.

Type of test	Raw water	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Turbidity	200	10.9	10.2	1.12	0.92	1.49	0.79	0.95	0.78	0.66	0.41

C-3 The Removal Efficiency of the Parameters of Physical and Chemical Properties of Water in Experimental Study Using River Soil.

From the Tables above (C-1) to (C-16) , which includes the results of the test of physical and chemical parameters, the removal efficiency for each parameter can be determined by equation (C-1) at each flow rate and at each turbidity value in each processing unit in this study Experimental.

$$\% \text{ Removal Efficiency} = \frac{W1-W2}{W1} * 100 \text{----------------} (C\text{-}1)$$

Tables (C-33) to (C-48) describe the Removal Efficiency for each parameter at each flow rate and turbidity value, using the river, has been identified from the equation (C-1).

Table (C-33) Shows the removal efficiency of parameters at flow rate = 0.475m³/hr, Turbidity = 20 NTU, Using river soil.

Table (C-33):Removal efficiency of parameters at flow rate = 0.475m³/hr, Turbidity=20 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}$ *100
Turbidity	20	1.98	90.1
EC	1254	1218	2.87
TDS	739	715	3.2
pH	7.9	7.8	1.26

Table (C-34) Shows the removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity = 20 NTU, Using river soil.

Table (C-34):Removal efficiency of parameters at flow rate = 0.712m^3/hr, Turbidity=20 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	0.55	97.25
EC	1295	1248	3.63
TDS	764	738	3.40
pH	7.7	7.5	2.6
Hardness	408	375	8.09
Calcium	120	101	15.83
Magnesium	26	30	-15.38
Alkalinity	88	72	18.18
Chloride	113	95	15.93

Table (C-35) Shows the removal efficiency of parameters at flow rate = 0.95m^3/hr, Turbidity = 20 NTU, Using river soil.

Table (C-35):Removal efficiency of parameters at flow rate = 0.95m^3/hr, Turbidity=20NTU,Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	0.82	95.9
EC	1246	1165	6.5
TDS	740	688	7.03
pH	7.9	7.6	3.80

Table (C-36) Shows the removal efficiency of parameters at flow rate = $1.18m^3/hr$, Turbidity = 20 NTU, Using river soil.

Table (C-36):Removal efficiency of parameters at flow rate = 1.18m³/h., Turbidity = 20 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	0.43	97.05
EC	1230	1201	2.36
TDS	725	704	2.9
pH	7.9	7.6	3.8
Hardness	428	388	9.34
Calcium	126	106	15.87
Magnesium	28	30	-7.14
Alkalinity	108	96	11.11
Chloride	107	93	13.08

Table (C-37) Shows the removal efficiency of parameters at flow rate = $0.475m^3/hr$, Turbidity = 30 NTU, Using river soil.

Table (C-37):Removal efficiency of parameters at flow rate = 0.475m³/hr, Turbidity=30NTU,Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	30	0.3	99
EC	1271	1149	9.6
TDS	749	677	9.61
pH	7.8	7.4	5.13
Hardness	408	340	16.67
Calcium	104	95	8.65
Magnesium	36	25	30.55
Alkalinity	108	80	25.92
Chloride	109	90	17.43

Table (C-38) Shows the removal efficiency of parameters at flow rate = 0.712m³/hr, Turbidity = 30 NTU, Using river soil.

Table (C-38):Removal efficiency of parameters at flow rate = 0.712m³/hr, Turbidity=30 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	30	0.5	98.33
EC	1321	1244	5.83
TDS	780	729	6.54
pH	7.7	7.4	3.9

Table (C-39) Shows the removal efficiency of parameters at flow rate = 0.95m³/hr, Turbidity = 30 NTU, Using river soil.

Table (C-39): Removal efficiency of parameters at flow rate = 0.95m³/hr, Turbidity=30 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	30	0.37	98.76
EC	1333	1277	4.2
TDS	788	750	4.8
pH	7.8	7.4	5.12

Table (C-40) Shows the removal efficiency of parameters at flow rate = 1.18 m^3/hr, Turbidity = 30 NTU, Using river soil.

