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Relationship between the Sludge Settling Characteristics and the Parameters of the Activated Sludge System

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RELATIONSHIP BETWEEN THE SLUDGE SETTLING CHARACTERISTICS AND THE PARAMETERS OF THE ACTIVATED SLUDGE SYSTEM

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To my wife Karina and my son Alejandro
You are the reason and inspiration of everything in my life

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ABSTRACT

The activated sludge process is one of the most commonly employed domestic and industrial waste treatment process. Different types of mathematical models have been proposed for design and operation of this process, most of which do not consider the relationship between the sludge settling characteristics and the aeration unit performance. This project studies the validity of a model developed by La Motta (2004b) which links the operating parameters of an activated sludge system and the classical limiting flux sludge settling theory. Favorable results were obtained demonstrating that the model predicts very similar values of the parameters of the system in comparison with the parameters observed in an activated sludge pilot plant that is located within installations of the Marrero Wastewater Treatment Plant, New Orleans, Louisiana. This research also demonstrated that the model is a helpful tool for the design and operation of an activated sludge system.

CHAPTER I

1.1. Background

The removal of suspended and soluble organic constituents measured as chemical oxygen demand (COD) or biochemical oxygen demand (BOD) in the incoming liquid stream is the primary purpose of wastewater treatment. According to Levine et al. (1985, 1991), a complex mixture of particles and soluble substance are the contaminants that must be removed from the wastewater. This organic and inorganic constituents range in size from less than 0.001 µm to over 100 µm, and represent the major fraction of the organic material in particulate form in municipal wastewaters.

Biological treatment is the most common method to treat municipal wastewaters. Different systems have been developed within this century based on three principal types: attached growth, suspended growth and ponds (WEF, 1998). The following table shows the typical aerobic process applications for suspended and attached growth biological treatment processes.

Table 1.1. Aerobic Biological Treatment Processes Used for Wastewater Treatment (Metcalf & Eddy 2003)

Type	Common name	Use
Suspended growth	Activated –Sludge processes	Carbonaceous BOD removal, nitrification
	Aerated lagoons	Carbonaceous BOD removal, nitrification
	Aerobic digestion	Stabilization, Carbonaceous BOD removal
Attached growth	Trickling filters	Carbonaceous BOD removal, nitrification
	Rotating biological contactors	Carbonaceous BOD removal, nitrification
	Packed-bed reactors	Carbonaceous BOD removal, nitrification
Hybrid (combined) suspended and attached growth processes	Trickling filter/activated sludge	Carbonaceous BOD removal, nitrification

Most suspended growth processes used in municipal and industrial wastewater treatment are operated under aerobic conditions. The activated sludge process is one of the most commonly employed domestic and industrial waste treatment process. This process was so named because it involves the production of an activated mass of microorganisms capable of stabilizing a waste under aerobic conditions. An activated sludge treatment process consists basically of three components: (1) a reactor where the microorganisms remain in suspension and are aerated, (2) a sedimentation tank for liquid-solid separation, and (3) a recycle system for returning solids removed by the sedimentation tank to the aeration basin.

Different types of mathematical models have been proposed for design and operation of the activated sludge process. The International Association on Water Pollution Research and Control (IAWPRC) established a Task Group for the development of these mathematical models, and as a result the Activated Sludge Models Nº1 (ASM1). ASM2, ASM2d and ASM3 have been produced. The main idea of these models is to provide a minimum complexity for the design and operation of this type of system. However, these models do not take into consideration the sludge settling characteristics which play an important role in the design and operation of an activated sludge system.

Commonly, the operating parameters of an activated sludge system are often selected using recommendations presented by the literature. Plant operators generally control the system by trial and error without considering the sludge settling properties. This present investigation attempts to study the relationship between the activated sludge operating parameters and the sludge settling characteristics using a model developed by La Motta (2004) and Homes, and a computer software that solves the respective equations (La Motta called the UNO 1D activated sludge model).

1.2. Objectives and Scope

The main objective of this research is to determine the relationship between the sludge settling characteristics and the parameters of the operating activated sludge system. This research was conducted using a wastewater treatment pilot plant and the UNO 1D Activated sludge model. The outcome of this research could offer the wastewater treatment practitioners a tool for design and operation of activated sludge systems, taking into consideration all of the important parameters and characteristics of the wastewater to maximize the quality of the effluent and minimize cost.

The specific objectives of this project are the following:

- Test the validity of the UNO 1D Activated Sludge Model
- Determine a relationship between the MLSS concentration and the empirical settling parameter *n*

This project was carried out at a wastewater treatment pilot plant located within the UWMRC facility at the Marrero Municipal Wastewater Treatment Plant. The pilot plant unit was fed municipal wastewater from the Marrero grit chamber splitter box.

CHAPTER II

2. LITERATURE REVIEW

2.1. Activated Sludge Process

The activated sludge is an aerobic suspended growth process in which microorganisms are grown in a variety of bioreactor configurations for the purpose of removing soluble and suspended organic matter. It is one of the most commonly employed domestic and industrial waste treatment processes. The process began to be worked by Dr. Angus Smith (1800s), who investigates the aeration of wastewater in tanks and the hurrying of the oxidation of the organic matter. However, the investigation during that time does not provide the awaited result until Arden and Lockett in 1914 found that the aerated sludge played a significant part in the results. Since Arden and Lockett's invention, the activated sludge process has become the most important technology for wastewater treatment.

This system is composed by four different elements: (1) a reactor which is responsible for converting soluble and particulate organic matter from the influent waste stream into live biomass; (2) a sedimentation tank which is in charge of the liquid solid separation; (3) a recycling line that returns sludge from the sedimentation tank into the reactor; and (4) the sludge wasting line.

Engineering innovation, technological advances in equipment, and better understanding of microbiological processes have resulted in different configurations of the activated sludge process (Mecalf & Eddy, 2003).

According to Pavoni et al. (1972), the activated sludge system inherently relies on two independent characteristics for the production of an acceptable effluent. The first and the most

important is the complete assimilation of the suspended and colloidal organic material by the active mass of microorganisms to a final end product of carbon dioxide, water, and inert material. This initial phase of the activated sludge process is commonly referred to as substrate utilization, and deals primarily with active synthesis of microbial mass. The second phase, and ultimately the most significant in the development of a high-quality effluent, is the flocculation of the microorganisms and other suspended or colloidal components into a readily settleable mass so that a clear low biochemical oxygen demand end product may be obtained.

2.2. Modeling of activated Sludge Processes

Modeling of activated sludge processes has become a common part of the design and operation of wastewater treatment plants. The International Association on Water Pollution Research and Control (IAWPRC), established in 1982 a task group of mathematical modeling design and operation of activated sludge processes (Henze 2000). The aim of this task group was to create a common platform that could be used for future development of models for nitrogen removal activated sludge processes. The result was the Activated Sludge Model Nº 1. This model was well received and has been widely used as a basis for further model development (Henze 2000).

Although the objective of the task group was to develop models with a minimum of complexity, these models have grown more complex over the years from ASM1, including nitrogen removal processes, to ASM2, including biological phosphorus removal and to ASM2d including denitrifying phosphorus accumulating organisms. Also, in 1998, the task group decided to develop a new modeling platform, the ASM3, in order to create a tool for use in the next generation of activated sludge models.

The Activated Sludge Model Nº2 (ASM2) is an extension of the Activated Sludge Model Nº1 (ASM1). ASM2 is more complex and includes many more components which are required in order to characterize the wastewater as well as the activated sludge. Additional biological processes are included, primary in order to deal with biological phosphorus removal. The most significant change from ASM1 to ASM2 is the fact that the biomass now has cell internal structure.

The Activated Sludge Model Nº 2d is a minor extension of ASM2. It includes two additional processes to account for the fact that phosphorous accumulation organisms (PAOs) can use cell internal organic storage products for denitrification. Whereas ASM2 assumes PAOs to grow only under aerobic conditions, ASM2d includes denitrifying PAOs. Whereas ASM1 was based entirely on COD for all particulate organic material, as well as the total concentration of the activated sludge, ASM2 includes poly-phosphates, a fraction of the activated sludge which is of prime importance for the activated sludge system, but which does not exert any COD.

Mathematical models related to ASM1 are currently implemented in various computer codes for the simulation of the behavior of activated sludge system treating municipal wastewater of mainly domestic origin. With over ten years of experience with the application of ASM1, some defects of this model have become apparent. For example, it does not include kinetic expressions that can deal with nitrogen and alkalinity limitations of heterotrophic organisms, the biodegradable soluble and particulate organic nitrogen cannot be easily measured, it is difficult to differentiate inert from particulate organic material in reality, it does not directly predict the frequently measured mixed liquor suspended concentration, and so on. Considering all these defects and the advance in experimental evidence on storage of organic compounds, the task group has proposed the Activated Sludge Nº 3 (ASM3) (Gujer et al., 1999) which should

correct for these defects and which could become a new standard for future modeling. Despite all these improvements, ASM3 still does not link the settling behavior to the aerator performance.

According to Ekama et al., (1997), settling tank mathematical models can be classified by their spatial resolution. There are very simple two-cell models and complex multi-cell three dimensional (3-D) models. In addition, the models can simulate steady-state or non-steady conditions in the settling tank

One dimensional models (1D) are based on the flux theory. It is assumed that in clarifiers, the profiles of horizontal gradients are uniform and that horizontal gradients in concentration are negligible. Consequently, only the processes in the vertical dimension are modeled. The resulting idealized settling cylinder is treated as a continuous flow reactor. Figure 2.1 illustrate the flow scheme.

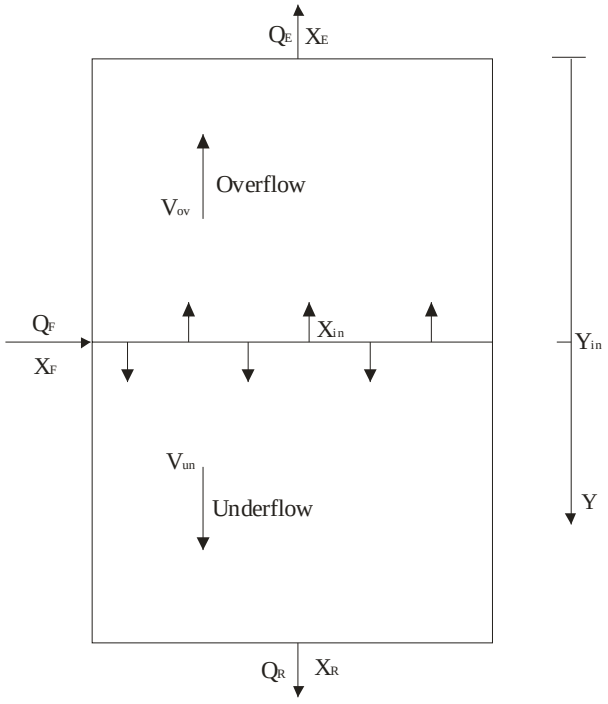

Figure 2.1 Flow Scheme of the one-dimensional continuous-flow settling tank approach. (Ekama et al., 1997)

In figure 2.1:

QE, F, R = flow rate for feed, effluent and recycle lines, respectively

XE, F, R = SS concentration for feed, effluent and recycle lines, respectively

Vov = Velocity in the overflow region

Vun = Velocity in the underflow region

At the inlet section, the inflow and the introduced suspension are homogeneously spread over the horizontal cross section, and the suspension is diluted by convection as well as other transport processes. The flow is divided into a downward flow towards the underflow exit at the bottom, and an upward flow towards the effluent exit at the top. Both liquid and suspended matters enter the cylinder through the inlet cross section and are withdrawn at the bottom and at the top (Ekama et al., 1997).

According to Ekama et al. (1997), designing secondary settling tanks using the 1D flux theory is done in two stages. Firstly, zone settling and thickening considerations are applied, which lead to the determination of the surface area and depth; secondly, internal features are included in the tank, which should optimize the clarification efficiency. This second stage is usually done following some semi-empirical rules (e.g., twenty minutes retention time in the flocculator center-well) and strongly relies on the engineer's experience.

The 1D models have proved adequate for coupling with the activated sludge models because they give a reasonable approximation of the sludge balance and of the sludge shift from aeration tank to the secondary clarifier where it is partly stored during wet weather loading. Moreover, the application of these models does not require too much computer capacity.

Application of the idealized steady state 1D flux theory (1DFT) to full scale circular and rectangular secondary settling tanks (SSTs) indicate that the design procedures based on this theory over-predict the permissible solids loading rate by about 25 percent (Ekama et al., 1997). However, there was no convincing evidence that an 80% reduction in the predicted SLR needed to be applied for all SST (Ekama and Marais, 2002). Definitely, different tank geometries and configurations might give different correction factors.

Based on the solid flux concept, Takacs et al., 1991 presented a multi-layer model of clarification/thickening process that is designed to predict the solids profile and underflow suspended solids. This model provides a unified framework for simulation of clarification and thickening processes under both steady state and dynamic conditions. Later, Grijspeerdt et al., 1995 studied Takscs model and concluded that it is the most reliable to fit the data for steady state and dynamic conditions. However, the main disadvantage of this model is the relatively long calculation time required for convergence.

Another settling tank model is presented by Dupont et al., (1995). A dynamic one-dimensional flux model for the secondary settling tank shows that it has been possible to set up the model which at the same time can predict the suspended sludge concentration profile near the effluent weirs and the return sludge concentration of a secondary settling tank, when density current and short-circuiting are included as it is proposed in his model. These models were just developed considering the settling tank and anyone has coupled in their models the settling tank and the aeration tank.

The internal geometry, which may control the clarification efficiency of the clarifier, can not be evaluated using 1D models. For that reason, two and three dimensional models account for hydrodynamic and internal geometry configurations.

2.3. Flocculation in Biological Wastewater Treatment

The activated sludge process for wastewater treatment is based on the growth of microbial population in flocculated form. In order for the activated sludge to be operate successfully, it is required to develop a flocculent biomass that settles rapidly and compacts correctly in the clarifier (Grady, 1999).

Mecalf and Eddy (2003) defined flocculation as a "transport step that brings about the collisions between the destabilized particles needed to form larger particles that can be removed readily by settling or filtration". The flocculation produced by air agitation or mechanical agitation increase the removal of suspended solids and also improves de performance of secondary clarifiers in the activated sludge process.

An ideal activated sludge floc is very strong and compact so that it settles rapidly, producing a dense sludge for recycle to the bioreactor and a clear, high quality supernatant for discharge as treated effluent. According to Clauss et al. (1998), several parameters such as floc size and density provide an indication as to how to achieve good activated biomass/treated water separation. Furthermore, the floc structure is important since it determines floc size and density and it will influence the solids removal efficiency during sedimentation.

2.3.1. Bioflocculation

Biological flocculation takes place due to action of bacterial exocellular polymers on colloids and other finely divided particles. The most common mechanism of particle flocculation is chemical bridging. This process occurs when a coagulant substance (exocellular polymers) forms threads or fibers, which attach to several colloids, capturing and binding them together.

Pavoni et al. (1972) define the process of bioflocculation as the interaction of high-molecular- weight extracellular polymers, which have accumulated sufficiently at the microbial surface during endogenous growth. The floc is the result of physico-chemical interactions between microorganisms, inorganic particles, extracellular polymers and multivalent cations (Urbain et al., 1993).

Several phases of floc growth occur during flocculation. Initially, particle growth is dominant, particles combine by coagulation and their size increases rapidly. As flocculation continues, the flocs form large, porous and open structures that are more susceptible to fragmentation by fluid shear (Spicer et al., 1996).

Based on the particle sizes, flocculation is classified in two types: (1) microflocculation (perikinetic flocculation) and (2) macroflocculation (orthokineric flocculation). Spicer and Pratsinis (1996) determined that the floc size increases quickly until eventually a steady-state size is reached. Also, the size of the flocs ranges from 20 to 200 μm according to Muller et al. (1967) but Parker et al. (1971) showed a bimodal size distribution from 0.5 to 5 μm and 25 to 3000 μm.

According to Thomas et al. (1999), the mathematical representation of flocculation has conventionally been based on considering the process as two discrete steps: transport and attachment. The transport step, leading to the collision of two particles, is achieved by virtue of local variations in fluid-particle velocities arising through (1) the random thermal “Brownian” motion of particles (microflocculation), the imposed velocity gradients from mixing (macroflocculation) and differences in the settling velocity of individual particles.

2.3.2. Kinetic of Bioflocculation

In several studies of the physical nature of flocculation processes, the mechanism of aggregation through interparticle collision generally has been emphasized more than the role of floc breakup. Nevertheless, the balance of the opposing processes of aggregation and breakup that determines flocculation performance (Parker et al., 1972). The flocculation performance is materially reduced by floc breakup from the levels that might be obtained by aggregation processes alone.

According to Parker et al. (1970, 1971, and 1972), the net rate of change in the number of primary particles with respect to time in a batch reactor is given by the following equation:

$$\frac{dn}{dt} = k_B.X.G^m - k_A.X.n.G \quad \text{.....(2.1)}$$

Where:

n = primary particle number concentration (mg/l)

t = time (s)

X = mixed liquor suspended solids (MLSS) concentration (mg/l)

G = root-mean-square velocity or shear gradient (s^{-1})

K_A = floc aggregation rate coefficient (l/mg)

K_B = floc-breakup rate coefficient $((s)^{m-1}/(mg))$

m = floc breakup rate exponent (dimensionless).

Parker (1970, 1971, and 1972) applied equation 2.1 to the case of a continuously stirred tank reactor (CFSTR) operating under steady state conditions, and obtained equation 2.2.

$$\frac{n_0}{n_t} = \frac{(1 + k_A.X.G.T)}{(1 + K_B.D_c.G^m.T)} \text{.....} \quad (2.2)$$

Where:

n_0 = the ratio of the number of particles entering the flocculation unit

n_t = the particles leaving the unit

T = mean hydraulic residence time (HRT) in the flocculator

D_c = dispersion coefficient given by the quotient X/n_0

Integration of the equation 2.1 gives the number of particles remaining in the bath reactor after time *t*:

$$n_t = \alpha + \beta.e^{-\lambda.t} \text{.....} \quad (2.3)$$

Where:

$$\alpha = \frac{k_B.B}{K_A} \text{.....} \quad (2.4)$$

$$\beta = n_0 - \frac{k_B.G}{k_A} \text{.....} \quad (2.5)$$

$$\lambda = k_A.G.X \text{ .....} \quad (2.6)$$

α is defined as the minimum amount of suspended solids that could be obtained by flocculation. β is the integration constant defining the range of values between the initial conditions of the sludge and the lowest concentration of solids defined by α. Finally, the rate of flocculation is directly proportional to λ.

