



## VERIFICATION OF PROCESS DESIGN OF AN EXTENDED AERATION FOR AL-SHURQAT WASTEWATER TREATMENT PLANT

Dr. Mohammed Saiwan Shamkhi\*

Lecturer, Civil Engineering Department, University of Wasit, Wasit, Iraq.

(Received:27/04/2015 ; Accepted:03/11/2015)

**Abstract:** The aim of this paper is to verify the treatment process design of Shourqat wastewater treatment plant (WWTP), taking into account the high yearly average temperature that occurs in Iraqi territory. An experimental observed yield ( $Y_{obs}$ ) has been detected in the laboratory and verified at different temperatures by application of GPS-X model. This paper described the equations of the process design for WWTP. Sludge age ( $\theta_c$ ) has been correlated to the observed yield ( $Y_{obs}$ ) and to the mixed liquor suspended solid (MLSS) showing that a valid correspondence has been found between the  $\theta_c$  calculate by dividing the total biomass present in the oxidation basin by the total biomass wasted and the  $\theta_c$  calculated with taking into account  $Y$ , MLSS and BOD removed. An experimental value of the observed yield ( $Y_{obs}$ ) has been detected, with high values ranging from 0.56 to 0.75.

**Keywords:** Biomass Yield, Model, Sludge Residence Time, Temperature.

### التحقق من تصميم عملية التهوية المطولة لمحطة معالجة مياه مجاري الشرقاط

**الخلاصة:** يهدف البحث الى التحقق من تصاميم محطة معالجة مياه المجاري في الشرقاط أخذين بنظر الاعتبار التغيرات الكبيرة في معدلات درجات الحرارة خلال السنة. تم قياس  $Y_{obs}$  مختبريا واجري التحقق منه في درجات الحرارة المختلفة بأستخدام برنامج GPX-X. كذلك استعرض البحث معادلات تصميم محطة المعالجة. في هذا البحث ظهر تطابق وثيق بين عمر الحمأة  $\theta_c$  المحسوب من تقسيم MLSS على  $Q_w$  وبين  $\theta_c$  المحسوب مع الأخذ بنظر الاعتبار MLSS و  $Y$  و BOD المزال. قيمة  $Y_{obs}$  التي تم الحصول عليها مختبريا كانت تتراوح بين 0.56 الى 0.75.

## 1. Introduction

The treatment of wastewater using the biomass as an agent to remove the colloidal and dissolved solids (mainly organic) termed biological treatment and it is regarded the most important unit in wastewater treatment plants [7.1], [7.2]. The most significant variable affecting the design of all wastewater treatment units are population, area served by network

\*[alattabi65@yahoo.co.uk](mailto:alattabi65@yahoo.co.uk)

and infiltration rate. Also, it is found that the temperature, sludge, total suspended solid (TSS) and biological oxygen demand (BOD) has a significant effects on the volume of the biological unit for (conventional activated sludge, extended aeration, oxidation ditches, and facultative ponds). Absolute quantity of total air required for aeration process for plants that need aeration, quantity of gas produced, volume of settling tanks, and volume of sludge conditioning facilities [7.3]. Controlling the eutrophication rate can be accomplished by controlling the nutrient load into the receiving water body. Nitrogen and phosphorus are both nutrients needed by plants. With a typical influent ammonia concentration, the facility must be capable of nitrification, converting nearly all of the ammonia to nitrate. Then,  $\text{NO}_3$  being reduced back to  $\text{NO}_2$  and then to  $\text{N}_2$  by the denitrification process [7.4]. Controlling phosphorus loading can be achieved by the mixed liquor suspended solid (MLSS) cycles through anaerobic followed by aerobic conditions where a type of bacteria (*Acinetobacter*) begins to accumulate in the biomass that uses phosphorous as an energy storage mechanism [7.5]. Magnaye et al., 2009[7.6], state that "for simultaneous removal of nitrogen and COD, it is more attractive to use combined anaerobic and aerobic systems".