Table (C-40):Removal efficiency of parameters at flow rate = 1.18 m^3/hr, Turbidity=30 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	30	0.47	98.43
EC	1290	1230	4.65
TDS	759	720	5.13
pH	7.8	7.4	3.9

Table (C-41) Shows the removal efficiency of parameters at flow rate = 0.475 $m^3/hr.$, Turbidity = 40 NTU, Using river soil.

Table (C-41): Removal efficiency of parameters at flow rate = 0.475 m^3/hr, Turbidity=40 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	30	0.73	98.17
EC	1337	1266	5.31
TDS	786	744	5.34
pH	7.7	7.4	3.9

Table (C-42) Shows the removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity = 40 NTU, Using river soil.

Table (C-42): Removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity=40 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	40	0.3	99.25
EC	1266	1208	4.58
TDS	747	710	4.95
pH	8	7.4	7.5

Table (C-43) Shows the removal efficiency of parameters at flow rate = 0.95 m^3/hr, Turbidity = 40 NTU, Using river soil.

Table(C-43):Removal efficiency of parameters at flow rate = 0.95 m^3/hr, Turbidity=40NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	40	0.22	99.45
EC	1276	1252	1.88
TDS	754	740	1.86
pH	8	7.5	6.25

Table (C-44) Shows the removal efficiency of parameters at flow rate = 1.18 m^3/hr, Turbidity = 40 NTU, Using river soil.

Table (C-44):Removal efficiency of parameters at flow rate = 1.18 m^3/hr, Turbidity=40NTU,Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}$ *100
Turbidity	40	0.22	99.45
EC	1274	1248	2.04
TDS	753	737	2.12
pH	7.7	7.4	3.9

Table (C-45) Shows the removal efficiency of parameters at flow rate = 0.475 m^3/hr, Turbidity = 50 NTU, Using river soil.

Table (C-45):Removal efficiency of parameters at flow rate = 0.475 m^3/hr, Turbidity=50 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{w1-w2}{w2}$ *100
Turbidity	50	0.5	99
EC	1226	1193	2.69
TDS	720	701	2.64
pH	7.7	7.3	5.19

Table (C-46) Shows the removal efficiency of parameters at flow rate = 0.712 m³/hr, Turbidity = 50 NTU, Using river soil.

Table (C-46):Removal efficiency of parameters at flow rate = 0.712 m³/hr, Turbidity=50NTU,Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.46	99
EC	1234	1210	1.94
TDS	725	700	3.45
pH	7.8	7.5	3.85

Table (C-47) Shows the removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity = 50 NTU, Using river soil.

Table (C-47): Removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity=50 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.3	99.4
EC	1254	1233	1.67
TDS	736	724	1.63
pH	7.8	7.3	6.41

Table (C-48) Shows the removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity = 50 NTU, Using river soil.

Table (C-48):Removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity=50 NTU, Using river soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.38	99.24
EC	1205	1188	1.41
TDS	707	697	1.41
pH	7.8	7.5	3.85

C- 4 The Removal Efficiency of the Parameters of Physical and Chemical Properties of Water in Pilot plant ,using Bentonite Soil.

From the Tables above (C-17) to (C-32) , which includes the results of the test of physical and chemical parameters, the removal efficiency for each parameter can be determined by equation (C-1) at each flow rate and at each turbidity value in each processing unit in this study experimental.

Tables (C-49) to (C-64) describe the removal efficiency for each parameter at each flow rate and turbidity value, using bentonite , has been identified from the equation (C-1).

Table (C-49) Shows the removal efficiency of parameters at flow rate = 0.475 m³/hr., Turbidity = 20 NTU, using bentonite soil.

Table (C-49):Removal efficiency of parameters at flow rate = 0.475 m^3/hr, Turbidity=20NTU, using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media without using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	2.22	88.9
EC	1167	1024	12.25
TDS	670	588	12.23
pH	8.4	8.1	3.57

Table (C-50) Shows the removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity = 20 NTU, using bentonite soil.