Equation 2.3 can be rewritten using more conventional terms according to Jimenez (2000) as presented in equation 2.7.

$$n = a + (n_0 - a).e^{-k.\bar{t}} \quad \text{.....(2.7)}$$

Where:

n = number of particles leaving the flocculator expressed as concentration of suspended solids. It can be estimated by the concentration of suspended solids in the supernatant of a mixed liquor sample after 30 min. of settling in mg/l.

a = asymptote of the curve in a batch reactor. It is the minimum number of particles that can be achieved by flocculation in the reactor as approaches infinite, mg/l.

n_0 = number of primary particles entering the flocculator expressed as concentration of the influent. It can be estimated by measuring the suspended solids concentration in the supernatant of an influent sample after 30 min. settling, mg/l.

k = fist-order rat constant, min^{-1}

$\bar{t}$ = reaction time, min

Using batch experiments Manrique (1999) demonstrated the validity of equation 2.3. Based on the equation 2.3 and taking the first derivative of it, Manrique (1999) found the rate of flocculation for batch reactors that can be expressed as follows:

$$r_{flocc} = \frac{dn}{dt} = -\lambda \cdot (n - \alpha) \quad \text{.....(2.8)}$$

A mass balance on particles in a continuous flow flocculation under steady state conditions yields:

$$0 = Q \cdot n_o - Q \cdot n + r_{flocc} \cdot V \quad \text{.....(2.9)}$$

Where:

Q = flow rate fed to the unit

n_0 = number of primary particles entering the flocculator, expressed as concentration. It can be estimated by measuring the suspended solids concentration in the supernatant of an influent sample after 30 min. settling.

n = number of primary particles leaving the flocculator expressed as concentration. n can be estimated by the concentration of suspended solids in the supernatant of a mixed liquor sample after 30 min. of settling.

V = volume of the reactor

r_{flocc} = rate of flocculation

Manrique (1999) substituted equation 2.8 in equation 2.9 and solved for *n*:

$$n = \frac{t \cdot \lambda \cdot \alpha}{1 + t \cdot \lambda} + \left(\frac{1}{1 + t \cdot \lambda} \right) \cdot n_0 \quad \text{.....(2.10)}$$

Jimenez (2000) rewrote equation 2.8 as follows:

$$r = -k.(n - a) \quad \text{.....(2.11)}$$

Where:

r = rate of flocculation, (mg/L min)

k = first order rate constant, min^{-1}

n = number of particles remaining unflocculated after t, expressed as concentration of SS

a = asymptote

Using equation 2.11 in a mass balance on particles in a continuous flow mixed reactor, such as a typical solids contact chamber, the concentration of particles in the effluent is given by the equation 2.12 (La Motta et al., 2004).

$$n = \frac{n_0 + k.\bar{t}.a}{1 + k.\bar{t}} \quad \text{.....(2.12)}$$

Where:

n_0 = number of unflocculated particles initially present in the reactor influent

$\bar{t}$ = average holding time in the solids contact chamber (min).

2.3.3. Biological Flocculation Process as an Organic Matter Removal mechanism

To study the removal mechanisms of organic matter in the activated sludge process, the total COD concentration can defined as the sum of particulate and dissolved COD in the liquid stream.

$$TCOD = PCOD + DCOD \quad \text{(2.13)}$$

Where:

$TCOD$ is the total COD concentration, mg/L

$DCOD$ is the COD due to dissolved or soluble organic material, mg/L

At the same time, particulate COD is defined as:

$$PCOD = SSCOD + CCOD \quad \text{(2.14)}$$

Where:

$SSCOD$ is the COD due to large organic suspended solids retained in the 0.45-μm filter, mg/L

$CCOD$ is the COD concentration due to organic colloids that passed the 0.45-μm filter, mg/L

As mentioned in section 2.2, the complex IWA/IAWQ Activated Sludge Models do not consider the biological flocculation process as an organic matter removal mechanism. There are few researchers that have considered the flocculation as an important process for COD removal (Parker et al., 1970; Wahlberg, 1992; and La Motta et al., 2004).

Parker et al. (1970) and Wahlberg (1992) focused their interest in flocculation studying the "tail end" of the activated sludge process, i.e., flocculation of primary particles eroded from floc in turbulent environments. La Motta et at. (2004), on the other hand, focused their research interest on studying the role of bioflocculation on chemical oxygen demand removal in the solids contact chamber of the combined trickling filter/solids contact process.

According to La Motta et al., (2004), in a TF/SC the trickling filter removes a major part of the soluble BOD contained in the primary effluent fed to the filter. Nevertheless, the final effluent from the trickling filter may be of unsatisfactory quality. Therefore, the solid contact chamber plays an important role in the process because it provides contact between the unflocculated particles in the influent and the recycled sludge, reverses the anaerobic condition of the sloughed off biofilm particles, and stimulates the production of extracellular polymeric substances (EPS) that promote particle flocculation.

La Motta et al., (2004) performed a series of batch reactor experiments to study and test the hypothesis that the biological flocculation is the mechanism responsible for most of the particulate COD removal. The experiment was developed in batch reactors, and considering of adding 2500 g of cation exchange resin (CER) as deflocculating agent to one of the two 17-L (4.5-gal) reactors containing samples of mixed liquor. By removing the divalent cations from solution by ion exchange, the chemical bridges used by the EPS were destroyed, and the original suspended solids and colloidal particles became free to remain in suspension. After that, suspended solids, total COD, colloidal COD and dissolved COD were measured in the supernatant at different time intervals. The results, presented in figure 2.2 and 2.3 demonstrated that flocculation has a huge impact on the quality of the supernatant with regard to supernatant suspended solids and total COD.

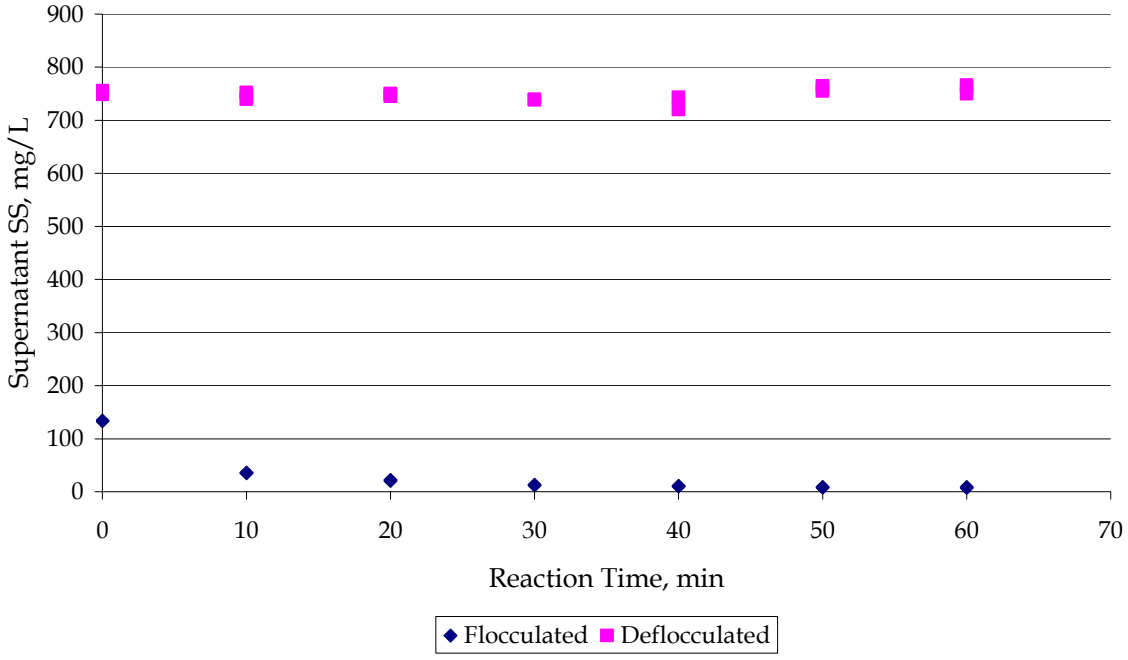

Figure 2.2 Effect of Bioflocculation on Supernatant SS Removal. (La Motta et al., 2004)

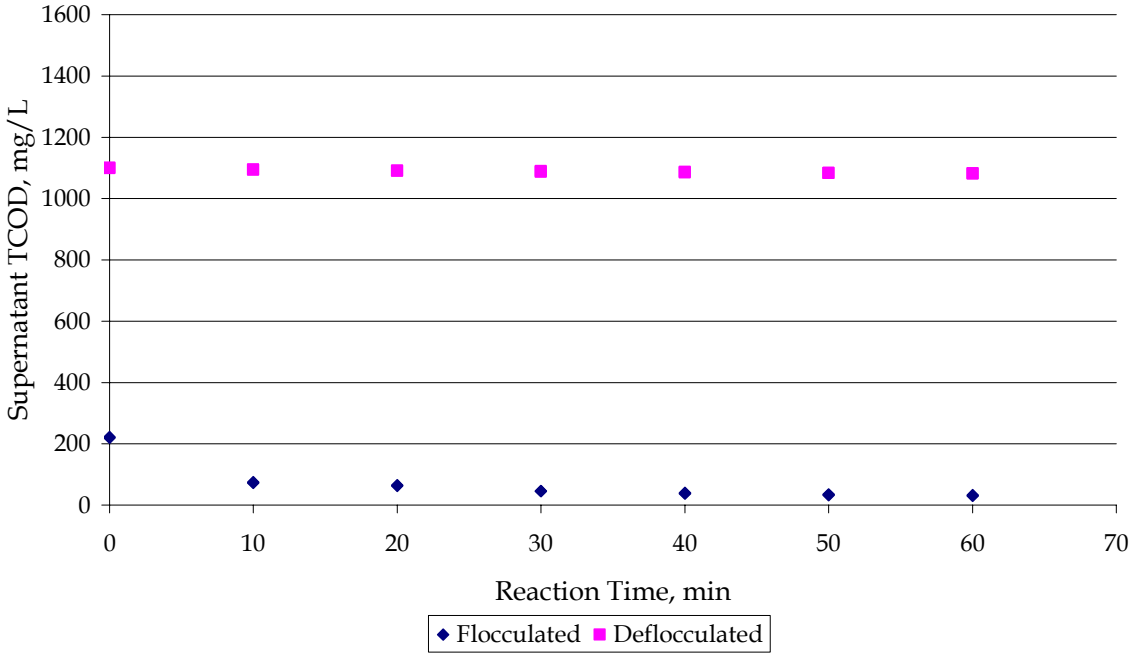

Figure 2.3 Effect of Bioflocculation on TCOD Removal. (La Motta et al., 2004)

According to Bioflocculation models proposed by Wahlberg et al., (1994) for batch and continuous flow mixed reactors, the flocculation efficiency is directly related to the holding time in the SCC. The validity of the flocculation kinetic model that is presented in equation 2.12 can be demonstrated in figure 2.4 (La Motta, et al., 2004), where the concentration of suspended solids in the supernatant after 30 min of settling of mixed liquor samples collected at various hydraulic detention times are plotted as a function of HRT.

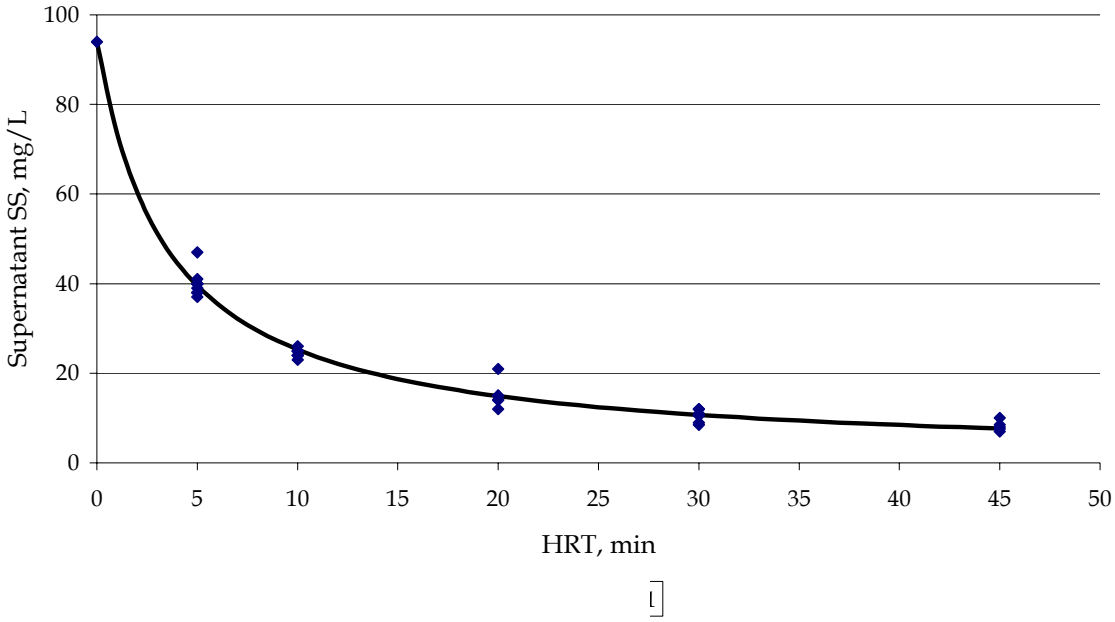

Figure 2.4 Effect of HRT on the Supernatant SS Removal. (La Motta et al., 2004)

The information presented above demonstrates that biological flocculation plays an important role in obtaining a good effluent quality. Considering that, most of the total COD in municipal wastewater is in particulate form; biological flocculation is a major mechanism for its removal in the aeration contact tank and allows its separation by gravity in the final

sedimentation tank. Therefore, flocculation optimization should be taken in consideration when designing the aeration tank.

Although the aeration tank plays an important role in the flocculation process, the secondary clarifier also promotes the formation of floc during the separation process. Camp (1945) recognized that flocculation in settling tanks is due to two causes:

1. Differences in the settling velocities of the particles whereby faster settling particles overtake those which settle more slowly and coalesce with them; and
2. Velocity gradients in the liquid, which cause particles in a region of higher velocity to overtake those in adjacent stream paths moving at slower velocity.

Parker et al. (1970, 1971, 1972) demonstrated the convenience of a flocculation zone prior to the final settling stage. They showed that often, the highly turbulent condition in the aeration chamber is so intense that it favors floc breakup over aggregation, resulting in a high level of dispersed solids. They recommended additional flow conditioning, through the incorporation of a mildly stirred flocculation step between the aeration basin and the clarifier to promote the incorporation of dispersed particles into the floc. This practice became popular to improve the final effluent of attached growth systems. However, La Motta et al., (2002a) demonstrated that if enough aeration and flocculation time and adequate environmental conditions are provided in the SCC, there will be no need for additional units to flocculate the outgoing mixed liquor.

The inclusion of flocculation zones inside the SSTs is a relatively new practice. Knop (1966) reported, on the basis of pilot plant studies, that the placement of a flocculator in the inlet zone of a rectangular SST improved effluent transparency. Lately, a full-scale plant was constructed including two sets of paddle flocculators. The induced flocculation in the clarifier improved the effluent SS by about 5 mg/L.

Parker (1983), Parker and Stenquist (1986), and Parker et al. (1996) presented full-scale research in circular clarifiers that showed that clarifiers equipped with flocculator centerwell (FCW) can yield good effluent suspended solid concentrations with high overflow rates. Parker and Stenquist concluded that "deep flocculator-clarifiers can achieve low suspended solids at overflow rates high enough to cause conventional shallow clarifiers to deteriorate." Even though they are comparing deep with shallow clarifiers, and it has been recognized that the distance of the sludge blanket from the effluent weir has a direct relation to effluent quality (Parker, 1983), the beneficial effects of the FCW are well defined in the aforementioned researches.

2.4. Sludge Settling Properties

Biological sludges usually show a strong flocculating tendency even at low concentrations (approximately 1,000 mg TSS/L). This gives rise to a zone settling behavior if a batch of mixed liquor is allowed to settle under quiescent conditions. (Ekama et al., 1997). During the process of a batch settling test, whether gently stirred or not, four distinct regimes of settling can be detected. This regimes or phases are namely lag regime or stage, the zone settling stage, transition, and compression. Figure 2.5 shows graphically the solid/liquid interface depth against the time.

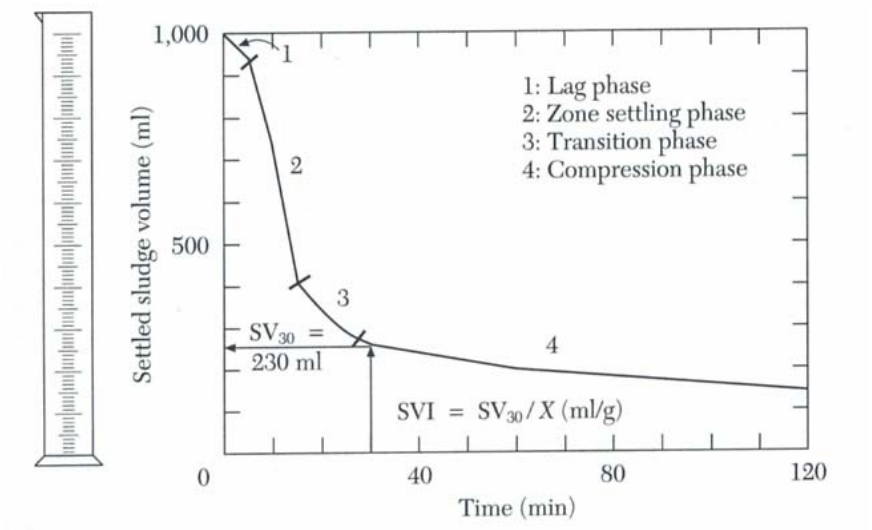

Figure 2.5 Plot of solid/liquid interface depth against the time in an unstirred measuring cylinder showing the appearance of the different stages of settlement at the solid/liquid interface (Ekama et al., 1997).

Two commonly used measures developed to quantify the settling characteristics of activated sludge are the sludge volume index (SVI) and the zone settling rate (WEF, 1998). The SVI is the volume occupied by 1 g of sludge after 30 min of settling. The SVI is determined by placing a mixed-liquor sample in a 1 to 2 L cylinder and measuring the settled volume after 30 min and corresponding sample MLSS concentration. The numerical value is computed using the following expression:

$$SVI = \frac{(settled.volume.of.sludge, ml/L)(10^3 mg/g)}{(suspended.solids, mg/L)} \quad \text{.....(2.14)}$$

According to Ekama et al., 1997, when the settling stage is reached, the gravitational forces causing the particles to settle and the hydraulic friction forces resisting the motion get in equilibrium, causing the particles to settle at a uniform (zone) terminal velocity. If the column is

stirred, the subsidence velocity of the interface is called the stirred zone settling velocity (SZSV) or V_{zs} (m/h) at the concentration with which the column was filled (X). The V_{zs} of the sludge is obtained from a solid/liquid interface depth-time plot (as it is shown in figure 2.6) and is given by the slope of the straight line part of the interface height versus time curve.

Considerable research has been undertaken to establish the form of the relationship between V_{zs} and X, and a number of mathematical expressions have been proposed, such as hyperbolic, logarithmic, power and exponential. Of the most popular form is the following (Vesilind, 1968):

$$V_{zs} = V_0 . e^{(-n.X)} \quad \text{.....(2.15)}$$

The constants V_0 and n (which have units of m/h and m^3/kg MLSS or L/g respectively) reflect the settling characteristics of the sludge. Ekama et al., 1997 established that well-settling sludges have V_0 values around 13 m/h and low n values around 0.25 m^3/kg, whereas poorly settling sludges have low V_0 values around 5 m/h and high n values around 0.5 m^3/kg.