The Iraqi rivers protection act number 25 for 1967 have been adopted and used as guidelines for quality of treated municipal wastewater to surface water courses in this paper. The most significant parameter limits (maximum concentrations) applicable to the wastewaters is: 20 mg/l for  $\text{BOD}_5$ , 30 mg/l for TSS, 10 mg/l for ammonia ( $\text{NH}_4\text{-N}$ ), and 50 mg/l for total nitrogen (N) should be respected by application of model and experimental work.

This paper are also described the standards and criteria adopted for the purpose of developing preliminary designs of the project facilities and establishing the bases for evaluating the various project alternatives.

The conventional activated sludge process and extended aeration process were met the established effluent discharge criteria to Tigris river. Both processes involve the use of the activated sludge process tailored to provide nitrification and denitrification for effective removal of  $\text{BOD}_5$ , suspended solids, ammonia-nitrogen and total nitrogen from the influent wastewater. The effluent standards for the category discharge to streams were selected from Iraqi National Standardsthe relevant literature on the study subject and the proposed approach or solution. The introduction should be general enough to attract a reader's attention from a broad range of scientific disciplines.This template, created in MS Word 2003/2007 provides authors with most of the formatting specifications needed for preparing their papers. Margins, line spacing, and tpestyles are built-in; examples of the type styles are providedthroughout this document.

### ***1.1. Description of case study***

The City of Shurqat is a part of Salah al-Deen Governorate. It is located approximately 125 km north of Salah Al-Deen city centre. The population figures for Al-Shurqat district are based on the 2012 population data obtained from the central organization of statistics and information technology of the ministry of planning and development. The planning timeframe used in this study is 25 years, through the year 2040. A growth rate of 3.0% per year has been

adopted. Based on the above the city of Al-Shurqat population in the year of 2012 population of 52000 capita and is expected to grow to 118972 capita by year of 2040.

Due to the relatively consistent topographic and geographic features in Shurqat, climatic conditions do not vary considerably within the city. The average annual temperature in Shurqat is 21.5 °C. The winter temperatures drop to an average of 8 °C in January, while the summer temperatures in July and August reach an average of 31.9 °C.

### 1.2. Objectives of the paper are:

- to investigate the biomass yield coefficient;
- to comparison the experimental results of observed yield ( $Y_{obs}$ ) with the value used in software output;
- to adequately export the results of the experimentation in full scale design.

## 2. Basic design data of preliminary designs Al-Shourqat WWTP project

The construction of wastewater treatment plant required huge fund investment, so, the design was divide the project into two phases. Phase I represent the design of project to serve the population for the period from the year of 2012 up to year of 2028. While phase II represent the upgrading the design of phase I to serve the population for the period from the year of 2028 up to year of 2040. The Basic design data of preliminary designs Al-Shourqat WWTP project as below:

Equivalent inhabitants	83445 capita (Phase I) and 118972 (Phase II)
Average dry weather flow	18358 m <sup>3</sup> /d (Phase I) and 26173 m <sup>3</sup> /d (Phase II)
Peak dry weather flow	36215m <sup>3</sup> /d (Phase I) and 48539 m <sup>3</sup> /d (Phase II)
Minimum dry weather flow	8678 m <sup>3</sup> /d (Phase I) and 12848 m <sup>3</sup> /d (Phase II)
BOD <sub>5</sub> loading:	5841 kg/d (Phase I) and 8328 kg/d (Phase II)
Wastewater Temperature: Design	20°C
Min	15°C
Max	35°C

The treated effluent limits according Iraqi rivers protection act number 25 for 1967 as below:

TSS	< 30 mg/L
COD	< 60 mg/L
BOD <sub>5</sub>	< 20 mg/L
TN	< 50 mg/L
NH <sub>3</sub> -N	< 10 mg/L
NO <sub>3</sub> -N	< 25 mg/L
NO <sub>2</sub> -N	Nil
TP	< 3 mg/L
pH	6.0-7.5

### 3. Process description

The process has been studied and implemented according to principal international literature in the specific field [7.7], [7.8]. The obtained plant design has been also verified and optimized by implementing the proposed plant layout in the GPS-X 6.1 modelling and simulation software (Hydromantis). The latter has been used to simulate and verify the process performance in conditions of minimum temperature, peak loading and typical fluctuations of these parameters.