Table (C-50): Removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity= 20 NTU, using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media without using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	0.97	95.15

Table (C-51) Shows the removal efficiency of parameters at flow rate = 0.95 m³/h., Turbidity = 20 NTU, using bentonite soil.

Table (C-51):Removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity=20NTU,usingbentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	0.43	97.85
EC	1183	1106	6.50
TDS	685	628	8.32
pH	8.2	7.9	3.66

Table (C-52) Shows the removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity = 20 NTU, using bentonite soil.

Table (C-52):Removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity=20NTU,using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	20	0.33	98.35

Table (C-53) Shows the removal efficiency of parameters at flow rate = 0.475 m³/hr, Turbidity = 50 NTU, using bentonite soil.

Table (C-53):Removal efficiency of parameters at flow rate = 0.475 m³/hr, Turbidity=50NTU, using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media without using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.15	99.7
EC	1180	1144	3.05
TDS	680	639	6.02
pH	8.3	8.1	2.41

Table (C-54) Shows the removal efficiency of parameters at flow rate = 0.712 m³/hr, Turbidity = 50 NTU, Using bentonite soil.

Table (C-54): Removal efficiency of parameters at flow rate = 0.712 m³/hr, Turbidity=50 NTU, Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.25	99.5

Table (C-55) Shows the removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity = 50 NTU, Using bentonite soil.

Table (C-55):Removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity=50NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.38	99.24
EC	1190	1065	10.5
TDS	683	518	24.15
pH	8.3	8	3.61

Table (C-56) Shows the removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity = 50 NTU, Using bentonite soil.

Table (C-56):Removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity=50NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	50	0.19	99.62

Table (C-57) Shows the removal efficiency of parameters at flow rate = 0.475 m³/hr, Turbidity = 120 NTU, Using bentonite soil.

Table (C-57):Removal efficiency of parameters at flow rate = 0.475 m³/hr, Turbidity=120NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	120	0.4	99.66

Table (C-58) Shows the removal efficiency of parameters at flow rate = 0.712 m³/hr, Turbidity = 120 NTU, Using bentonite soil.

Table (C-58):Removal efficiency of parameters at flow rate = 0.712 m³/hr, Turbidity=120 NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	120	0.38	99.68

Table (C-59) Shows the removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity = 120 NTU, Using bentonite soil.

Table (C-59):Removal efficiency of parameters at flow rate = 0.95 m³/hr, Turbidity=120 NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	120	0.35	99.70
EC	1215	1123	7.57
TDS	692	643	7.08
pH	8.3	7.7	7.23

Table (C-60) Shows the removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity = 120 NTU, Using bentonite soil.

Table (C-60):Removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity=120NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	120	0.3	99.75

Table (C-61) Shows the removal efficiency of parameters at flow rate = 0.475 m³/hr, Turbidity = 200 NTU, Using bentonite soil.

Table (C-61):Removal efficiency of parameters at flow rate = 0.475 m^3/hr, Turbidity=200NTU,Using bentonite soil.

Test Type	Raw Water (W1)	AC in dual filter media using plate settler (W2)	Removal Efficiency = $\frac{W1-W2}{W1} * 100$
Turbidity	200	0.27	99.86
EC	1274	1218	4.39
TDS	670	588	12.24
pH	8	7.6	5

Table (C-62) Shows the removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity = 200 NTU, Using bentonite soil.

Table (C-62):Removal efficiency of parameters at flow rate = 0.712 m^3/hr, Turbidity=200NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1} *100$
Turbidity	200	0.58	99.71

Table (C-63) Shows the removal efficiency of parameters at flow rate = 0.95 m^3/hr, Turbidity = 200 NTU, Using bentonite soil.