Numerous models and methods have been presented in the literature for describing mixed-liquor settling characteristics. Dick and Ewing (1967) and Vesilind (1968) investigated various tests and procedures for developing reliable clarifier predictive models in the late 1960s. Dick and Ewing (1967) first developed the flux approach used in modern clarifier design and advocated the abandonment of old rules of thumb used for designing processes. According to Randall et al. (1992), these procedures for secondary clarifiers were originally defined by Coe and Clevenger (1916), Kynch (1952), and Yoshioka et al., (1957), then defined by Dick (1967,1976), Dick and Young (1972), and Keinath et al., (1977).

Vesilind (1968) promoted the settling column analysis procedure for evaluating the settling characteristics of biosolids. He recommended that the cylinder diameter should be as large as possible, about 20 cm (8 in.) and the initial height should be the same as the prototype thickener depth. When this is not practical, a 0.9 m (3 ft) depth was suggested and the sample should be stirred very slowly at approximately 0.5 revolutions per minute (rpm) throughout the test. One of the difficulties of this procedure is the time consuming. Every time the settling characteristics of the mixed-liquor change, a new test must be run.

2.5. Relationship between Activated Sludge Operating Parameters and the Sludge Settling Characteristics.

The primary function of a wastewater treatment plant operator is to properly monitor and control his treatment process so that he can consistently produce a high quality effluent that complies with the permit standards discharges. However, normally the operator defines the best operating conditions by trial and error without considering some important factors such as the settling characteristics. La Motta, (2004b) presents a set of equations linking the sludge settling properties, the settling tank and the aeration basin. With all those equation, he found a direct relationship between the operating parameters and the sludge settling characteristics. Using those equations, an approximation of the operation and design of an activated sludge system can be developed.

A summary of the equations developed by La Motta (2004) is presented below.

When mass balances on suspended solids in the aerator and in the settling tank (figure 2.6) are performed, and both equations are divided by the influent flow rate (Q), equation 2.16 and 2.17 respectively are obtained.

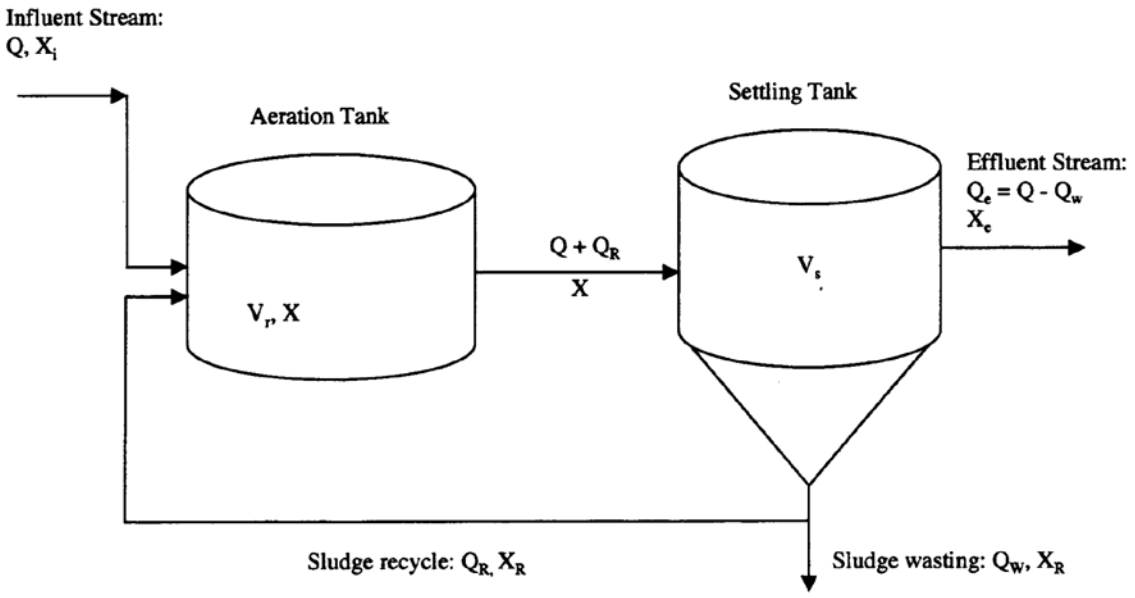

Figure 2.6 Complete-Mix Aerator Tank with Sludge Recycle. (La Motta, 2004)

$$X.\bar{t}.r_g + X_i = (1+\alpha).X - \alpha.X_R \quad \text{.....(2.16)}$$

$$(1+\alpha).X = (1-w).X_e + (\alpha + w).X_R \quad \text{.....(2.17)}$$

Where:

Q = influent flow rate, m^3/d

Q_R = recycle flow rate, m^3/d

Q_W = waste sludge flow rate, m^3/d

r_g = rate of growth of suspended solids, kg SS/kg MLSS.d

V_r = reactor (aerator) volume, m^3

V_s = settling tank volume, m^3

X = MLSS concentration in the influent to the aerator, kg/ m^3

X_e = SS concentration in the final effluent, kg/ m^3

X_R = SS concentration in the recycle line, kg/ m^3

X_w = SS concentration in the waste stream, kg/ m^3

α = recycle ratio, $\frac{Q_R}{Q}$

w = waste ratio, $\frac{Q_w}{Q}$

La Motta, (2004) combined equations 2.16 and 2.17 which gives the equilibrium of solids in the system with no accumulation of solids. Equation 2.18 shows the combination of both equations.

$$X.\bar{t}.r_g + X_i - w.X_R - (1-w)X_e = 0 \quad \text{.....(2.18)}$$

Defining the solids retention time:

$$\bar{t}_c = \frac{Mass\ .of\ .solids\ .in.the.system\ .(aerator\ + settling\ .\tan\ k), kgSS}{Rate\ .of\ .solids\ .withdrawal\ .from\ .the.system\ , kgSS\ /\ d}$$

Assuming that under equilibrium conditions, the average concentration of solids in the settling tank is the same as the concentration in the aerator equation 2.19 is obtained.

$$\bar{t}_c = \frac{X.\bar{t}.(1+\frac{V_s}{V_r})}{w.X_R + (1-w).X_e} \quad \text{.....(2.19)}$$

La Motta also considered that the growth of MLSS in the aerator takes place by microbial growth (net growth) and by biological flocculation of inorganic and organic particles contained in the wastewater. Equations 2.20 and 2.21 define the rate of growth in the reactor and the rate of flocculation respectively.

$$r_g = r_{ng} + r_f \quad \text{.....(2.20)}$$

$$r_f = k_x.(X_e - a_x) \quad \text{.....(2.21)}$$

Where:

r_{ng} = net rate of growth of microorganisms, kg SS/kg MLSS.d

r_f = rate of flocculation of particles, kg SS/Kg MLSS.d

a_x = kinetic parameter of TSS flocculation, kg/m^3

k_x = first order constant of TSS flocculation, m^3/d.kg

According to Metcalf and Eddy (2003), the rate of disappearance of dissolved COD is given by equation 2.22.

$$-U = \frac{1}{Y}.(r_{ng} + k_d) \quad \text{.....(2.22)}$$

Where:

k_d = endogenous respiration coefficient, d^{-1}

U = rate of uptake of dissolved COD, kg COD/kg MLSS.d

Y = true yield coefficient, kg biomass/kg DCOD consumed

Doing a mass balance on dissolved COD around the aerator and writing it in terms of total COD, La Motta presented equation 2.23.

$$-U = \frac{(1-f_p).(S_{T_i} - S_T)}{X.\bar{t}} \quad \text{.....(2.23)}$$

Where:

F_p = PCOD/TCOD

S_T = total COD concentration in the aerator, kg/m^3

S_{Ti} = total COD concentration in the influent stream, kg/m^3

Next, a combination of equations 2.20, 2.21, 2.22, and 2.23 results in equation 2.24:

$$r_g = \frac{Y}{X.\bar{t}}.(1-f_p).(S_{T_i} - S_T) + k_x.(X_e - a_x) - k_d \quad \text{.....(2.24)}$$

Finally, a combination of equations 2.18, 2.19, and 2.24 yields equation 2.25:

$$\bar{t}_c = \frac{\bar{t}.X.\left(1+\frac{V_s}{V_r}\right)}{X_i + Y.(1-f_p).(S_{T_i} - S_T) + \bar{t}.X.\left[k_x.\left(X_e - a_x\right) - k_d\right]} \quad \text{.....(2.25)}$$

Additionally, a relationship between the aerator and the settling tank was studied. The limiting flux theory was used for the settling tank; to be able to handle the sludge with given settling characteristics; equation 2.26 must be hold:

$$\frac{F_L.A}{Q} = (\alpha + w).X_R \quad \text{.....(2.26)}$$

The sludge settling characteristics are commonly described by the equation 2.27.

$$F_B = X.v_0.e^{n.X} \quad \text{.....(2.27)}$$

Where:

F_B = batch sludge flux, kg/d.m^2

V_0 = settling velocity parameter, m/d

n = empirical parameter, m^3/kg (n<0)

The liming flux for settling tank design, usually found by drawing a tangent through the batch flux plot, starting on the horizontal axis at an underflow concentration X_R. Figure 2.7 represent graphically the limiting flux for settling tank design.

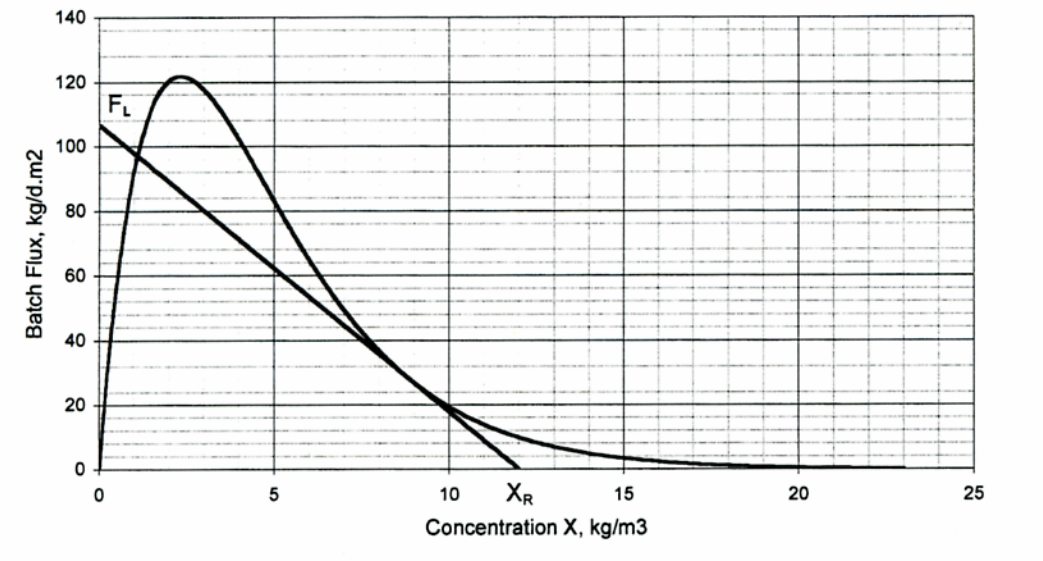

Figure 2.7 Graphical representation of the Limiting flux for settling tank design.
(La Motta, 2004)

The limiting flux can be expressed as a function of the sludge concentration in the recycle line and the sludge settling characteristics, as presented in equation 2.28.

$$F_L = \frac{v_0 . e^{\left[\frac{n}{2}.\left(X_R+\sqrt{X_R^2+\frac{4.X_R}{n}}\right)\right]}}{\frac{1}{\frac{1}{2}\left(X_R+\sqrt{X_R^2+\frac{4.X_R}{n}}\right)}-\frac{1}{X_R}} \quad \text{.....(2.28)}$$

A mass balance on non-settleable solids was performed making an assumption that the secondary clarifier is 100% efficient. There are two factors that affect the concentration of supernatant suspended solids. These two factors are the rate of growth of colloidal particles and the rate of flocculation. La Motta (2004) took into account those factors in the mass balance on unflocculated particles. Equation 2.29 displays the mass balance on unflocculated particles solving for the effluent concentration.

$$X_e = \frac{X_i + \alpha.(SSS)_R + \bar{t}.k_x.a_x.X}{1 + \alpha + \bar{t}.(k_x - k_g).X} \quad \text{.....(2.29)}$$

Considering $\bar{t}$ equal to zero, the following plotting point can be obtained:

$$X_0 = \frac{X_i + \alpha.(SSS)_R}{1 + \alpha} \quad \text{.....(2.30)}$$

Finally, combining equation 2.29 and 2.30, yields:

$$X_e = \frac{(1 + \alpha).X_0 + \bar{t}.X.k_x.a_x}{1 + \alpha + \bar{t}.X.(k_x - k_g)} \quad \text{.....(2.31)}$$

Performing a mass balance on particulate COD and solving for the particulate COD concentration in the aerator, La Motta obtained equation 2.32.

$$S_p = \frac{S_{p_i} + \alpha.S_{P_R} + k_P.a_P.\bar{t}.X}{1+\alpha+\left(k_g - k_{gp}\right).\bar{t}.X} \quad \text{……(2.32)}$$

Where:

S_{Pi} = particulate COD concentration in the influent of the aerator, kg/m^3

S_{PR} = particulate COD concentration in the recycle line, kg/m^3

k_P = first order constant of PCOD flocculation, m^3/d.kg

a_P = kinetic parameter of PCOD flocculation, kg/m^3

k_{gp} = first order constant of PCOD grwth, m^3/d.kg

When $\bar{t}$ = 0, $S_P = S_{P0}$. Therefore:

$$S_{P0} = \frac{S_{P_i} + \alpha.S_{P_R}}{1+\alpha} \quad \text{……(2.33)}$$

Later, combining equations 2.32 and 2.33, and using the relationship $S_P = f_P.S_T$, yields equation 2.34.

$$S_T = \frac{S_{T0}.(1+\alpha) + \frac{k_p.a_P}{f_P}.\bar{t}.X}{1+\alpha+\bar{t}.X.\left(k_P - k_{gp}\right)} \quad \text{……..(2.34)}$$

Finally, based on the assumption that there is no accumulation of solids in the system leads to the following simplified equations are obtained:

$$w = 1 - \frac{\frac{F_L.A}{Q}.\left(1 - \frac{X}{X_R}\right)}{X - X_e} \quad \text{.....(2.35)}$$

$$\alpha = \frac{F_L.A}{Q.X} - 1 + \frac{X_e}{X}.\left(1 - \frac{X.\frac{\bar{t}}{\bar{t}_c}.\left(1 + \frac{V_s}{V_r}\right) - X_e}{X_R - X_e}\right) \quad \text{.....(2.36)}$$

$$X = X_e + \frac{F_L.A}{Q.(1+\alpha)}.\left\{1 - \frac{X_e}{\frac{X}{\alpha}.\left[1 + \alpha - \frac{\bar{t}}{\bar{t}_c}.\left(1 + \frac{V_s}{V_r}\right)\right]}\right\} \quad \text{.....(2.37)}$$

In order to find a solution for the design and operating of an activated sludge system, equations 2.25, 2.28, 2.31, 2.34, 2.35, 2.36 and 2.37 must be solved simultaneously. The software UNO 1-D Activated Sludge Model (La Motta and Homes, 2004) solves those equations giving results instantaneously.

CHAPTER III

3. EXPERIMENTAL PHASE

3.1 Introduction

This experimental program is primarily oriented to establishing a relationship between the sludge settling characteristics and the operating parameters of the activated sludge system. This study was developed and carried out utilizing a pilot-scale activated sludge system. The experimental pilot plant is located within the installations of the Marrero Wastewater Treatment Plant, 6250 Lapalco Boulevard, Marrero, Louisiana. This facility is a 34000 m^3/d tricking filter/solid contact process that treats domestic sewage with a design average flow rate of 24,226.64 m^3/d (6.4 MGD). Nevertheless, the actual average flow rate that is currently running into this plant is about 34068.71 m^3/d (9.0 MGD).

During this research the aeration basin was operated with two different hydraulic retention times and maintaining constant the other three parameters: recirculation ratio, sludge wasting ratio. Raw and treated wastewater samples were taken during the period March 2004 to October 2004 in order to generate data required for this project.

3.2. Pilot Plant Description

The pilot plant is an Aerobic Activated Sludge System. It is composed by the following parts: an inlet mechanism, a rotating screen, a reservoir tank, an aeration basin, a secondary clarifier and a final effluent collection tank. Figure 3.1 illustrates a diagram of the pilot plant.

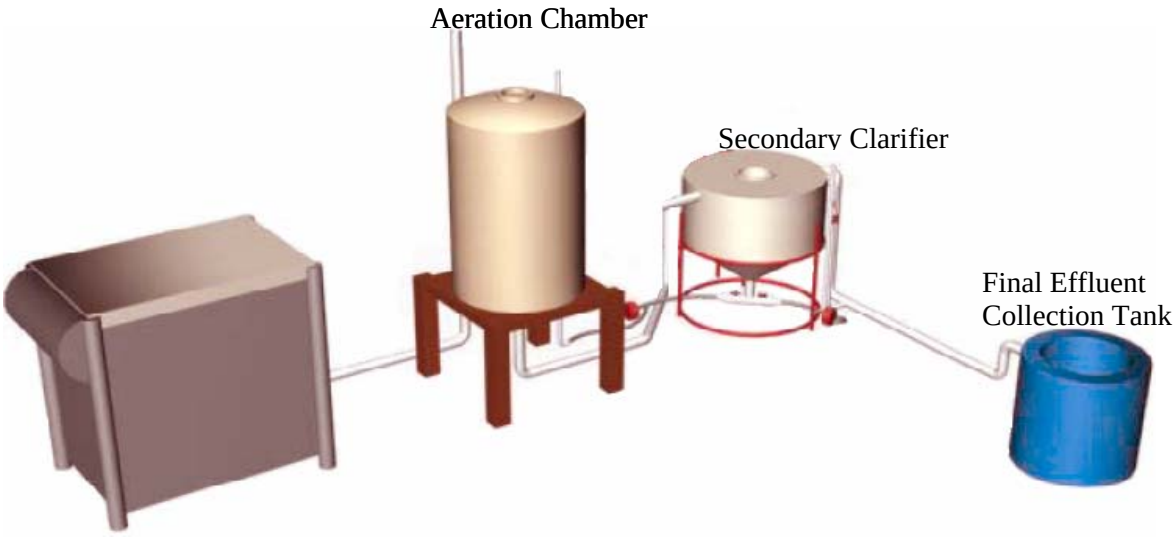

Figure 3.1 Pilot Plant Diagram (Jimenez 2002)

3.2.1. Inlet mechanism

The pilot plant is fed by pumping the wastewater form a grit chamber splitter tank in the full scale plant. This inlet mechanism consists of 372.5 W (1/2 hp) centrifugal pump, Model TEEL / 3P55, self-priming, ½ HP, 115/230 volts, 0.220 m^3/min (58 GPM) at 3.048 m (10 ft) of head. This pump has a straining device connected to the influent opening to prevent coarse solids from clogging the suction system. The straining apparatus consists of a perforated 0.91 m (3ft) section of 10.2 cm (4in) diameter PVC pipe covered with a metal screen of 9.5 mm (3/8 in) opening size. To protect the straining device, this apparatus is cased with a 20.32 cm (8 in) diameter and 3 m long PVC pipe.

The suction pump delivers the wastewater through 50 m (164 ft) of 2.54 cm (1 in) diameter PVC pipe to a rotational screen. The Figure X-2 shows the straining device.