### 4. Process verification and simulations with GPS-X 6.1

Fig.1 shows the assumed daily variations of flow which have been simulated according to typical flow rate profiles for municipal sewers [7.4], respecting the average, peak and minimum design flow rate. Variations for 24 hours of typical operation (Phase II).

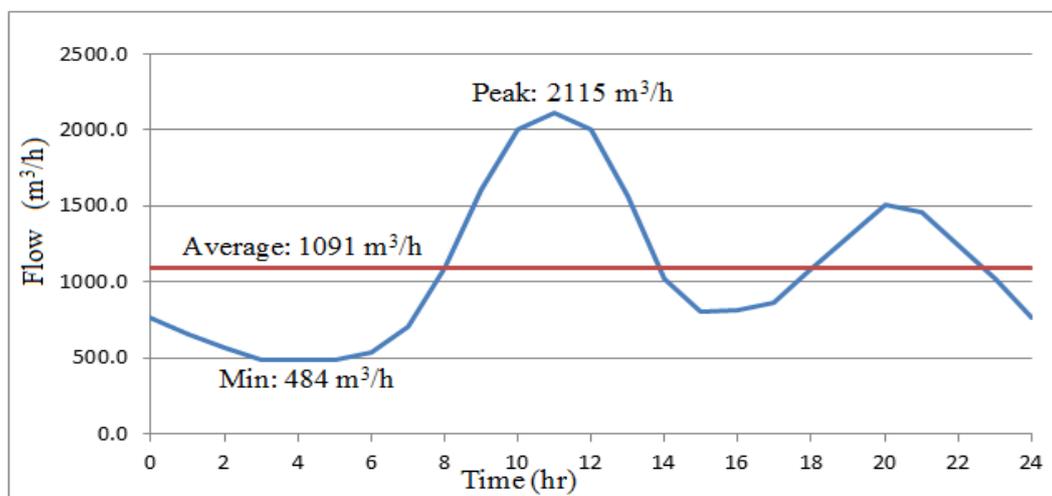


Figure 1. Daily variation of flow (phase II).

The yearly variations of sewage and ambient temperature have been simulated according to typical profiles for municipal sewers, as in (Fig. 2) for one year of operation (Phase II).

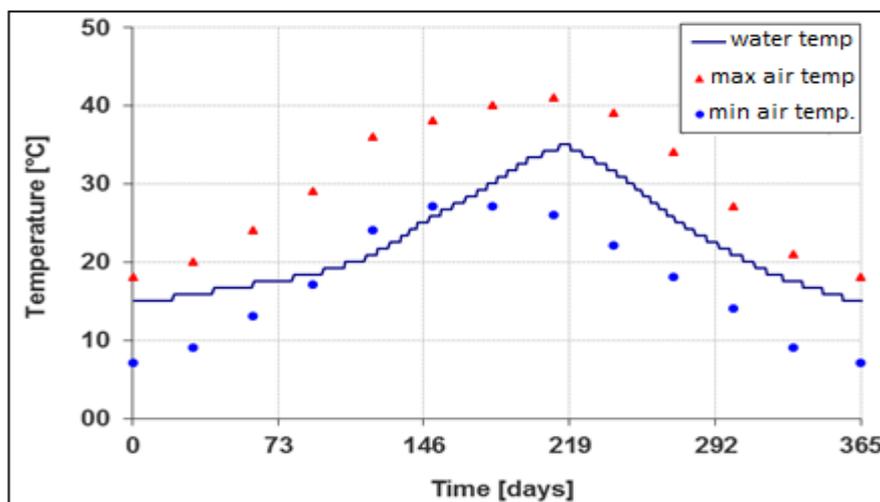


Figure 2 Yearly variation of ambient and sewage temperatures (phase II).