Table (C-63):Removal efficiency of parameters at flow rate = 0.95 m^3/hr, Turbidity =200NTU,Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1} *100$
Turbidity	200	0.22	99.89
EC	1265	1167	7.03
TDS	730	672	7.94
pH	8.1	7.7	4.94

Table (C-64) Shows the removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity = 200 NTU, Using bentonite soil.

Table (C-64):Removal efficiency of parameters at flow rate = 1.18 m³/hr, Turbidity=200 NTU, Using bentonite soil.

Test type	Raw water (W1)	AC in dual filter media using plate settler (W2)	Removal efficiency = $\frac{W1-W2}{W1}*100$
Turbidity	200	0.41	99.79

C-5 Chemical and Physical Parameters Which Examined in the Real Treatment Plant for the Years (2014 to 2019) at Raw Water Turbidity Value 20 NTU.

Tables (C-65) to (C-70) shows the monthly average of chemical and physical parameters which examined for the years (2014-2019) at raw water turbidity 20 NTU for real plant**.**

Table (C-65) : The monthly average of chemical and physical parameters which examined for the year (2014) at raw water turbidity20 NTU for real plant.

2014	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	20	0.86	390	485	1056	1143	8.03	7.65
March	19.9	1.4	563.5	552.3	1116.0	1105.3	7.9	7.7
April	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
May	20.1	1.3	606.0	586.4	1206.4	1183.5	7.7	7.8
June	20.7	1.2	566.4	562.1	1062.7	1055.4	7.6	7.7
July	18.8	1.5	515.5	509.0	1001.0	954.3	7.7	7.7
Augest	19.4	1.8	533.1	527.8	1049.3	1045.5	7.4	7.6
September	20.7	0.85	542.67	575.67	1197.67	1208.33	7.68	7.73
October	22.7	1.75	539	554	1259	1291	7.68	7.69
November	18.2	1.57	574	584	1377	1372	8.32	8.2
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	**180.5**	**12.1**	**4830.2**	**4936.2**	**10325.0**	**10358.3**	**70.1**	**69.6**
Average	**20.1**	**1.3**	**536.7**	**548.5**	**1147.2**	**1150.9**	**7.8**	**7.7**

2014	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	396	442	98	114	37	38	140	134	85	110
March	421.0	413.5	104.0	97.5	39.5	41.5	134.5	127.5	107.5	111.3
April	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
May	414.0	395.4	102.6	97.6	38.3	37.0	129.3	126.3	110.3	107.6
June	391.1	407.6	102.7	98.0	32.7	34.6	129.6	126.9	114.3	115.1
July	339.0	329.0	81.8	78.3	32.8	32.5	136.5	130.5	103.8	103.3
Augest	332.9	327.7	69.6	66.3	38.3	39.1	126.6	121.4	110.0	110.3
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	**2294.0**	**2315.1**	**558.6**	**551.7**	**218.5**	**222.7**	**796.4**	**766.6**	**630.8**	**657.5**
Average	**382.3**	**385.9**	**93.1**	**91.9**	**36.4**	**37.1**	**132.7**	**127.8**	**105.1**	**109.6**

Table (C-66) : The monthly average of chemical and physical parameters which examined for the year (2015) at raw water turbidity 20 NTU for real plant.

2015	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
March	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
April	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	18.0	1.3	663.0	634.0	1390.0	1397.0	7.4	7.5
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	18.0	1.3	663.0	634.0	1390.0	1397.0	7.4	7.5
Average	18.0	1.3	663.0	634.0	1390.0	1397.0	7.4	7.5

2015	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
March	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
April	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	419.0	423.0	121.0	125.0	28.0	26.0	120.0	122.0	127.0	135.0
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	419.0	423.0	121.0	125.0	28.0	26.0	120.0	122.0	127.0	135.0
Average	419.0	423.0	121.0	125.0	28.0	26.0	120.0	122.0	127.0	135.0

Table (C-67) : The monthly average of chemical and physical parameters which examined for the year (2016) at raw water turbidity 20 NTU for real plant.