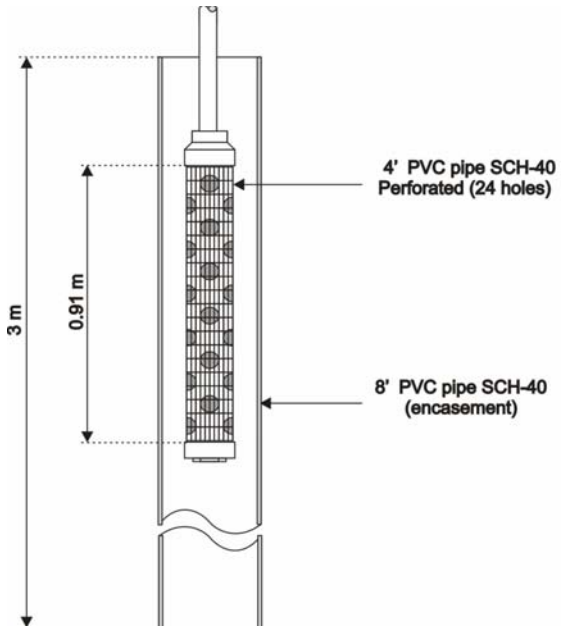

Figure 3.2 Straining Device (Padron 2004)

3.2.2. Rotating Screen and Reservoir Tank

Wastewater that comes from the splitter box flows to a rotating cylinder screen, model WaterLink Rotostrainer® / RSA2512UBCR, 1/3 HP, 120 volts with 0.5 mm (0.020 in) of clear spacing. This rotating cylindrical screen removes the solids larger than 0.5 mm; a blade attached to the front of the apparatus scrapes off the solids, which fall into an external collection basin. The screened wastewater is pumped out with a centrifugal pump, model TEEL / 1P809, submersible, 74.57 W (1/10 HP), 115 volts, 900 GPH at 0.3048 m (1 ft) of head to a 0.11 m^3 (30 gal) polyethylene tank. This is to obtain a well- mixed wastewater and to avoid the settling of particles and also serves as a holding container for the pilot plant. Finally, another centrifugal pump, model TEEL / 1P809, submersible, 74.57 W (1/10 HP), 115 volts, 3.41 m^3/h (900 GPH) at 0.3048 m (1 ft) of head pumps the wastewater to the aeration basin.

3.2.3. Aeration Basin

The aeration basin consists of a 152 L polyethylene tank, equipped with 8 heat-bonded silica fine-pore diffusers with a 15 cm length, 4 cm width, a maximum pore size of 80 microns and a bubble size between 0.5-2.0 mm. The diffusers are fed with air by an air compressor, model GAST / 4F742, 559.27 W (¾ HP), 115/230 volts, free air flow at 0.254 m (10 in.) and Vacuum 0.178 m^3/min (6.3 CFM), to maintain the required dissolved oxygen levels and to provide uniform mixing. The air is regulated by an air flow meter with valve, model FR4500A67BV with a maximum capacity of 0.15 m^3/min (5 ft^3/ min), and it is placed in the outlet of the air compressor to regulate the amount of air that is sent to the aeration basin.

Also, two inlet PVC pipes with a 0.0381 m (1 ½-in) in diameter are connected to the aeration tank. The first inlet comes from the reservoir tank to provide the wastewater to the system; the flow rate is regulated by a 0.0381 m (1 ½ in) PVC ball valve. The second inlet comes from the recirculation line to provide the sludge recycled from the bottom of the secondary clarifier. The inlets, the wastewater, and the recycled sludge are fed to the bottom of the aeration basin where they are mixed with the reactor contents. The mixed liquor leaves the aeration basin by gravity through a center well to the secondary clarifier. Figure 3.3 exemplifies the aeration basin.

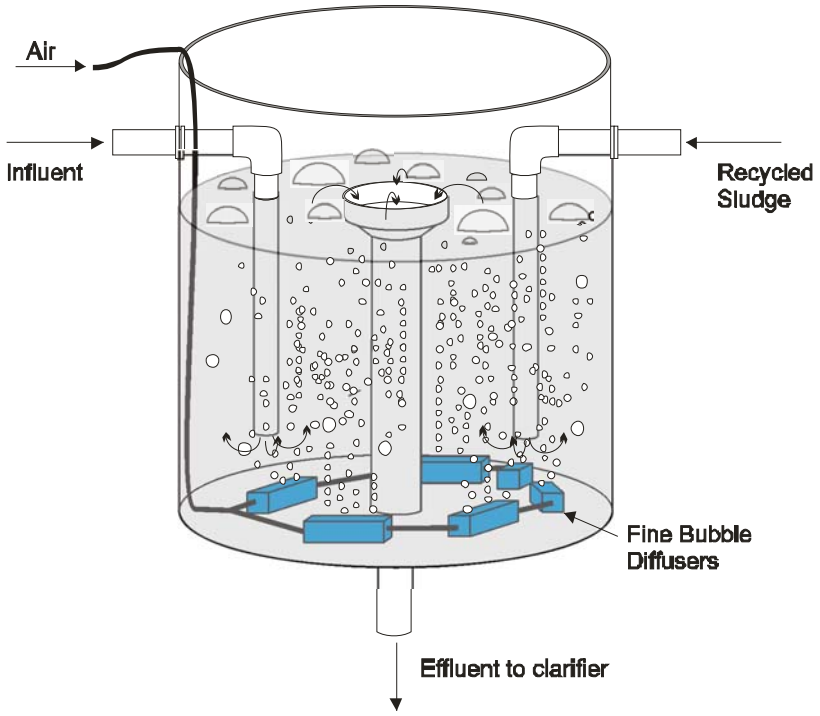

Figure 3.3 Aeration Basin
After Padron, (2004)

3.2.4. Secondary Clarifier

The secondary clarifier consists of a 280 L (70 gal) polyethylene conical tank which is loaded with the mixed liquor that leaves the aeration basin. The aeration basin effluent flows into the secondary clarifier through a 3.81 cm (1 ½ in) PVC pipe that delivers the mixed liquor tangentially into a 20.3 cm (8 in) diameter center well to minimize the inflow energy and to create a circular motion that provides additional flocculation to the mixed liquor solids. A rotary arm or scrapper is located at the bottom and it is moved by a parallel shaft synchronous AC gearmotor, model T2602-001, 1 rpm, with a full load torque 7.257 kg (16 lb), 60 Hz. The scrapper avoids the formation of solids clumps in the conical section of the secondary clarifier.

The clarifier has three effluent collectors, 3.81 cm (1 ½ in) in diameter. They are placed vertically on the surface of the water in opposite location of the tank.

Part of the sludge that settles in the bottom of the secondary clarifier is recycled to the aeration basin and another part is wasted and sent to a collection tank using two different lines. The two sludge systems are delivered by using two open air/submersible pumps, model TEEL / 1P808, 14.9 W (1/50 HP), 115 volts, 1.51 m^3/h (400 GPH) at 0.3048 m (1 ft) of head, and they are connected and separated with a PVC tee at the bottom of the clarifier. The flow rate from the recirculation line and the waste line are regulated by two repeated cycle timers, model H3CR-F8-300AC100-240, 100-240 VAC, 5 amps. Figure 3.4 shows graphically the secondary clarifier.

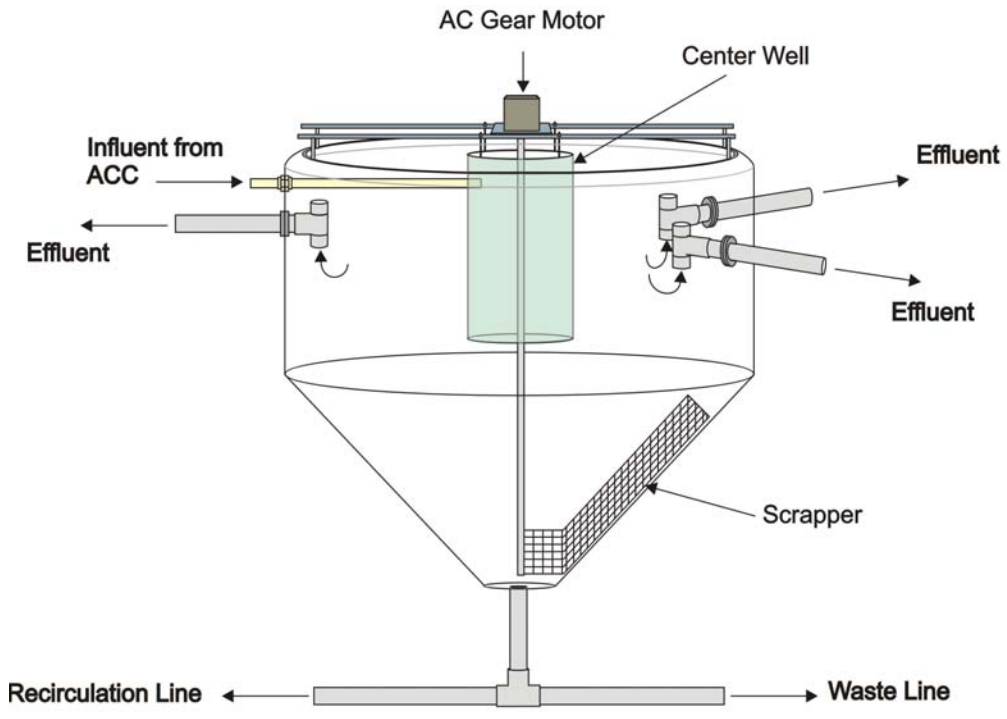

Figure 3.4 Secondary Clarifier
After Pardon (2004)

3.3. Sampling and Laboratory Analysis

3.3.1. Sampling

The sampling phase was initiated in February 2004 and finished in October 2004. Sample collection for each hydraulic retention time was made usually in the morning depending of the plant operation conditions. Samples were collected at different points during the experimental phase: influent from the aeration basin, the sludge from the recirculation line, the mixed liquor from the aeration basin and the effluent from the secondary clarifier. Also, samples were taken from the sludge dilutions made to develop the settling test in order to obtain the sludge settling parameter. Finally, supernatant samples were collected from the mixed liquor and from a mixture that consists of 1000 ml of the aeration basin influent and 1000 ml of the recycled sludge multiplied by the recirculation ratio. The mixture was made to simulate the values of the supernatant suspended solids in a system with hydraulic retention time equal to zero. Sampling and analysis were developed following the recommendations and procedures published in the Standard Method (AWWA, 1995).

3.3.2. Laboratory Analysis

Three parameters were measured at the Civil and Environmental Laboratory located at the Center for Energy Resources Management (CERM): total suspended solids (TSS), total chemical oxygen demand (TCOD), dissolved chemical oxygen demand (DCOD). The particulate chemical oxygen demand (PCOD) was calculated as the difference between the TCOD and DCOD. Table 3.1 shows the parameters performed for each sample collected.

Table 3.1 Water Quality Parameters at Each Sampling Point

Sampling Points	Parameters		
	TSS	TCOD	DCOD
Aeration Basin Influent	X	X	X
Aeration Basin Mixed Liquor	X		
Recycled Sludge	X		
Secondary Clarifier Effluent	X	X	X
Supernatants	X	X	X
Settling Test Dilutions	X		

3.3.2.1. ***Total Suspended Solids***

Total suspended solids (TSS) analysis is used to quantify the amount of organic and inorganic suspended matter present in the sample. This analysis was developed following method 2040D of the Standard Methods (APHA, 1999).

3.3.2.2. ***Total Chemical Oxygen Demand***

To perform and determine the chemical oxygen demand, method 5220D of the Standard Methods was followed. This analysis is used to quantify the amount of oxygen necessary to chemically oxidize the organic matter present in the sample. Samples were homogenized by mixing with magnetic stirrers.

3.3.2.3. ***Dissolved Chemical Oxygen Demand***

A physical-chemical method described by Mamais, et al., (1993) was implemented to quantify the dissolved organic matter in the samples analyzed. First, 100 mL of the sample was isolated in a 125 ml flask. Then, the pH of the sample was adjusted to approximately 10.5 with a 6 M sodium hydroxide solution. Next, the samples were flocculated by adding 1 mL of a 100 g/L zinc sulfate solution to the 100 mL of the sample. The sample was vigorously mixed with a magnetic stirrer for approximately 1 min, and was allowed to settle quiescently for approximately 3 min (Standard Methods, Section 417 B, 1998). The flocculated solids, 20-30 mL of the clear supernatant was carefully withdrawn with a pipette and passed through a Hach No. 30 qualitative filter paper, with a 0.47-µm pore size. Due to the existence of starch in the filters, it was important to wash the filters with deionized water prior to filtration. The COD of the supernatant filtrate was taken as the truly dissolved COD of the sample.

3.3.2.4. ***Particulate Chemical Oxygen Demand***

As it was mentioned before, particulate chemical oxygen demand was calculated with the difference of the total chemical oxygen demand and the dissolved chemical oxygen demand. Particulate COD is comprised of colloidal and suspended solids.

3.3.3. Field Measurements

3.3.3.1. *Pilot Plant Flow Rate*

The pilot plant flow rate was measured every day to ensure that the it was constant along the experimental phase. This flow rate was measured at the influent of the aeration basin filling a 1000 mL graduated cylinder during a known period of time.

3.3.3.2. ***Recirculation Flow Rate***

The flow rate from the recycled line was measured every two days using the aforementioned method. The flow rate of the sludge that is recycled form the settling tank to the aeration basin is controlled by a timer. During one of the cycles of the timer, The measurement of the flow rate was developed. The sludge collected was returned to the aeration basin to maintain the exact amount of recycled sludge at every cycle.

3.3.3.3. ***Supernatants***

Supernatant from the mixed liquor and from the mixture of 1000 mL of influent and 1000 mL multiplied by the value of the recirculation ratio were performed. To obtain the supernatants, a one liter mixed liquor sample from the aeration basin was collected using a graduated cylinder and allowed to settle for 30 minutes. Next, around 500 mL of supernatant were collected with a siphon to avoid getting solids from the sludge blanket.

3.3.3.4. ***Sedimentation Characteristics***

The settling parameters were obtained developing consecutive settling test according to Method 2710 E of the standard Methods (APHA, 1998). Five different concentrations were tested in order to obtain the setting parameters. A sample of each concentration was sent and analyzed in the laboratory to determine the exact amount of suspended solids concentration.

3.3.3.5. ***Dissolved Oxygen and Temperature***

The dissolved oxygen was measured every day with an YSI handheld dissolved oxygen and temperature meter, Model 55. The probe was introduced in the aeration basin until a stable reading was obtained. Temperature was measured at the same time using the same apparatus. It

is important to mention that the equipment was frequently calibrated according to the manufacturer's instructions.

3.3.3.6. ***pH***

A mixed liquor sample from the aeration basin was collected for a pH measurement. Samples were measured using a WTW pH meter, model 330. Previously, the apparatus was calibrated according to the manufacturer's instruction. Subsequently, an electrode was introduced into a flask filled with the sample. The lecture was taken when the screen indicated a stable value.

CHAPTER IV

4. ANALYSIS AND DISCUSSION OF THE RESULTS

For the development of this project, the activated sludge system was operated with two different hydraulic retention times, 45 min and 30 min. The hydraulic retention time was changed using two different flow rates as indicated in Table 4.1.

Table 4.1 Pilot Plant Flow Rate Used for the Procurement of the Different HRT

Aeration Tank, m^3 (gal)	Plant Flow Rate, m^3/d (gpd)	HRT, min
0.175 (46.23)	5.599 (1479.23)	45
0.175 (46.23)	8.388 (2215.97)	30

Two variables were maintained constants at each HRT the recirculation ratio (α) and the waste ratio (w). The following table illustrates the values of the recirculation ratio and the waste ratio at different HRT.

Table 4.2 Recirculation Ratio and Waste Ratio Used during the Experimental Phase

HRT (min)	α	w
45	0.701	0.019
30	0.712	0.035

The dissolved oxygen (DO) could be maintained constant just for the hydraulic retention time of 45 min. For the HRT of 30 min the DO vary between 0.9 and 2.69 mg//L due to some mechanical problems with the air compressor during the sampling. The unit performance was erratic in this case, but the results obtained in this case provided useful information for the development of this project. Appendix B shows in detail the operation of the aerobic activated sludge system pilot plant.

During the experiments with both hydraulic retention times (45 and 30 min), the system was operated with concentrations of mixed liquor suspended solids more than 1900 mg/L. Samples were taken daily during the sampling period after the system had reached stable operating conditions. Laboratory analysis of TSS in the mixed liquor (MLSS), recycled sludge (X_R), influent (X_i), the supernatant of the mixture of 1000 ml of the raw wastewater and 1000 ml of the recycled sludge multiplied by the recirculation ratio (X_0) and also the TCOD in the influent (S_{ti}) and in the supernatant of the mixture (S_0) were perform in order to run the 1D Activated sludge Model and find the predicted parameters. Table 4.3 shows a summary of the results of the aforementioned experiments. Detailed experimental data are presented in Appendix C.

Table 4.3 Parameters obtained in the Activated sludge system for the running of the 1D Activated Sludge Model during the HRT of 45 and 30 min

HRT	**TSS (kg/m^3)**					**TCOD (kg/m^3)**		***n***	***V_0***
(min)	DO	MLSS	X_R	X_i	X_0	S_{ti}	S_0	**(m^3/kg)**	**(m/d)**
45	0.0035	1.978	5.225	0.289	0.046	0.257	0.052	-0.9192	414.80
45	0.0046	2.376	6.720	0.293	0.032	0.301	0.115	-0.7907	400.30
45	0.0041	2.470	6.015	0.203	0.024	0.255	0.093	-0.8998	450.24
45	0.0035	2.836	6.320	0.115	0.036	0.115	0.078	-0.7684	348.48
45	0.0045	3.014	6.235	0.471	0.029	0.361	0.068	-0.6692	344.54
45	0.0034	3.242	7.790	0.520	0.040	0.356	0.115	-0.6442	224.15
45	0.0042	3.348	7.975	0.357	0.017	0.381	0.138	-0.5721	298.91
45	0.0036	4.645	9.145	0.438	0.041	0.438	0.118	-0.4080	214.85
30	0.0013	2.022	4.150	0.159	0.050	0.341	0.152	-0.7727	143.29
30	0.0009	2.600	5.915	0.201	0.094	0.403	0.223	-0.7328	272.43
30	0.0027	3.093	6.495	0.123	0.032	0.359	0.134	-0.6780	260.77
30	0.0013	3.154	6.390	0.187	0.073	0.421	0.208	-0.4838	199.57
30	0.0016	3.515	6.530	0.205	0.093	0.422	0.252	-0.4971	179.69

Each set of experimental results shown in Table 4.3 was introduced in the UNO 1D Activated sludge Model including the sludge settling characteristics and the recirculation ratio. The model output include predicted values of the mixed liquor concentration, the recycled sludge concentration, the waste ratio, the waste flow rate, the recirculation flow rate, the solid retention time, the effluent SS concentration, and the organic matter effluent concentration. The results generated by the model are presented in detail in Appendix D. Table 4.4 shows the values observed in the field, and a summary of the model predictions

Table 4.4 Parameters Observed in the field and Predicted by the 1D Activated Sludge Model

MLSS observed	MLSS predicted	X_R observed	X_R predicted	SRT observed	SRT predicted
1.978	3.987	5.225	6.985	0.93	0.40
2.376	3.579	6.720	8.089	0.96	0.49
2.470	3.179	6.015	7.290	1.15	0.61
2.836	3.500	6.320	8.147	1.18	0.82
3.014	4.156	6.235	9.249	1.19	0.41
3.242	3.987	7.790	8.723	1.06	0.34
3.348	4.615	7.975	10.569	1.16	0.60
4.645	6.037	9.145	13.770	1.34	0.56
2.022	2.591	4.150	5.771	0.71	0.30
2.600	3.314	5.915	7.295	0.59	0.26
3.093	3.425	6.495	7.884	0.80	0.52
3.154	4.510	6.390	10.267	0.45	0.42
3.515	4.304	6.530	9.656	0.76	0.33

Finally, temperature and pH were measured daily to have a record of them during the samples collection. These values are shown in Appendix B.