#### 4.1. Design of Equalization Basin

During the day the flow exceeding the lifting capacity of the inlet screw pumping station will be conveyed to the equalization basin by means of an overflow weir. With missing of inflow data required to draw inflow mass diagram, the capacity of equalization tank will take  $0.2 Q_{ave}$ , so:  $0.2 \times Q_{ave\ phase II} = 5235\ m^3/d$  [7.1].

$$\text{Capacity of equalization tank} = 0.2 \times 26173 = 5235\text{m}^3$$

use 1-rectangular tank with B:L= 2:1

and Total Depth=Water depth+ Free board=5m

$$\text{the surface area of tank} = (5235 \div 4.5)=1308.75\text{m}^2$$

$$\therefore 2B \times B = 1308.75 \rightarrow B=21\text{ m} \rightarrow L= 2B=42\text{ m}$$

Use coarse air diffusers to agitate the waste water in tanks

The capacity of the pump is;

$$Q_{\text{pump}}=5235 \div 24=218.13\text{m}^3/\text{hr use } 220\text{m}^3/\text{hr}.$$

No. 1 duty + 1 standby submergible pumps will be installed inside the equalization basin and they will discharge in the channel after the inlet screw pumping station.

With  $Q_{\text{pump}}=220\text{ m}^3/\text{h}$  there is no overflow from the equalization basin and pump also stops for few hours a day (equalization basin emptied).Therefore  $Q_{\text{pump}}=220\text{ m}^3/\text{h}$  is definitively selected for the pump installed in the equalization tank.The presence of the equalization basin has reduced the peak flow going to the biological tank to  $1355\text{ m}^3/\text{h}$  from the  $2115\text{ m}^3/\text{h}$  of the peak influent flow.

#### 4.2. Secondary Treatment Units

##### 4.2.1. Design Aeration tanks

The combined process oxidizes a high proportion of the influent organics relative to the  $\text{NH}_3\text{-N}$  concentration. Due to the high yearly temperature that occurs in Salah Al-Deen governorate, it has been detected experimentally a high observed yield ( $Y_{\text{obs}}$ ) that measures the mass of new cell produced per unit of substrate removed by microorganisms in the reactor because the value of yield depends on the nature of substrate, the type of organisms and the temperature [7.1]. The observed yield ( $Y_{\text{obs}}$ ) ranges from 0.56 to 0.75 and verified by GPS-X 6.1 model. This result has been tested for the samples of wastewater were taken from influent and the reactor in laboratory.

However a conservative option has been selected for taking in the account the possibility of winter-low temperature because higher molecular weight compounds are more recalcitrant and will take several hours to be degraded and removed. For this reason it should be more conservative to consider an  $\text{O}_2$  excess as derived from the following equation [7.9]:

$$\text{O}_2 = \left[ (1.08 \times \text{BOD}_{5\text{removed}}) + (4.57 \times \text{NH}_4^+_{\text{removed}}) \right] \times Q_{\text{ave}} \times 10^{-3} \quad (1)$$

Table 1 gives the calculation of biological tank design which was designed by using the following equations;

$$V = \frac{Q_{\text{ave}} \text{BOD}_{5\text{removed}}}{\text{MLVSS}^F/M} \quad (2)$$

$$HRT = \frac{V}{Q} \quad (3)$$

$$P_{X_{VSS}} = Y_{obs} \times Q_{ave} \times BOD_{5_{removed}} \quad (4)$$

$$P_{X_{SS}} = \frac{P_{X_{VSS}}}{0.8} \quad (5)$$

$$Q_w = \frac{P_{X_{VSS}}}{0.008 \gamma_{sludge}} \quad (6)$$

$$Q_r = Q_{ave} \quad (7)$$

$$\theta_c = \frac{V}{Q_w} \times \frac{MLSS}{MLSS_R} \quad (8)$$

$$MLSS_R = 0.8 \times MLSS \quad (9)$$

Where;