2016	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
January	19.7	0.8	569	608	1244.5	1254.5	7.30	7.40
February	18.3	1.0	543.7	560.0	1154.3	1153.0	7.6	7.4
March	18.4	1.0	488.2	504.0	1003.5	1007.8	7.6	7.5
April	19.43	1.08	537.8	539.6	1087.8	1094.2	7.71	7.65
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	75.9	3.8	2138.6	2211.6	4490.1	4509.5	30.2	30.0
Average	19.0	1.0	534.7	552.9	1122.5	1127.4	7.5	7.5

2016	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
January	440.5	427.5	119	121	35.5	36	96	99	131	133.5
February	464.3	470.0	116.7	117.3	41.7	41.3	100.0	102.7	121.3	129.3
March	390.8	394.0	92.7	92.7	38.7	39.0	99.0	101.3	124.3	127.2
April	419	426	101	103.5	40	40.75	114.5	111.5	131.75	138.5
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	1714.7	1717.5	429.3	434.5	155.8	157.1	409.5	414.5	508.4	528.5
Average	428.7	429.4	107.3	108.6	39.0	39.3	102.4	103.6	127.1	132.1

Table (C-68) :The monthly average of chemical and physical parameters which examined for the year (2017) at raw water turbidity 20 NTU for real plant.

2017	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
March	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
April	17	0.85	514	510	1119	1114	No Test	No Test
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	17	0.8	608	602	1027	1007	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	34	1.7	1122	1112	2146	2121	No Test	No Test
Average	17	0.83	561	556	1073	1060.5	No Test	No Test

2017	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
March	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
April	412	510	108	105	35	35	96	94	125	129
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	448	440	115	112	39	38	126	124	132	137
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	860	950	223	217	74	73	222	218	257	266
Average	430	475	112	109	37	37	111	109	129	133

Table (C-69) :The monthly average of chemical and physical parameters which examined for the year (2018) at raw water turbidity 20 NTU for real plant.

2018	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	22.7	2.0	468.0	449.0	1012.0	1010.0	No Test	No Test
March	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
April	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	20.66	0.93	553.5	556.38	1107.63	1108.25	No Test	No Test
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	43	2.9	1022	1005	2120	2118	No Test	No Test
Average	22	1.47	511	503	1060	1059.1	No Test	No Test

2018	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
January	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
February	400.0	398.0	100.0	94.0	37.0	40.0	100.0	94.0	142.0	136.0
March	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
April	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
May	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
June	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
November	407.13	405.5	104.13	101.88	35.75	36.13	81	82	124.88	131.5
December	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Sum	807	804	204	196	73	76	181	176	267	268
Average	404	402	102	98	36	38	91	88	133	134

Table (C-70) :The monthly average of chemical and physical parameters which examined for the year (2019) at raw water turbidity 20 NTU for real plant.

2019	1 Raw Turb	2 Clear Turbidity	3 Raw TDS	4 Clear TDS	5 Raw EC	6 Clear EC	7 Raw PH	8 Clear PH
January	20.9	1.729	647.5	647.7	1231.7	1226.3	No Test	No Test
February	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
March	23.4	0.6	639.0	640.0	1278.0	1280.0	No Test	No Test
April	20	1.00	637	635	1271	1269	No Test	No Test
May	20.9	0.7	736.2	725.2	1168.6	1151.0	No Test	No Test
June	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	20.5	3	507	501	805	795	No Test	No Test
November	21.69	1.71	561.75	565.00	889.75	896.5	No Test	No Test
December	19.32	2.05	512.50	514.00	805.67	815.17	No Test	No Test
Sum	147	10.8	4241	4227	7450	7432	No Test	No Test
Average	21	1.54	606	604	1064	1061.8	No Test	No Test