4.1. Relationship between the MLSS Concentration Observed in the Field and the MLSS Concentration Predicted by the UNO 1D Activated Sludge Model.

Figure 4.1 presents a relationship between the MLSS predicted by the model and the actual observed values.

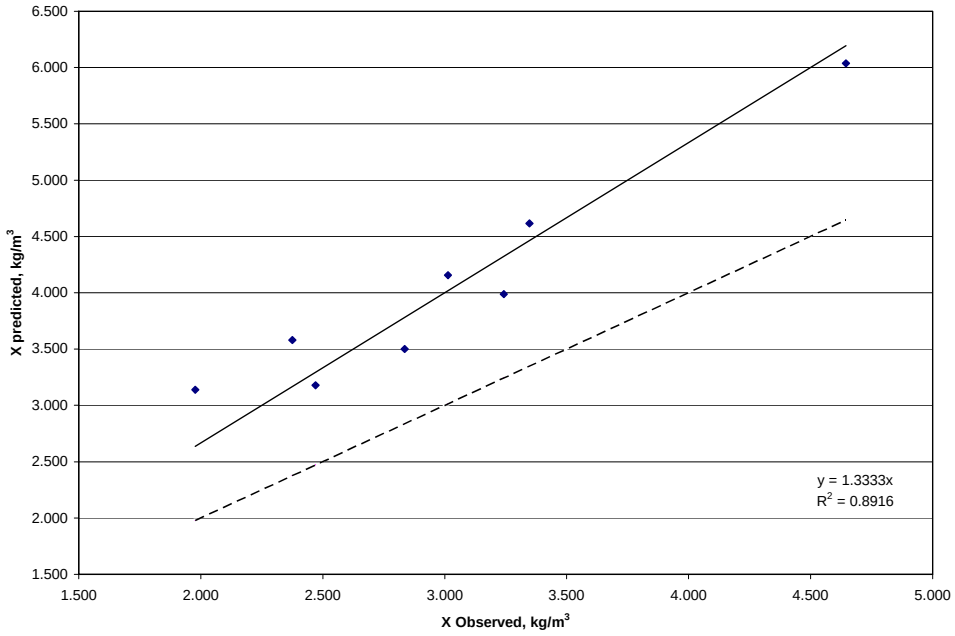

Figure 4.1 Relationship between the MLSS Concentration Observed in the Field and the MLSS Concentration Predicted during a HRT of 45 min

Figure 4.1 shows that there is a linear relationship between MLSS predicted and MLSS observed. A linear regression analysis generated the equation 4.1.

$$X_{predicted} = 1.333.X_{observed} \quad \text{.....(4.1)}$$

Where:

$X_{predicted}$ *and* $X_{observed}$ are in kg/m^3

The coefficient of determination (R^2) for the data is 0.89 which means that 89% of the variability of the data could be explained by equation 4.1. Although this is an excellent correlation, the model is overpredicting the actual value of MLSS by 33%, which is a substantial difference.

A close examination of equation 4.2 reveals the 1D model prediction of X relies heavily on the value of the limiting flux, F_L. Similarly, equation 4.3 shows that X_R is directly proportional to X. Therefore, any overprediction of the value of X will result in a similar overprediction of X_R. Equation 4.2 and 4.3 are shown below:

$$X = \frac{\frac{F_L.A}{Q} + (1-w).X_e}{1+\alpha} \quad \text{.....(4.2)}$$

$$X_R = \frac{(1+\alpha)X - (1-w)X_e}{(\alpha + w)} \quad \text{.....(4.3)}$$

Figure 4.2 illustrates the erratic unit performance observed at HRT = 30 min. A linear correlation analysis yielded a low coefficient of determination (R^2) of 0.36 and the following equation:

$$X_{predicted} = 1.3325.X_{observed} \quad \text{.....(4.4)}$$

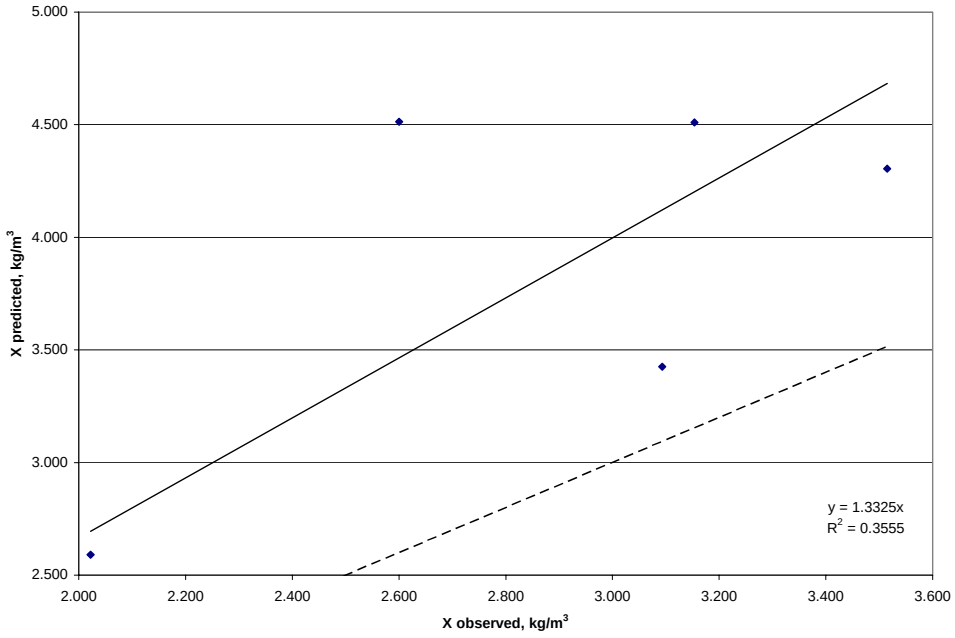

Figure 4.2 Relationship between the MLSS Concentration Observed in the Field and the MLSS Concentration Predicted during a HRT of 30 min

This low correlation can be attributed to the lack of dissolved oxygen in the system, which resulted in poor flocculation and unstable system behavior. The reliability of this set of data, therefore, is questionable.

4.2. Relationship Between the Recycled Sludge Concentration Observed in the Field and the Recycled Sludge Concentration Predicted by the UNO 1D Activated Sludge Model

Recycled sludge concentration plays an important role in the design and operation of a wastewater treatment plant because it defines the value of the reactor MLSS concentration. Figure 4.3 shows the relationship between the observed and the predicted recycled sludge concentration. The line of best fit has a coefficient of determination (R^2) equal to 0.78 was

obtained. These results show that 78% of the variability of the data could be explained using the following equation:

$$X_{predicted} = 1.3203.X_{observed} \quad \text{.....(4.5)}$$

As in the case of X, there is an overprediction of X_R by the model. As indicated before, since both X and X_R are directly proportional, the overprediction of the X_R (32%) is very similar to the overprediction of X (33%)

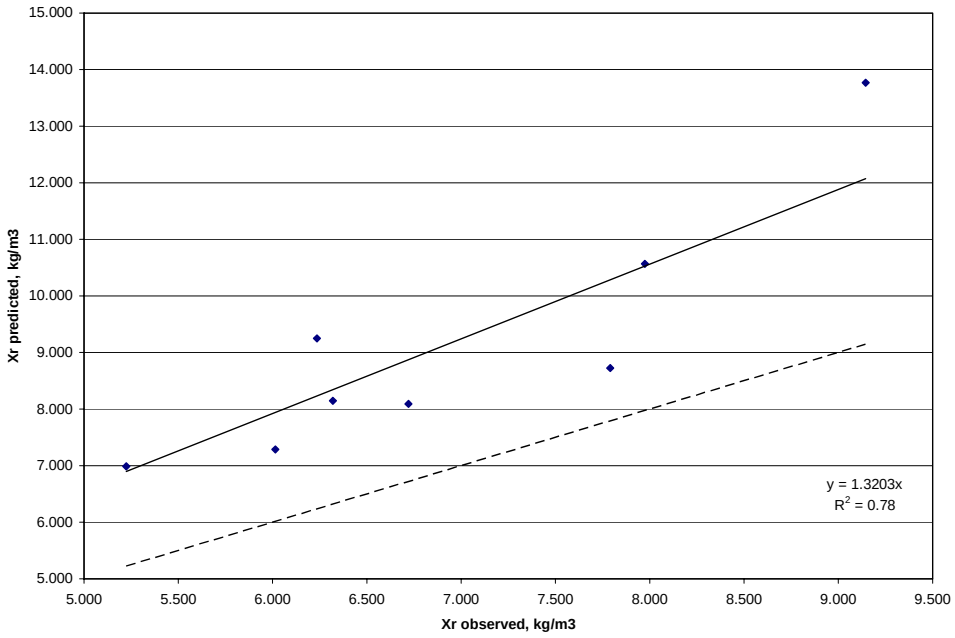

Figure 4.3 Relationship between the X_R Concentration Observed in the Field and the X_R Concentration Predicted during a HRT of 45 min

On the other hand, the data collected during the HRT of 30 min in the recycled line did not provide the expected results due to the insufficient amount of oxygen injected into the aeration

basin. Figure 4.4 illustrates the results and shows that the coefficient of determination (R^2) obtained for these data were low (0.5975). The line of best fit is the following:

$$X_{R\,predicted} = 1.49.X_{R\,observed} \quad \text{.....(4.6)}$$

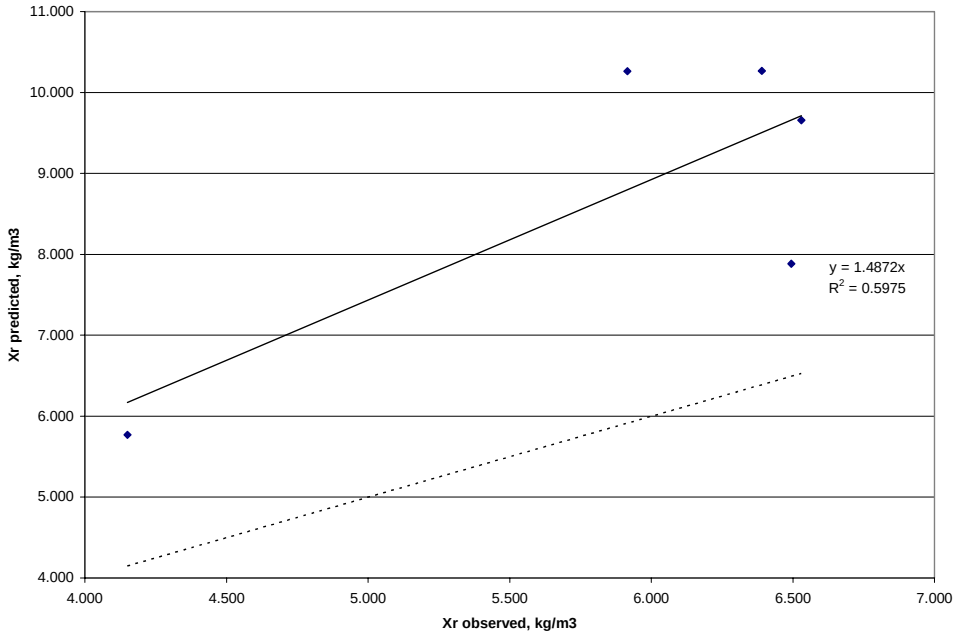

Figure 4.4 Relationship between the X_R Concentration Observed in the Field and the X_R Concentration Predicted during a HRT of 30 min

4.3 Relationship Between the Solids Retention Time Observed in the Filed and the Solids Retention Time Predicted by the UNO 1D Activated Sludge Model

The solids retention time is a very important parameter for operation and design of the activated sludge units. Figures 4.5 and 4.6 were plotted to determine if there exists any relationship between the data collected and the values predicted by the model.

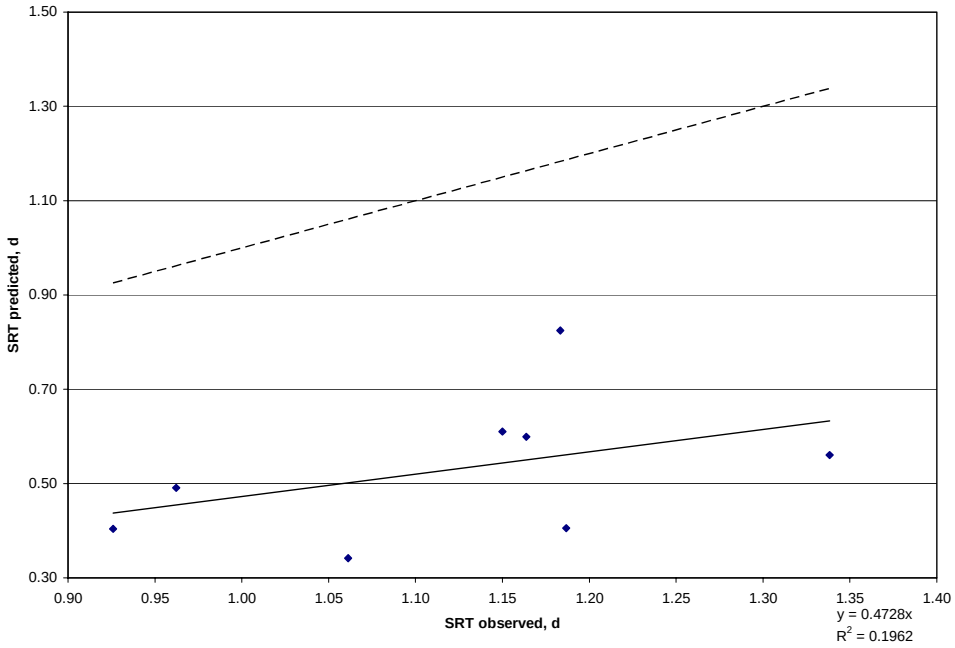

Figure 4.5 Relationship between the SRT Observed in the Field and the SRT predicted during a HRT of 45 min

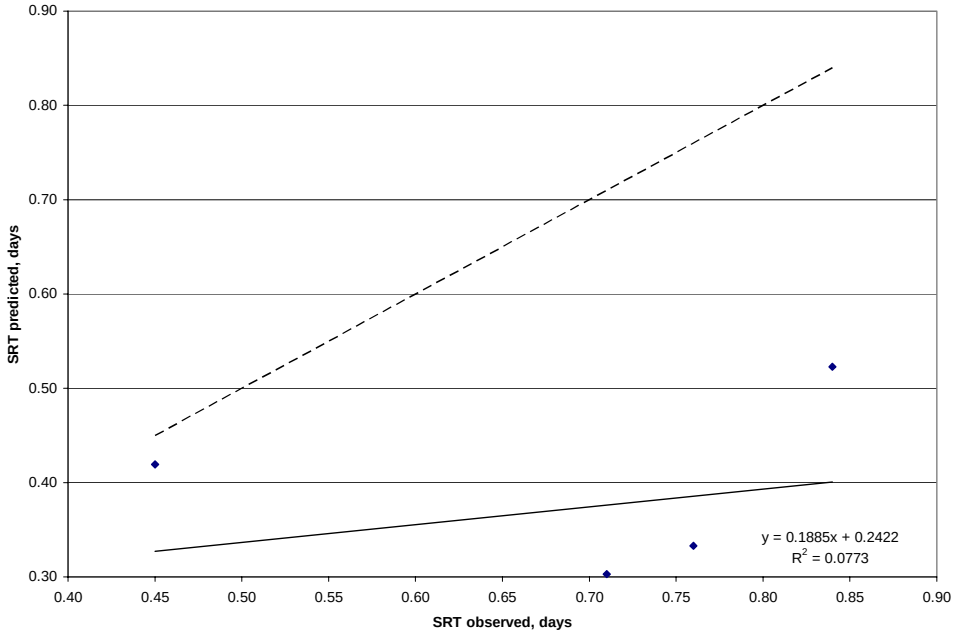

Figure 4.6 Relationship between the SRT Observed in the Field and the SRT Predicted during a HRT of 30 min

Figure 4.5 demonstrate that there is low correlation between the SRT predicted and the SRT observed. The coefficient of determination (R^2) is 0.1962 which means that just only the 19.62 % of the variability of the data could be explain in equation 4.5.

$$SRT_{predicted} = 0.47.STRT_{observed} \quad \text{.....(4.7)}$$

Although the correlation is low, Table 4.4 and Equation 4.5 show that the predicted value of SRT is roughly one half of the SRT actually used to operate the system. Figure 4.6 demonstrate that there is not a statistically significant correlation between the predicted SRT and observed SRT when a HRT of 30 min was used. The lack of correlation observed in Figure 4.5 is consistent with the poor performance of the unit due to low DO levels and with the low correlations observed before for X and X_R.

It is interesting to note that the settling tank volume has an important effect in defining the magnitude of the SRT value. This can be clearly seen in the equation used by the UNO 1D Activated Sludge Model:

$$\bar{t}_c = \frac{\bar{t}.X.\left(1+\frac{V_s}{V_r}\right)}{X-\alpha.(X_R - X)} \quad \text{.....(4.8)}$$

To test the effect of Vs on SRT, Vs was changed by increments of 0.5 m^3, and the results of the predicted values of SRT are given in table 4.5.

Table 4.5 SRT predicted by the UNO 1D Activated Sludge Model increasing the SST volume

		Settling Tank's Volumes (m^3)					
HRT (min)	**SRT observed (days)**	0.14537	0.64537	1.14537	1.64537	2.14537	2.64537
		SRT Predicted (days)					
45	0.93	0.40	1.03	1.66	2.29	2.92	3.55
45	0.96	0.49	1.26	2.02	2.79	3.55	4.32
45	1.15	0.61	1.56	2.51	3.47	4.42	5.37
45	1.18	0.82	2.11	3.40	4.68	5.97	7.25
45	1.19	0.41	1.04	1.67	2.30	2.94	3.57
45	1.06	0.34	0.87	1.41	1.94	2.47	3.01
45	1.16	0.60	1.54	2.48	3.42	4.36	5.29
45	1.34	0.56	1.44	2.31	3.19	4.06	4.94
30	0.71	0.30	0.78	1.25	1.72	2.19	2.67
30	0.59	0.26	0.68	1.09	1.50	1.91	2.32
30	0.84	0.52	1.34	2.15	2.97	3.78	4.60
30	0.45	0.42	1.07	1.73	2.38	3.04	3.69
30	0.76	0.33	0.85	1.37	1.89	2.41	2.93

It is important to mention that decreasing the recirculation ratio (α) could also increase the SRT of the system. However, making changes of the recirculation ratio will affect the value of the MLSS concentration and the sludge recycled concentration, with just minor changes in SRT.