- $V$ = volume of biological tank ( $m^3$ ) ;  $Q_{ave}$ = average discharge ( $m^3/d$ )
- $MLVSS$ = mixed liquor suspended solid ( $mg/l$ ) ;  $F/M$ = food microorganism ratio
- $HRT$ = hydraulic retention time (hr) ;  $P_{X_{VSS}}$ = the sludge produce ( $Kg/d$ )
- $Y_{obs}$ = observed yield ;  $P_{X_{SS}}$ =sludge produce in term of TSS( $Kg/d$ )
- $Q_w$  = sludge wasting discharge ( $\frac{m^3}{d}$ ) ;  $Q_r$  = return sludge discharge ( $\frac{m^3}{d}$ )
- $\theta_c$ = sludge retention time (d) ;  $MLSS_R$ = return mixed liquor suspended solid ( $mg/l$ )
- $\gamma_{sludge}$ = sludge density ( $kg/m^3$ )

Table 1. Biological basin process parameters

Parameter	Unit	Criteria	Phase I (2028)	Phase II (2040)
$Q_{ave}$	$m^3/d$	----	18358	26173
MLSS (assumed)	$mg/l$	3000-6000	4000	4000
MLVSS (equation 9)	$mg/l$		3200	3200
F/M (assumed)	$kgBOD_5/kgMLSS.d$	0.05-0.15	0.1	0.1
$V$ (equation 2)	$m^3$	----	17211	24537
HRT (equation 3)	h	12-36	22.50	22.50
$Y_{obs}$ (laboratory test results)	$kgVSS/kg(BOD_5)_r$	0.56- 0.75	0.6	0.6
$P_{X_{VSS}}$ (equation 4)	$kg/d$	---	3304	4711
$P_{X_{SS}}$ (equation 5)	$kg/d$	---	4130.55	5888.93
$Q_w$ (equation 6)	$m^3/d$	Use 0.8% solid concentration	516	736

$Q_r$	(equation 7)	$m^3/d$	18358	26173
$O_2$	(equation 1)	$kgO_2/d$	9723	13863
		$kgO_2/h$	405	578
$O_2$ supplied		$kg/h$	500	700
$\theta_c$ (equation 8)		d	10 – 40	17
Side water depth		m	3.40	3.40
Surface area of tank		$m^2$	5062	7217
$O_2$ required for each cell		$kg O_2/h$	42	39
Aerator of 22 kwatt		N.	12	6

With return activated sludge equal to 100% of average flow and wasted activated sludge equal to 578  $m^3/d$ , the concentration of MLSS reaches an of approximate value of 4000 mg/l as per preliminary design. Concentration of sludge pumped from the bottom of the secondary clarifier reaches an average value of 7800 mg/l in the preliminary design.

#### 4.2.2. Design Secondary clarifier

The secondary settling tank (clarifier) is an integral part of the activated sludge process. The main purpose of providing the secondary settling tank is to separate the large volume of suspended solid (MLSS) coming from the aeration tank and to obtain a very clear stable effluent having low concentration of BOD and suspended solid. Table 2 explain the calculation of secondary sedimentation tank design.

The perforated inlet pipe flow to the settling tank of diameter 0.50 m and 2 m depth having 20 opening of 20 cm. The horizontal velocity of flow inside clarifier ( $V_h$ ) ranges from 0.15 to 0.6 m/sec.