2019	9 Raw Hardness	10 Clear Hardness	11 Raw Ca	12 Clear Ca	13 Raw Mg	14 Clear Mg	15 Raw Alkalinity	16 Clear Alkalinity	17 Raw Chloride	18 Clear Chloride
January	404.3	407	104	103.9	35.1	36	84.6	81.2	134.5	138.6
February	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
March	391.0	395.0	100.0	102.0	34.0	34.0	90.0	92.0	138.0	143.0
April	385	385	94	90	37	38	78	80	136	139
May	400.8	396.4	108.2	106.2	30.4	30.6	131.2	126.8	96.0	97.2
June	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
July	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
Augest	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
September	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test	No Test
October	296	288	71	71	29	27	120	118	76	78
November	333.50	325.5	72.50	69.38	37.125	37.00	128.5	121	102.63	109.5
December	326.00	324.00	73.83	72.17	34.33	34.83	130.67	126	86.67	90.33
Sum	2536	2521	623	614	237	237	763	745	769	795
Average	362	360	89	88	34	34	109	106	110	114

C.6 Calculation of SOR and Area of Plate Settler in Sedimentation Basin

C.6.1 Design Parameters:

Available dimension of sedimentation basin ($R_{outr} = 19m$,R_{inner}=7.5m ,water depth = 4.19m) .

The important parameters used in the design are :detention time ,surface over flow rate and ,horizontal velocity (radial velocity)which are calculated as follows:

A-Conventional sedimentation process at flow rate 1050 m³/hr.

-Detention time (D_t) = $\frac{\forall}{Q}$ = $\frac{((19)^2-(7.5)^2))*4.19*\pi}{1050}$ *60 = 229 min.

- Surface over flow rate = $\frac{Q}{A}$ = $\frac{1050m^3/h}{957m^2}$ = 1.09 m³/m² hr.

- Horizontal velocity (V_h) = $\frac{Q}{2\pi rh}$ = $\frac{1050}{2*3.14*19*4.19}$ = 2.1m /hr.= 0.035 m /min.

B- High Rate Sedimentation at Flow Rate 2625m³/hr.

When increasing the flow rate of 2.5 times as much as the initial flow rate will increase the surface load on the settling basin , and the solution is to use a plate settler in the settling basin .

C.6.2 Area of Plates Settler

Plates angle of inclination(θ)= 60°,(JA Salvato, 2003),(Fadel and Baumann , 1990.

The distance between the outer wall and flocculation wall = 11.5m.

The net diameter of sedimentation tank 36m .

The first frustum has two radius (upper one 36m + lower radius 31.74m).

The oblique height (L) = 4.27m.

The height of cone (H) = 3.7 m.

Area of first plate = π* L (R1+R2).

A_1= 3.14*4.27(31.74+ 36) = 908m^2.

The second frustum has two radius (upper radius 35.8 m + lower radius 31.54 m).

The oblique height (L) = 4.27m.

The height of cone (H) = 3.7 m.

Area of second plate = π* L (R1+R2).

A_2= 3.14*4.27(31.54+ 35.8).

A_2= 903m^2.

Total area of plate settler = (908 +904) = 1811m^2

Check SOR (1.2-4.5) m^3/m^2 hr.

SOR= $\frac{Max.flow\ rate}{Area} = \frac{2625\ m^3}{1811m^2}$ = 1.449 m^3/m^2 hr..(within criteria).

At Flow Rate 2100 m^3/hr.

SOR = $\frac{2100\ m^3/h}{1811m^2}$ = 1.15m^3/m^2 hr.(within criteria).

The spacing between two plates is (20cm).

Therefore can be used area of plate settler =1811m^2 when the flow rate is max.(2625m^3/hr.), and when flow rate = 2100m^3/hr..

As for the flow rate 1575m^3/hr., and 1050 m^3/hr. , they do not need to use the plate settler because they are small flow rate .

الخلاصة

تهدف هذه الدراسة الى تطوير وتحسين كفاءة محطة تنقية المياه في كربلاء وتتضمن خطة البحث تحسين المحطة من الناحيتين الكمية والنوعية بما يضمن تلبية المتطلبات المعيارية والتي سيتم تطويرها ضمن البدائل المحلية المقبولة.