4.4. Analysis of the Effluent Concentration and COD removal observed in the Field and the Effluent Concentration and COD Removal predicted by the Model

Total suspended solids and TCOD in the effluent of the secondary settling tank were measured to determine the amount of solids and organic matter. Table 4.6 shows the values observed in the system and the values predicted by the model.

Table 4.6 Effluent Total Suspended Solids and TCOD Concentrations Observed in the Field and Predicted by the Model during HRT of 45 and 30 min in kg/m^3

MLSS	TSS observed	TSS predicted	TCOD observed	TCOD predicted
1.978	0.0230	0.0056	0.0400	0.0240
2.376	0.0140	0.0038	0.0210	0.0254
2.470	0.0090	0.0033	0.0380	0.0251
2.836	0.0180	0.0042	0.0260	0.0283
3.014	0.0280	0.0032	0.0400	0.0230
3.242	0.0270	0.0042	0.0580	0.0241
3.348	0.0140	0.0022	0.0480	0.0240
4.645	0.0260	0.0033	0.0510	0.0266
2.022	0.0370	0.0093	0.0690	0.0296
2.600	0.0660	0.0138	0.0750	0.0279
3.093	0.0280	0.0052	0.0720	0.0278
3.154	0.0420	0.0086	0.0850	0.0247
3.515	0.0630	0.0110	0.0790	0.0271

Unfortunately, a low correlation between the effluent TSS observed and the effluent TSS predicted by the model for the two different HRT of 45 and 30 min was found. However, the observed values at HRT of 45 min were between 0.009 and 0.028 kg/m^3 and the predicted values were between 0.0022 and 0.0056 kg/m^3, i.e. within the same order of magnitude.

In the same fashion, it could not found a relationship between the TCOD observed in the field and the TCOD removal predicted by the model but they also are in the same order of magnitude. For HRT of 30 min, the same scenario is presented

A sensitivity analysis of some of the kinetic constants was developed when the system was operated with HRT of 45 min. The idea this analysis was to verify if changing of these constants, the values predicted by the model will improve. However, changing some of the kinetics constants does not significantly improve the predicted values. Table 4.7 shows the best group of kinetic constants found during the sensitivity analysis.

Table 4.7 Kinetics Constants

Kinetic Constant	Value	Unit
k_p	916.26	m/d
k_d	0.06	d^{-1}
k_x	289.85	m^3.kg
Y	0.5	kg (VSS) / kg (DCOD)
a_p	0.016	Kg/m^3
a_x	0.000009	Kg/m^3
k_g	276	Kg/m^3
k_{gp}	150	Kg/m^3

4.5. Relationship between the MLSS concentration and the empirical parameter *n*.

Settling tests were performed every time the samples were collected during the experiments, and the results are presented in Appendix E. It is interesting to observe that at a HRT of 45 min, a linear relationship between the MLSS concentration and one of the sludge settling characteristics (*n*) could be observed. Figure 4.7 shows a very good correlation between the parameters with a coefficient of determination (R^2) of 0.919.

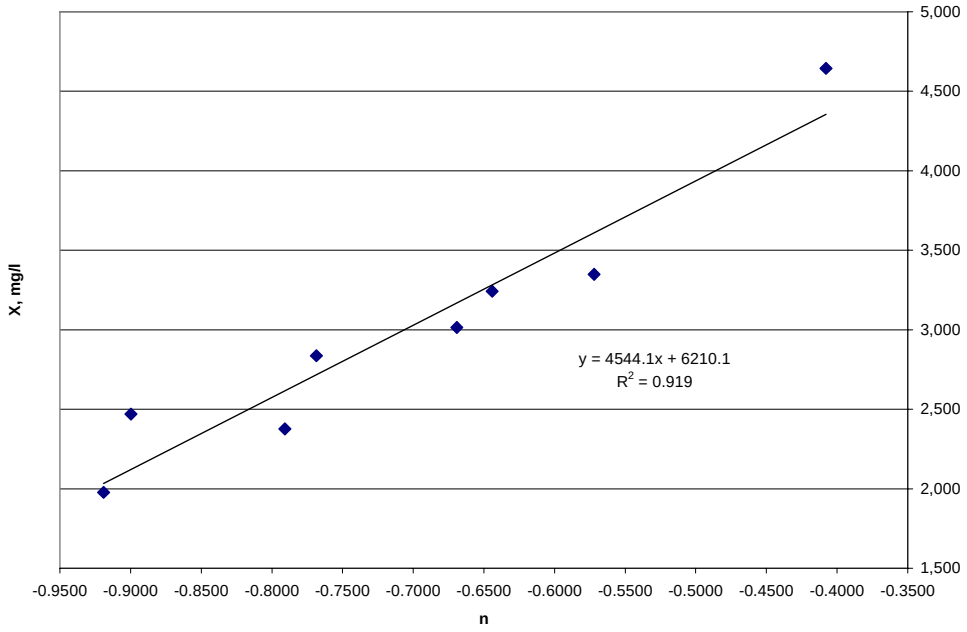

Figure 4.7 Relationship between the MLSS concentrations and the empirical parameter *n* during a HRT of 45 min

This plot allows to conclude that a value of *n* can be predicted once a MLSS concentration for the design or operation of an activated sludge system has been selected. The empirical parameter *n* is very sensitive and changes significantly when the concentration in the aeration basin changes. Using this graph the value of *n* could be predicted without performing the sludge settling test, which is time consuming. Therefore, this is a very helpful tool for both design and operation of activated sludge units.

Figure 4.8 presents similar data corresponding to a HRT of 30 min. In this case, the coefficient of determination dropped to 0.73.

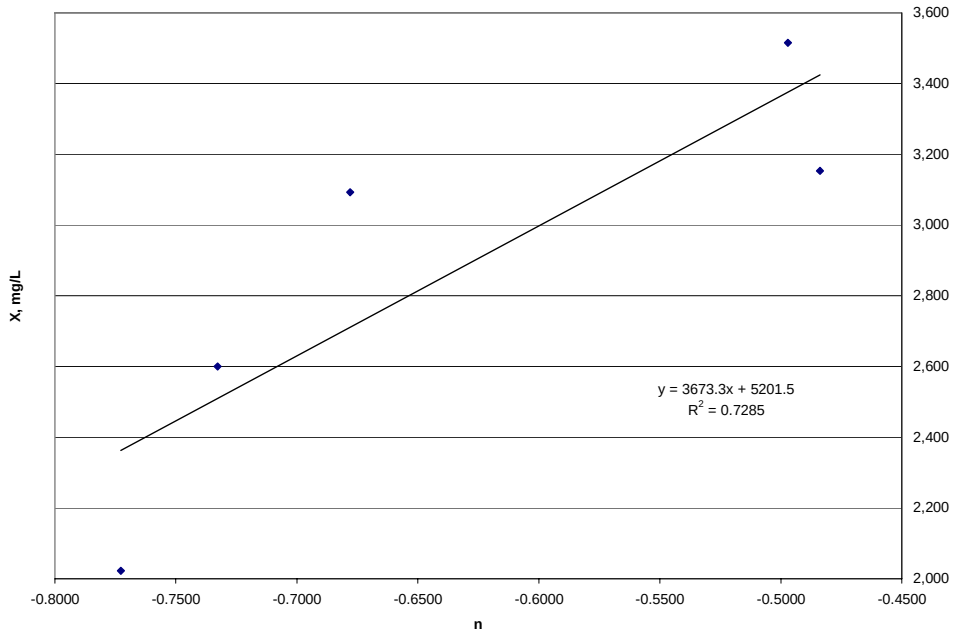

Figure 4.8 Relationship between the MLSS concentrations and the empirical parameter *n* during a HRT of 30 min

4.6. Relationship between the MLSS concentration and the Sludge Settling Velocity, V_0.

A linear relationship between the MLSS concentration and one of the sludge settling characteristics V_0 is presented when the system was operated at HRT of 45 min. Figure 4.9 shows a good correlation between the parameters with a coefficient of determination (R^2) of 0.7388.

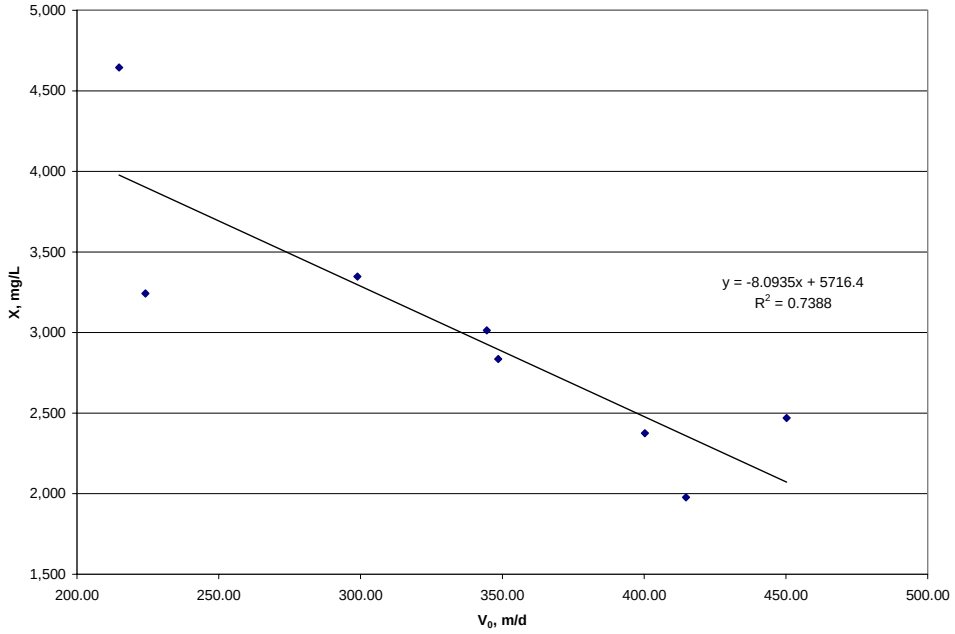

Figure 4.9 Relationship between the MLSS concentrations and the empirical parameter V_0 during a HRT of 45 min

This plot also allows to conclude that a value of V_0 can be predicted once a MLSS concentration for the design or operation of an activated sludge system has been selected. The linear of best fit shows that higher velocities are reached when low concentrations in the aeration basin are selected for operating and design purposes. Therefore, this is a very helpful tool for both design and operation of activated sludge units.

Figure 4.10 presents the data corresponding to HRT of 30 min. In this case, the coefficient of determination dropped to 0.0508. This low correlation can be attributed to the lack of DO in the system, which resulted a poor flocculation in the system. Good settling characteristics can just be obtained when a good flocculation is reached. Therefore, the reliability of this data is questionable.

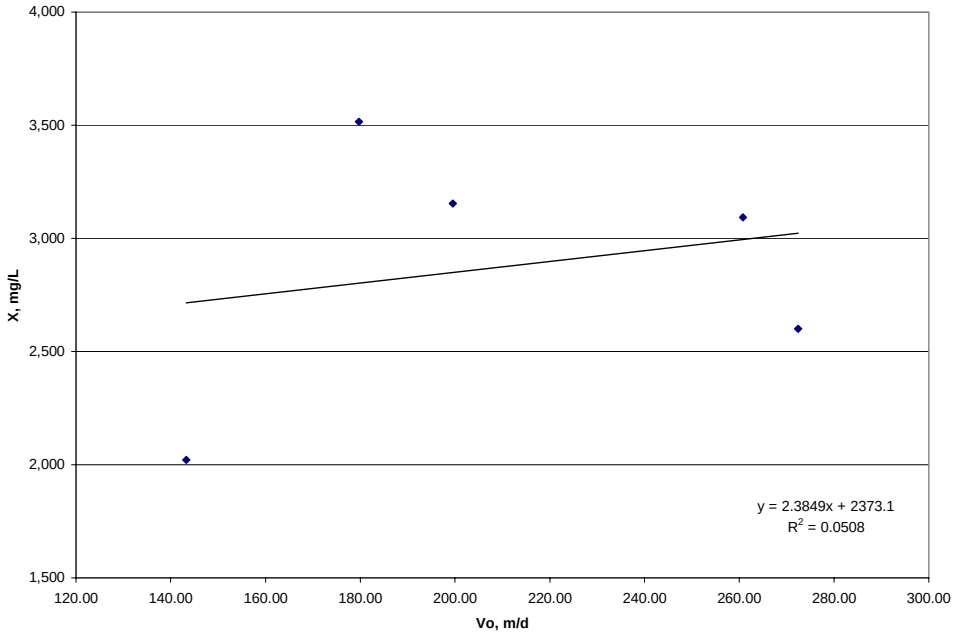

Figure 4.10 Relationship between the MLSS concentrations and the empirical parameter V_0 during a HRT of 30 min

4.7 Accumulation of SS concentration with time.

According to La Motta (2004), equation 2.18 gives the equilibrium of solids in the system without accumulation of solids. The UNO 1D Activated Sludge Model was developed based on this condition. However, it was noticed that the values of X that were measured during the sampling period increased with time, thus showing accumulation. Indeed, figures 4.11 and 4.12 demonstrate an almost linear accumulation of biomass in the aeration basin. This is a response to the fact that the system was operated using a SRT that was more than double the theoretical SRT corresponding to equilibrium conditions. Therefore, the system was not operated at steady state, and it could not operate at the short theoretical SRT without causing a significant system disruption. In fact, several attempts to operate the unit using the theoretical SRT resulted in complete system failure due to poor flocculation and settling characteristics.

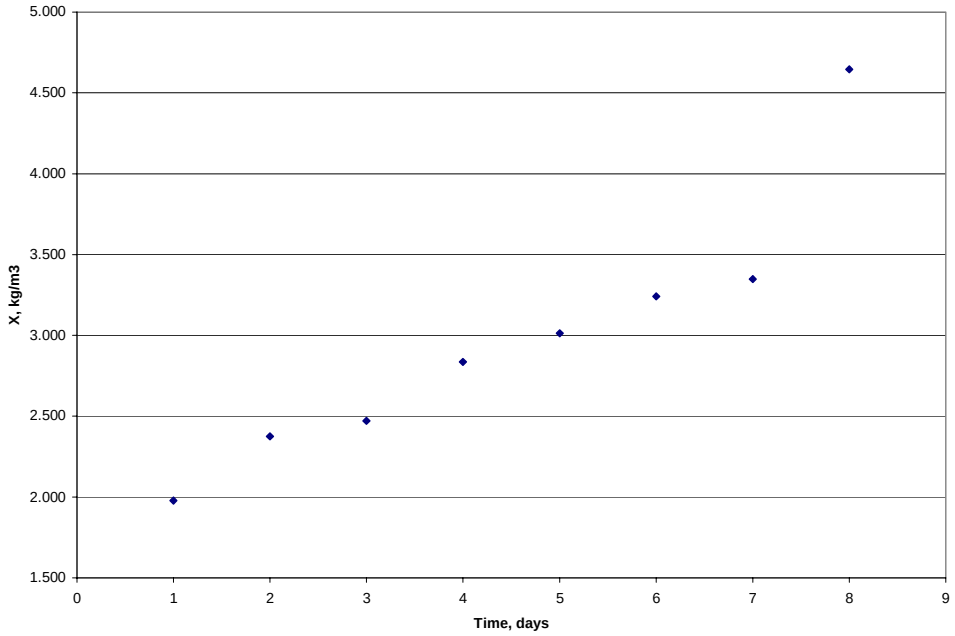

Figure 4.11 MLSS concentration vs. time during HRT of 45 min

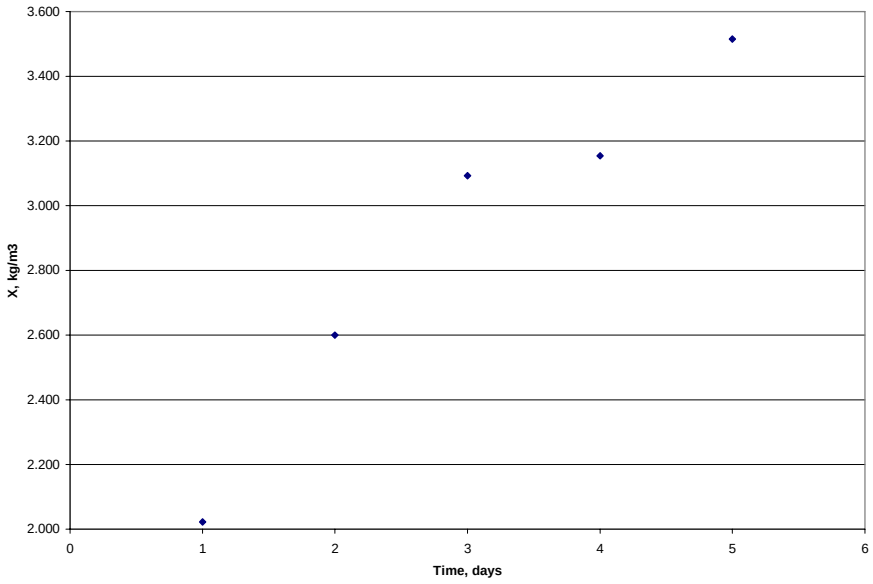

Figure 4.12 MLSS concentration vs. time during HRT of 30 min

The observed increase of suspended solids concentrations in the reactor is also due to the accumulation of sludge in the secondary settling tank during the operation of the system. This accumulation can be seen in figures 4.13 and 4.14 where the recycled line suspended solids concentration is observed to increases with time. Again, this behavior can be attributed to the SRT used for the system operation, which was larger than the theoretical value.