Table 2 Secondary sedimentation parameters

Parameter	Unit	Phase (2028)	Phase I (2040)	II
$Q_{ave}$	$m^3/d$	18358	26173	
$Q_{peak}$	$m^3/d$	37920	50760	
$Q_r$	$m^3/d$	18358	26173	
$Q_w$	$m^3/d$	516	736	
Overflow rate for $Q_{ave}$	$m^3/m^2*d$	16	15.24	
Surface area; $A_s = Q_{ave} / \text{overflow rate}$	$m^2$	1147.38	1717.39	
Tank with diameter of 28 m	no	2	1	
Depth	m	3,5	3,5	
Tank volume	$m^3$	2004		
Check detention time	h	5	5,5	
Check solid loading for $Q_{ave}$ $SLR = \frac{(Q_{ave} + Q_r)MLSS}{A_s}$ for each tank	$kg/m^2.h$	5.33	5.08	
Check weir loading	$m^3/m*d$	108	103	

### 4.3. Sludge line

Sludge produced in the secondary settling tank is large in volume due to high water content (0.2-0.12%) and contain more complex matter. The objective of the sludge is treatment to reduce the water content of the sludge and stabilize the organic content of the sludge.

A 12 m diameter sludge thickener for the phase 1 and two 12 m diameter sludge thickeners for phase II with a mechanical scraper will be installed for sludge thickening from 0.5-1 % dewatering to 2-4 % dewatering . 1 duty +1stand by mono pumps for phase(I) and 2 duty +1 stand for phase (II) will be provided for the sludge pumping. Pump station flow will range from 10 to25 m<sup>3</sup>/h.

One sludge belt press for phase( I) and 2 duty for phase ( II) having a belt width equal to 2.5 m and capacity 375 kg SS/h will be installed in the dewatering building.

### 4.4. Drying beds

Drying beds are the option for sludge drying in emergency conditions or during maintenance of belt press. The emergency drying beds have been sized as follows:

- Sludge bed depth: 300 mm;
- Width of bed: 8 m;
- Length of bed: 30 m;
- Surface of bed cell: 240 m<sup>2</sup>;
- Number of bed cell: 2 for Phase I and 1 more for Phase (II).

Fig. 3 reports the mass balance flow calculated for flow phase (I).

## 5. Results analysis

Fig. 4 show the variation of principal parameters concentration given by GPS-X 6.1. The GPS-X 6.1 simulations have been run considering the simultaneous operation of 18 surface aerators, adsorbed power 17 kW, oxygen transfer rate 39.1 kgO<sub>2</sub>/h, for a total oxygen transfer rate of 938.4 kgO<sub>2</sub>/h. The installed power was 22 kW/each, with suitable safety factor respect to the adsorbed power. On the basis of the domestic wastewater characteristics that inflow at Al-Shourqat WWTP, the characteristics of effluent for the proposed extended aeration process were according the Iraqi limits for treated wastewater which discharge to the rivers.

Results from simulation shows that the oxygen requirement was 703.8 kgO<sub>2</sub>/h (standard oxygen requirement) including the oxygen for BOD removal, endogenous respiration and nitrification and the savings due to the denitrification step. It has been considered also the non-biodegradable volatile suspended solid VSS fraction eventually present in the influent so that in any case the discharge limits are respected.

An experimental value of observed yield ( $Y_{obs}$ ) has been detected as ranges from 0.56 to 0.75. Sludge age ( $\theta_c$ ) has been correlated to the observed yield ( $Y_{obs}$ ) and to the mixed liquor suspended solid (MLSS) showing that a valid correspondence has been found between the  $\theta_c$  calculate by dividing the total biomass present in the oxidation basin by the total biomass wasted and the  $\theta_c$  calculated with taking into account  $Y$ , MLSS and BOD removed. It has been found that the  $\theta_c$  with criteria that promote the extended aeration process.