بالاضافة الى التحليل النظري تناولت الدراسة سلسلة من نتائج الاختبارات واالدراسات في ضوء البيانات المتوفرة في المحطة للفترة ٢٠١٤-٢٠١٩ . تم بناء نموذج فيزياوي لمحاكاة محطة معالجة المياه التقليدية باستخدام اربعة قيم لمعدل التدفق وهي (٠,٤٧٥ ، ٠,٧١٢ ، ٠,٩٥ و ١,١٨ م٣ / ساعة) باستخدام مستويات مختلفة من العكارة.

يتكون هذا النموذج من وحدات التخثير ،التلبيد والترسيب الاساسية واستخدم تطبيق التشابه الديناميكي بين المحطة كنموذج والمحطة الاصلية بما يحقق الحصول على نفس رقم فرود بينهما.

تم استخدام تربة البنتونايت وتربة النهر لتحضير تراكيز العكارة المطلوبة .جرى اعتماد وحدة ترسيب عالية السرعة في المحطة التجريبية باستخدام صفائح الترسيب ومقارنتها مع المحطة الاصلية .اضيف مرشح وسائط مزدوج مع الانثراسايت بالاضافة الى وحدة ترشيح مفردة موجودة في المحطة بنفس الابعاد الاصلية حيث عمق الطبقة 0.7 متر مقسمة الى ٠,٣٥ متر رمل و ٠,٣٥ متر انثراسايت.

تم وضع احد مرشحات الكاربون النشط بعد الوسط الخاص بالمرشح المنفرد وتم وضع الاخر بعد الوسط الخاص بالمرشح المزدوج . لقد وجد ان افضل تدرج سرعة في خزان الخلط السريع هو $(G_{rapid}) \ge 750\ \bar{s}^{1}$ وفي حوض التلبيد $15\ s^{-1} < G_{flocc} < 60\ \bar{s}^{1}$ أما قيم $(Gt_{flocc}) =$.[$(15*10^4)$- (10^4)]. لقد وجد ايضا بأن أفضل كفاءة ازالة في حوض الترسيب باستخدام الصفائح يتراوح بين ٩٤,٠٧% الى ٧٢,٠٧% باستخدام البنتونايت و تربة النهر على التوالي اي ان كفاءة الازالة قد تحسنت بمقدار ٢٣% عند استخدام البنتونايت. كانت افضل كفاءة ازالة باستخدام البنتونايت وتربة النهر في حوض الترسيب ٧٤,٠٢% و ٦٦,٢٨% وبدون استخدام الصفائح المائلة على التوالي

لقد تحسنت كفاءة الازالة حوالي ١١% عند استخدام الصفائح وكانت كفاءة الازالة باستخدام الصفائح المائلة هي الافضل خاصة عند مضاعفة معدل التدفق بمقدار ١,٥ – ٢,٥ مرة كما وجد ان افضل الحالات عندما يكون معدل التدفق ٢١٠٠ م٣ بالساعة والعكارة عند 50 NTU باستخدام تربة النهر واللوح المائل ووسط ترشيح مزدوج مع مرشح كاربون نشط حيث كانت كفاءة الازالة

٩٨,٢٣% بينما كانت كفاءة الازالة ٩٩,٢٧% عند استخدام البنتونايت بدون الواح مائلة وباستخدام مرشح الكاربون النشط ووسط الترشيح المزدوج عند عكارة مقدارها 200NTU ومعدل تدفق 1050 م^{٣} بالساعة .

نشير هنا الى ان استخدام البنتونايت في زيادة عكارة الماء له تأثير ايجابي في زيادة كفاءة الازالة . اخيرا فان هناك زيادة معنوية تتراوح بين ١٥٠% الى ٢٥٠% في انتاج المحطة وكفاءتها اضافة الى الحصول على تكلفة اقتصادية معتدلة.

تطوير محطة تجريبية لتقييم محطة معالجة مياه كربلاء ، العراق

اطروحة مقدمة الى قسم الهندسة المدنية

كلية الهندسة في جامعة كربلاء

كجزء من متطلبات نيل شهادة الماجستير في الهندسة الصحية

من قبل

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اذار ٢٠٢٢