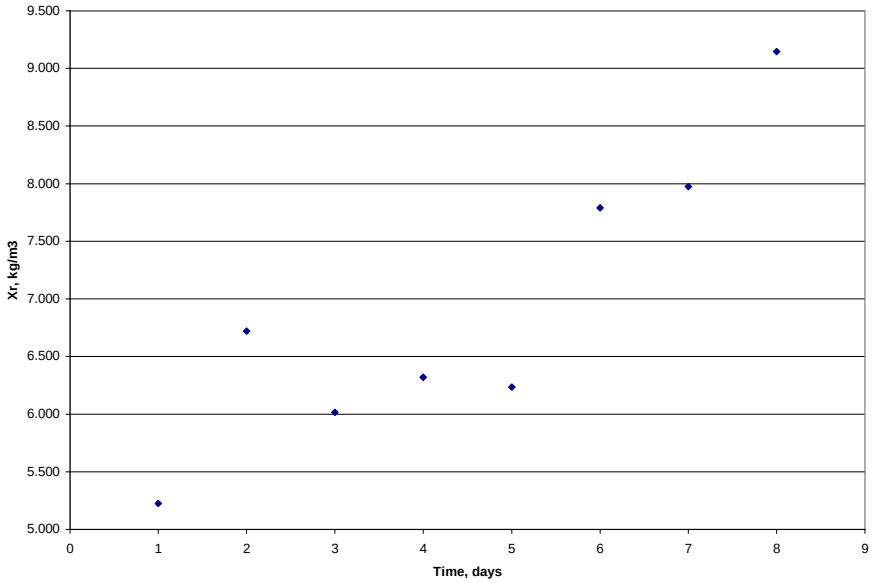

Figure 4.13 Recycled Sludge concentration vs. Time during HRT of 45 min

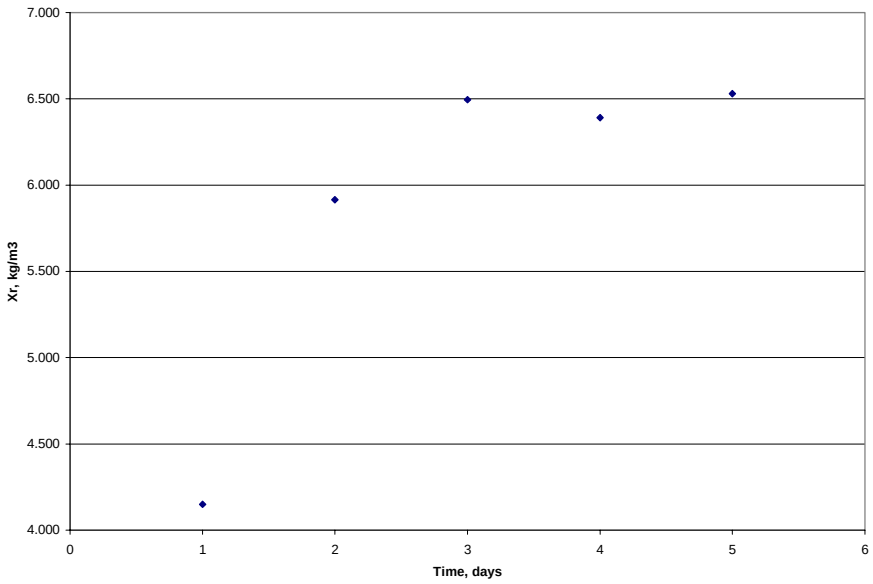

Figure 4.14 Recycled Sludge Concentration vs. Time during HRT of 30 min

CHAPTER V

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The following conclusions can be drawn from this research project:

- The UNO 1D Activated Sludge Model is very useful predictive tool of the actual operating conditions of the pilot plant regarding both MLSS and the recycled sludge concentration. The model overpredicts both X and X_R by factor of approximately 1.3.
- The limiting flux theory results in an overprediction of the actual values of X and X_R.
- For design purposes, the volume of the secondary settling tank should be used in computing the SRT. Unless the secondary settling tank has adequate volume, it is impossible to achieve values of SRT recommended in the literature (5-10 days) just based on the aerator volume.
- Unless the SRT used for operation is equal to the theoretical SRT, it is very difficult to reach steady state conditions (equilibrium in the system) in an activated sludge system because sludge accumulation will result in the secondary settling tank and in the aerator.
- The empirical settling parameter *n* varies significantly as a function of the MLSS concentration. This sludge settling characteristic is very sensitive and is very important to be considered during the design and operating process. Unless the system operates at equilibrium (constant MLSS) the sludge settling characteristics will vary with time.
- Based on an optimum MLSS concentration of between 250 and 3500 mg/L, it can be concluded that the ideal value of *n* for a design and operation of an activated sludge system

should be in the range of -0.8165 and -0.5964 when the system is operated with HRT of 45 min and a range of -0.7354 and -0.4632 with HRT of 30 min.

- The dissolved oxygen concentration is a very important parameter for the successful operation of an activated sludge system. Only with stable and adequate DO levels it is possible to obtain good flocculation and good settling characteristics.

5.2 Suggestions for Further Research

- Improve this study using data from lower HRT to verify if the UNO 1D Activated Sludge Model behaves in the same manner.
- Perform a complete sensitivity analysis with the first order constants of PCOD and TSS flocculation and growth (k_p, k_x, k_{gp}, k_g), the endogenous respiration coefficient (k_d), the yiled coefficient (Y) and the kinetic parameters of PCOD and TSS flocculation (a_p, a_x) to determine if there exist any values of this group of parameters that can be used for design purpose.
- Change the volume of the pilot-plant settling tank so that the operating SRT can be equal to the theoretical SRT necessary for good flocculation (a minimum of 3 days), and equilibrium conditions can be achieved.
- Complete this model to 2D hydrodynamic clarifiers.
- Analyze the model in a full scale activated sludge system.

CHAPTER VI

6. References

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APPENDIX A

Picture A-1 Rotating Screen

Picture A-2 Aeration Basin

Picture A -3 Secondary Clarifier

APPENDIX B

Table B-1 Field Sheet, HRT 45 min (Day 1)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Thursday, July 29, 2004

Parameters	Value	Units
DO	3.52	mg/l
T	30.1	ºC
pH	7.31	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	414.8	m/d
n	- 0.9192	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	15.43	5599.482	on	off
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-2 Field Sheet, HRT 45 min (Day 2)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Friday, July 30, 2004

Parameters	Value	Units
DO	4.61	mg/l
T	30	ºC
pH	7.31	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	400.3	m/d
n	-0.7907	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	15.43	5599.482	on	off
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-3 Field Sheet, HRT 45 min (Day 3)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Saturday, July 31, 2004

Parameters	Value	Units
DO	4.1	mg/l
T	30	ºC
pH	7.27	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	450.24	m/d
n	-0.8998	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	15.43	5599.482	on	off
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-4 Field Sheet, HRT 45 min (Day 4)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Sunday, August 01, 2004

Parameters	Value	Units
DO	3.51	mg/l
T	30.2	ºC
pH	7.36	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	348.48	m/d
n	-0.7684	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	15.43	5599.482	on	off
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-5 Field Sheet, HRT 45 min (Day 5)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Monday, August 02, 2004

Parameters	Value	Units
DO	4.51	mg/l
T	30.2	ºC
pH	7.33	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	344.54	m/d
n	-0.6692	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
				on	off
Influent	1	15.43	5599.482		
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-6 Field Sheet, HRT 45 min (Day 6)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Tuesday, August 03, 2004

Parameters	Value	Units
DO	1.06	mg/l
T	30.2	ºC
pH	7.36	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	224.15	m/d
n	-0.6442	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
				on	off
Influent	1	15.43	5599.482		
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-7 Field Sheet, HRT 45 min (Day 7)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Wednesday, August 04, 2004

Parameters	Value	Units
DO	4.2	mg/l
T	29.9	ºC
pH	7.28	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	298.91	m/d
n	-0.5721	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	15.43	5599.482	on	off
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-8 Field Sheet, HRT 45 min (Day 8)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Thursday, August 05, 2004

Parameters	Value	Units
DO	1.34	mg/l
T	29.9	ºC
pH	7.24	
Vr	175	L
Vs	145.37	L
Alpha	0.701	
w	0.019	
Vo	214.85	m/d
n	-0.408	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	15.43	5599.482	on	off
Recirculation	1	7	3927.27	56	120
Waste	1	3	106.27	40	10800

Table B-9 Field Sheet, HRT 30 min (Day 1)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Thursday, August 12, 2004

Parameters	Value	Units
DO	1.32	mg/l
T	28.7	ºC
pH	7.28	
Vr	175	L
Vs	145.37	L
Alpha	0.712	
w	0.017	
Vo	143.29	m/d
n	-0.7727	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	10.3	8388.35	on	off
Recirculation	1	4.6	5976.28	56	120
Waste	1	3	145.92	55	10800

Table B-10 Field Sheet, HRT 30 min (Day 2)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Friday, August 13, 2004

Parameters	Value	Units
DO	0.9	mg/l
T	32	ºC
pH	7.3	
Vr	175	L
Vs	145.37	L
Alpha	0.712	
w	0.017	
Vo	272.43	m/d
n	-0.7328	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
Influent	1	10.3	8388.35	on	off
Recirculation	1	4.6	5976.28	56	120
Waste	1	3	145.92	55	10800

Table B-11 Field Sheet, HRT 30 min (Day 3)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Saturday, August 14, 2004

Parameters	Value	Units
DO	2.69	mg/l
T	30.6	ºC
pH	7.28	
Vr	175	L
Vs	145.37	L
Alpha	0.712	
w	0.017	
Vo	260.77	m/d
n	-0.678	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
				on	off
Influent	1	10.3	8388.35		
Recirculation	1	4.6	5976.28	56	120
Waste	1	3	145.92	55	10800

Table B-12 Field Sheet, HRT 30 min (Day 4)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Sunday, August 15, 2004

Parameters	Value	Units
DO	1.26	mg/l
T	31.2	ºC
pH	7.26	
Vr	175	L
Vs	145.37	L
Alpha	0.712	
w	0.035	
Vo	199.57	m/d
n	-0.4838	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
				on	off
Influent	1	10.3	8388.35		
Recirculation	1	4.6	5976.28	56	120
Waste	1	3	290.38	110	10800

Table B-13 Field Sheet, HRT 30 min (Day 5)

FIELD DATA

Graduate Assistant: *Jose Angel Rojas*

Date: Monday, August 16, 2004

Parameters	Value	Units
DO	1.58	mg/l
T	33	ºC
pH	7.27	
Vr	175	L
Vs	145.37	L
Alpha	0.712	
w	0.017	
Vo	179.69	m/d
n	- 0.4971	m3/kg

Description	Volume (L)	Time (s)	Q (L/d)	Timers (s)	
				on	off
Influent	1	10.3	8388.35		
Recirculation	1	4.6	5976.28	56	120
Waste	1	3	145.92	55	10800

APPENDIX C

Table C-1 Influent and Effluent characterization during the HRT of 45 and 30 min in the system

HRT	SRT	MLSS	Effluent (kg/m^3)				Influent (kg/m^3)	
(min)	(days)	(kg/m^3)	TSS	TCOD	DCOD	PCOD	DCOD	PCOD
45	0.93	1.978	0.023	0.040	0.008	0.032	0.045	0.212
45	0.96	2.376	0.014	0.021	0.009	0.012	0.058	0.243
45	1.15	2.470	0.009	0.038	0.015	0.023	0.048	0.208
45	1.18	2.836	0.018	0.026	0.013	0.014	0.047	0.108
45	1.19	3.014	0.028	0.040	0.012	0.028	0.052	0.309
45	1.06	3.242	0.027	0.058	0.022	0.036	0.052	0.304
45	1.16	3.348	0.014	0.048	0.017	0.031	0.055	0.326
45	1.34	4.645	0.026	0.051	0.026	0.025	0.108	0.330
30	0.71	2.022	0.037	0.069	0.021	0.048	0.074	0.267
30	0.59	2.600	0.066	0.075	0.040	0.035	0.039	0.364
30	0.80	3.093	0.028	0.072	0.022	0.050	0.080	0.279
30	0.45	3.154	0.042	0.085	0.038	0.047	0.026	0.395
30	0.76	3.515	0.063	0.079	0.041	0.038	0.046	0.376

Table C-2 MLSS Supernatant and Mixture Supernatant Characterization during the HRT of 45 and 30 min in the system

MLSS	MLSS Supernatant (kg/m^3)				Mixture's Supernatant (kg/m^3)	
(kg/m^3)	TSS	TCOD	DCOD	PCOD	DCOD	PCOD
1.978	0.014	0.037	0.014	0.024	0.050	0.003
2.376	0.014	0.017	0.010	0.007	0.058	0.057
2.470	0.004	0.026	0.025	0.001	0.058	0.035
2.836	0.021	0.026	0.014	0.012	0.043	0.035
3.014	0.015	0.034	0.012	0.022	0.046	0.022
3.242	0.027	0.048	0.017	0.031	0.064	0.051
3.348	0.003	0.043	0.026	0.018	0.042	0.096
4.645	0.018	0.052	0.023	0.029	0.068	0.050
2.022	0.040	0.067	0.020	0.047	0.064	0.088
2.600	0.046	0.064	0.035	0.030	0.038	0.196
3.093	0.028	0.055	0.020	0.035	0.067	0.067
3.154	0.041	0.077	0.039	0.038	0.044	0.164
3.515	0.038	0.079	0.035	0.044	0.029	0.224

Table C-3 Sludge Settling Parameters during Sampling Collection

MLSS (kg/m^3)	V_0 (m/d)	n (m^3/kg)
1.978	414.80	-0.9192
2.376	400.30	-0.7907
2.470	450.24	-0.8998
2.836	348.48	-0.7684
3.014	344.54	-0.6692
3.242	224.15	-0.6442
3.348	298.91	-0.5721
4.645	214.85	-0.4080
2.022	143.29	-0.7727
2.600	272.43	-0.7328
3.093	260.77	-0.6780
3.154	199.57	-0.4838
3.515	179.69	-0.4971

APPENDIX D

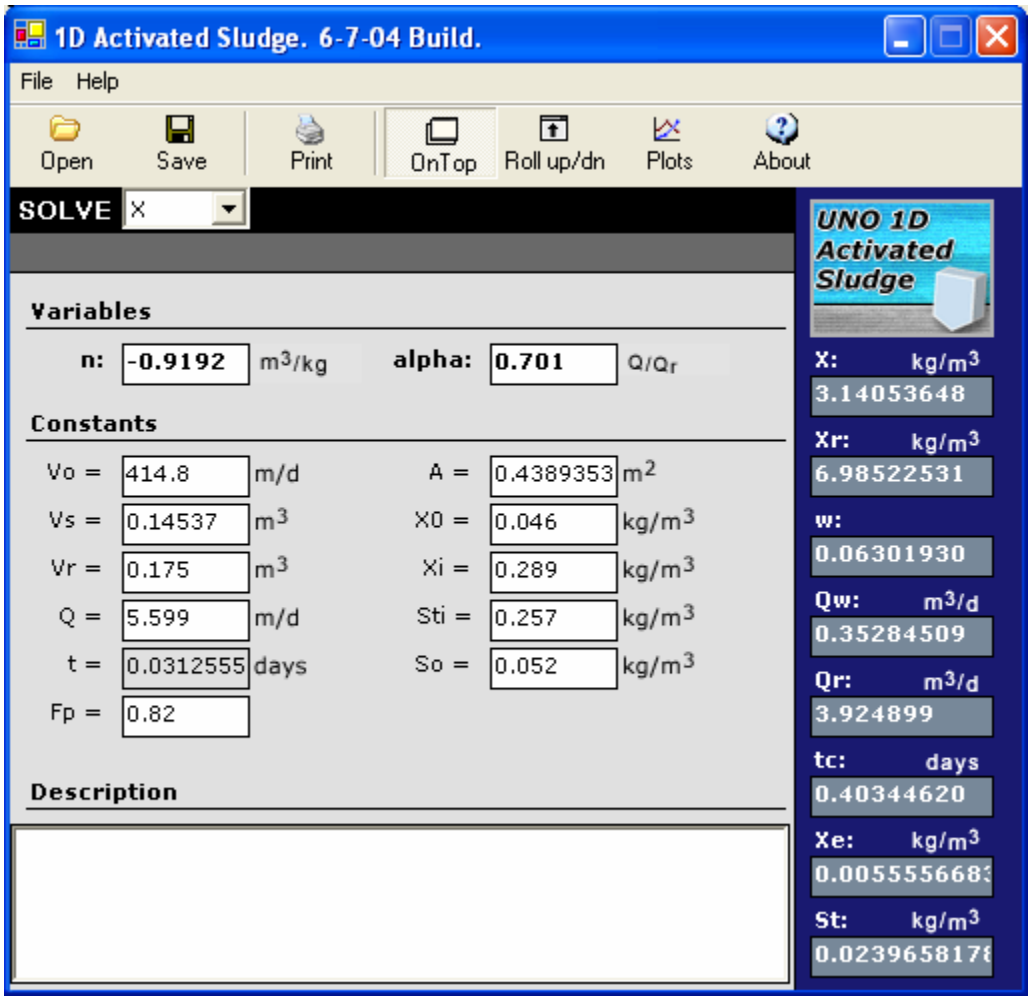

Figure D-1 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 1)

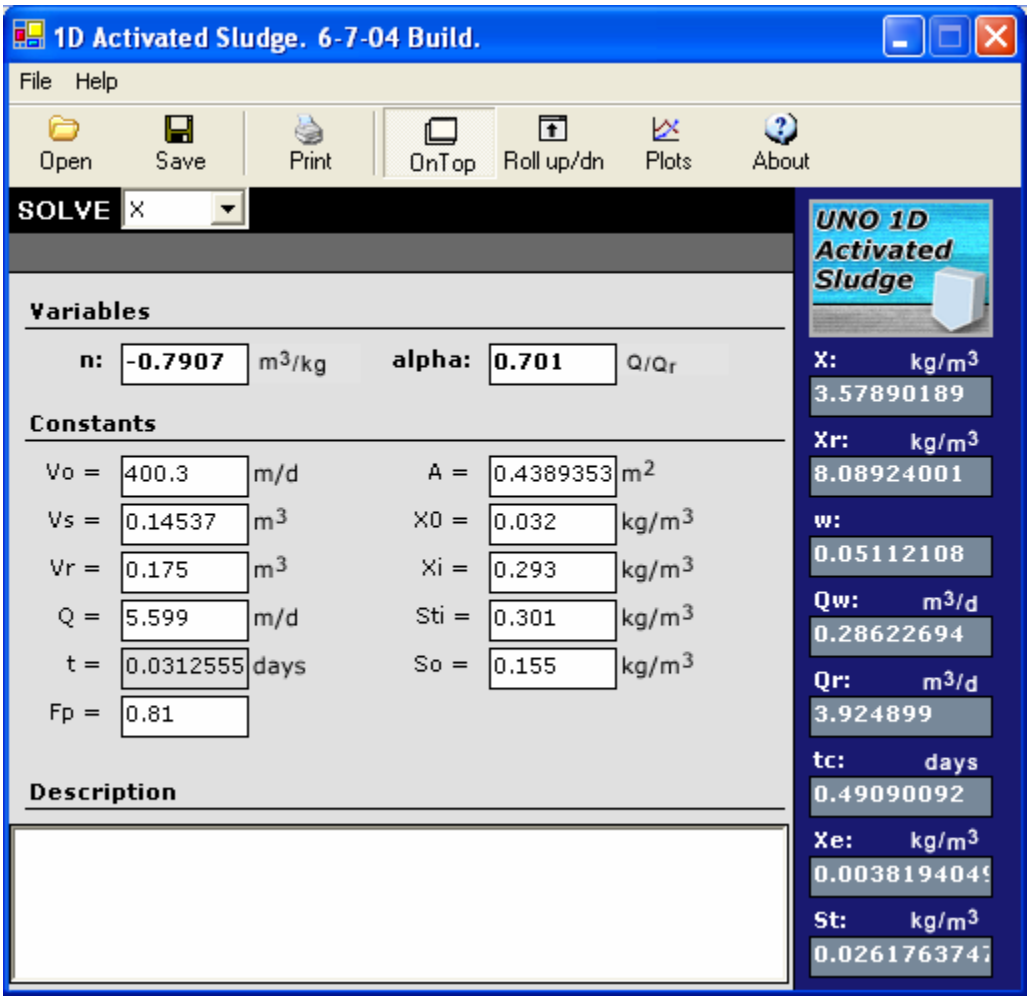

Figure D-2 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 2)