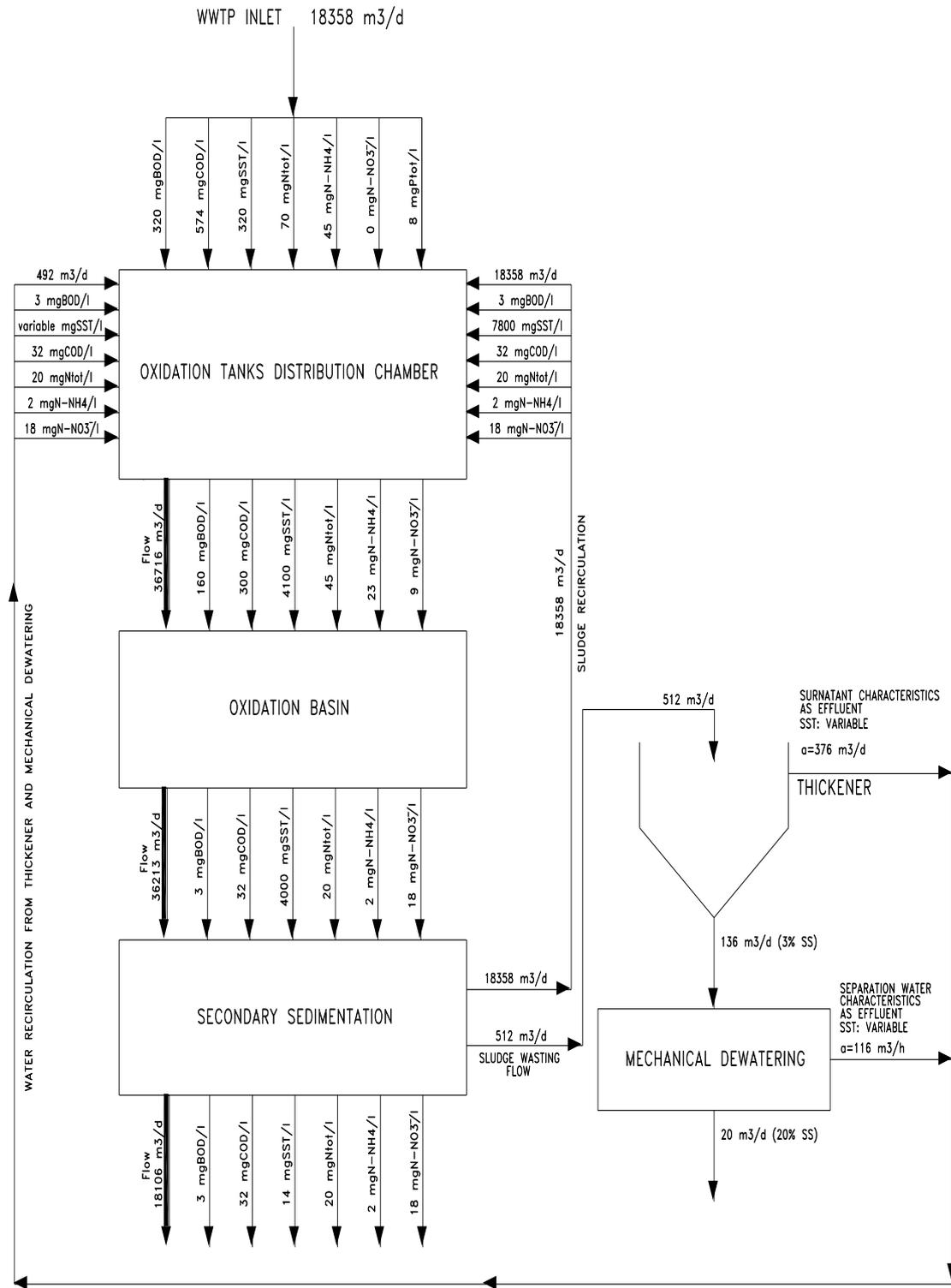


Figure3 Mass balance flow (Phase I)

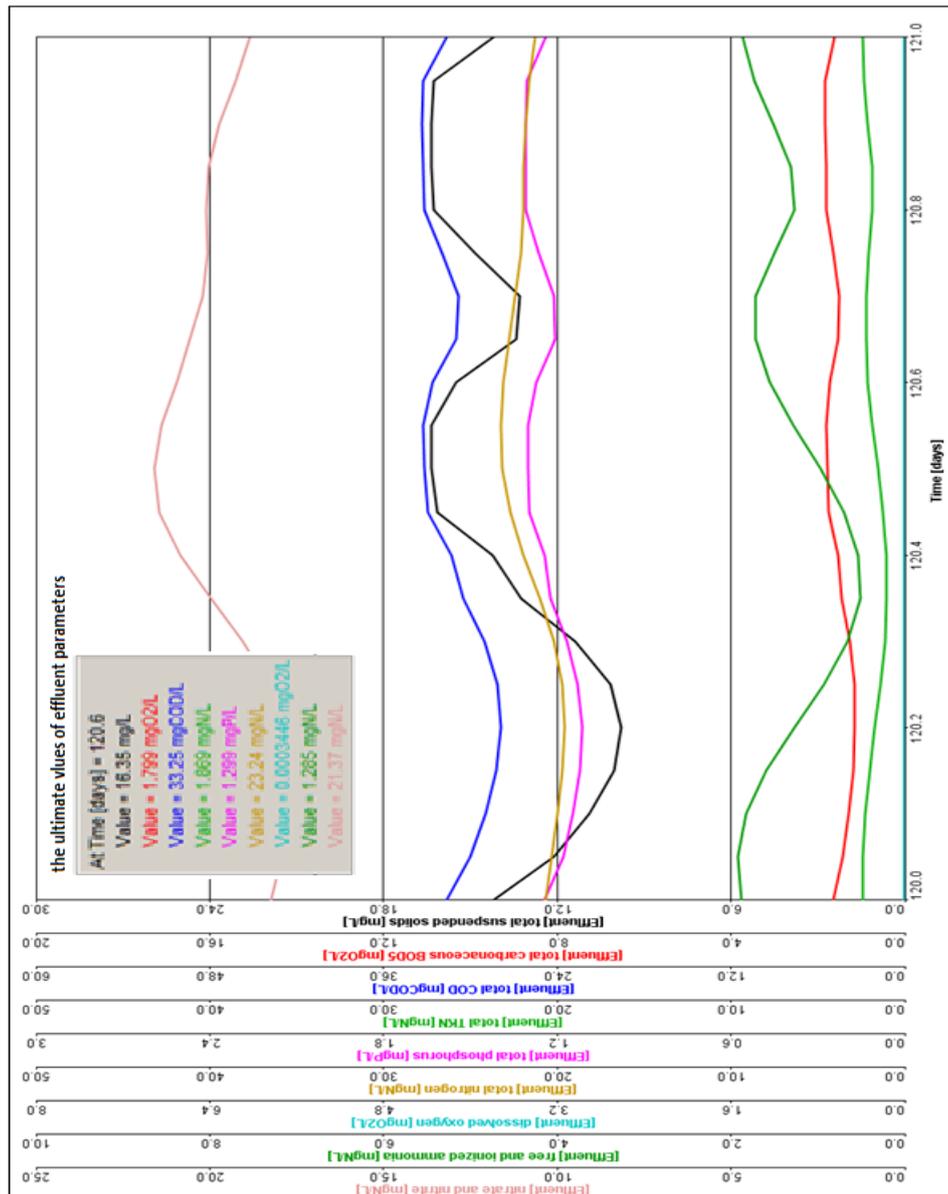


Figure 4 Principal parameters effluent values (Phase II - T=35°C)

### 6. Conclusions

At all different sewage temperatures, the variation of principal parameters concentration of effluent wastewater given by GPS-X 6.1 model were within the standard limits. So, the design of biological tank for Al-Shurqat WWTP is still worked during with high efficiency. The high yearly temperature that occurs in Salah Al-Deen governorate helped to get a high observed yield ( $Y_{obs}$ ) as it was detected experimentally with values ranging from 0.56 to 0.75.

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