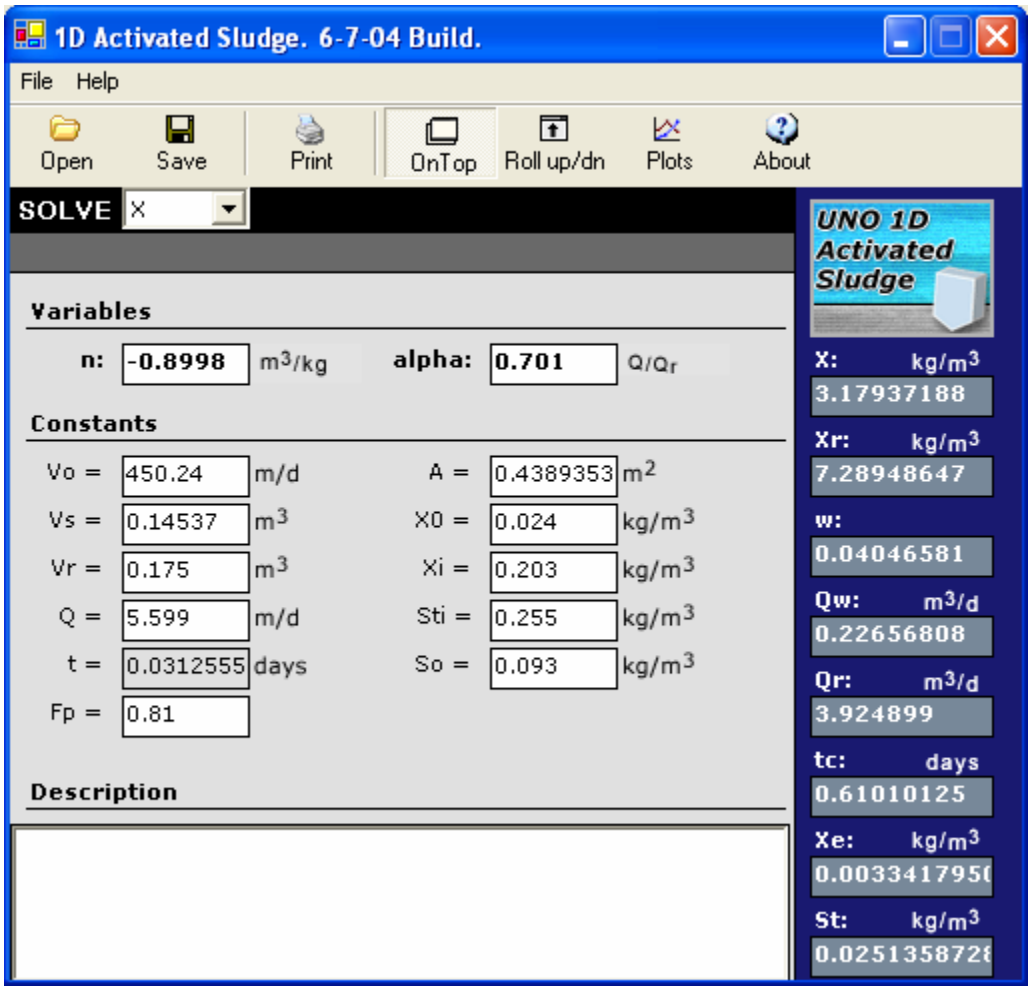

Figure D-3 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 3)

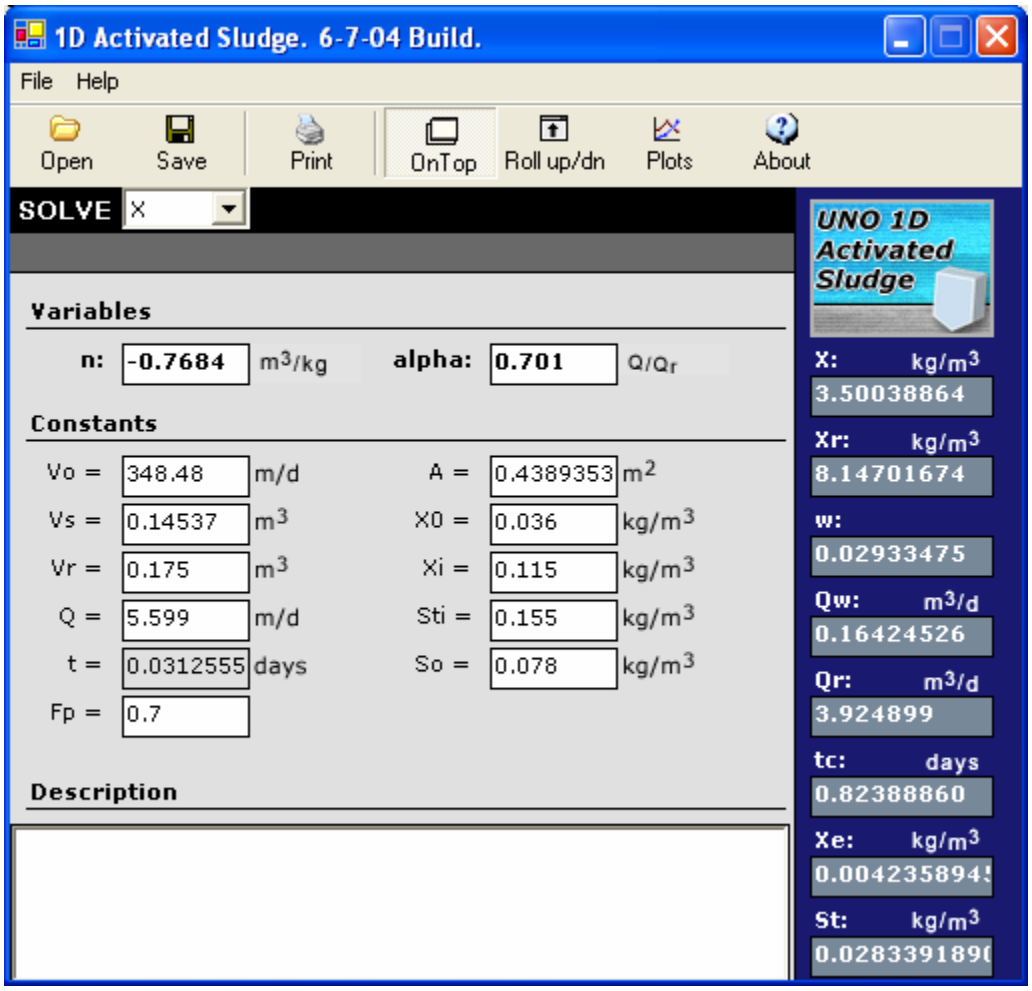

Figure D-4 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 4)

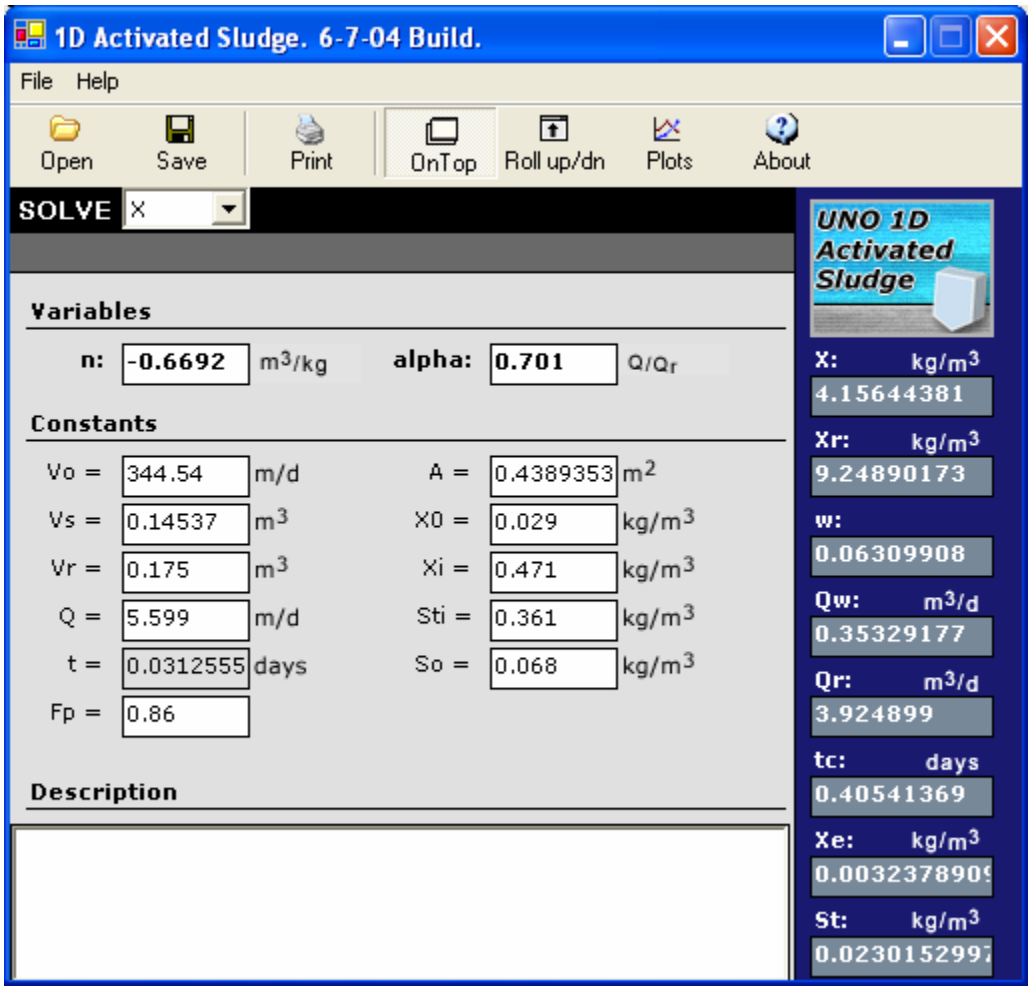

Figure D-5 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 5)

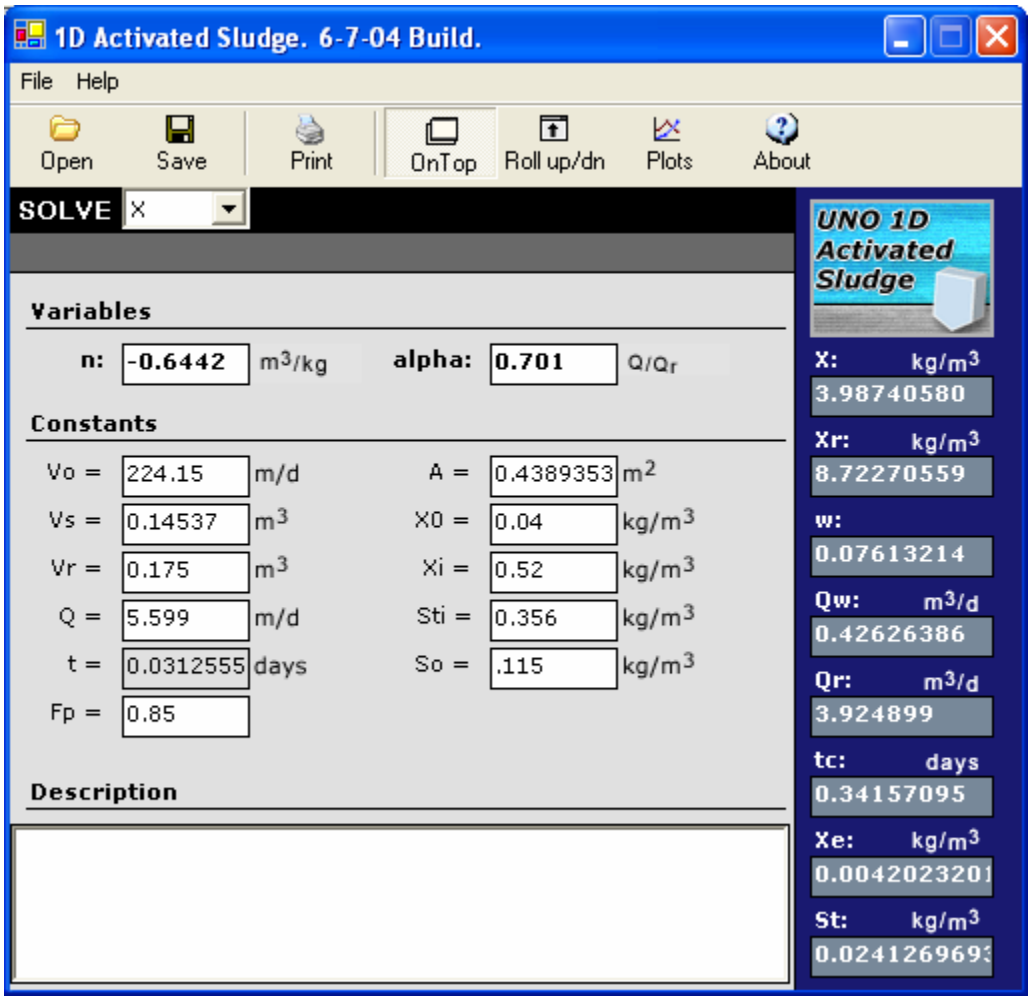

Figure D-6 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 6)

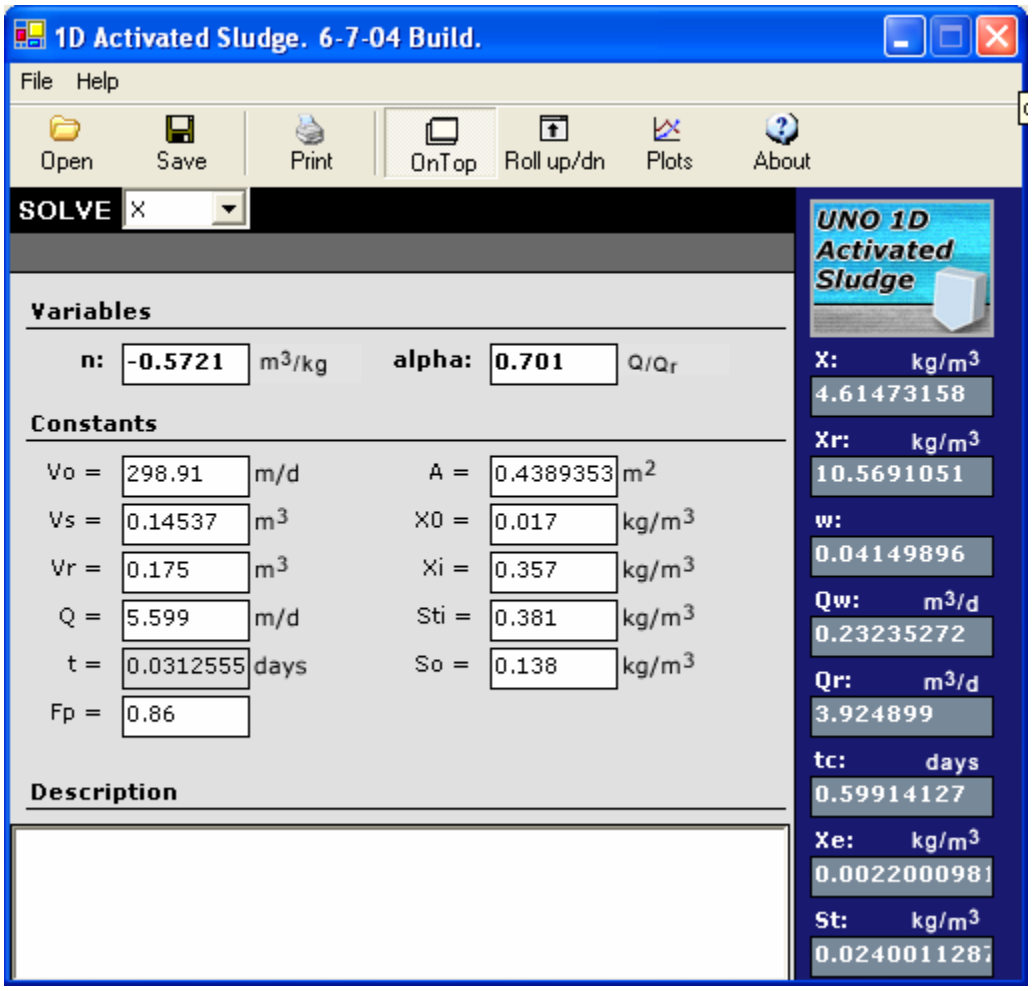

Figure D-7 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 7)

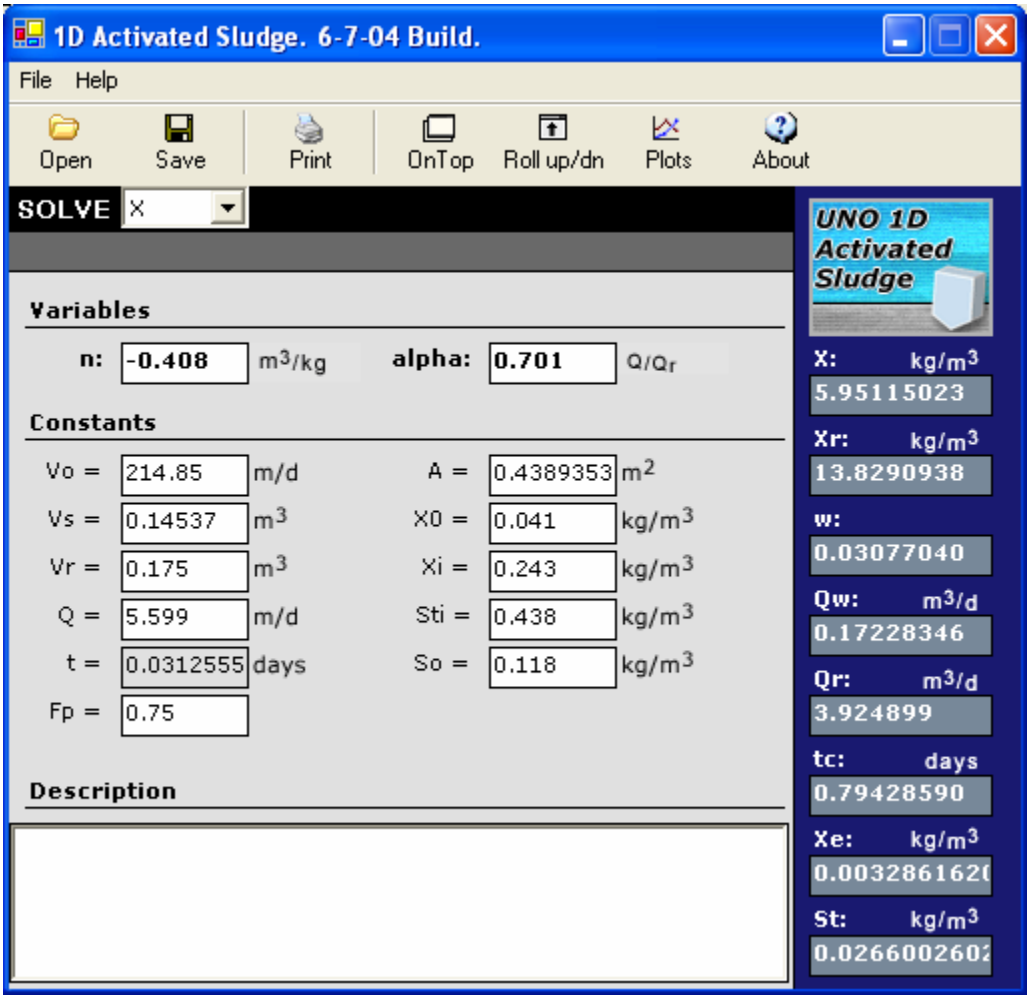

Figure D-8 Data predicted by the UNO 1D Activated Sludge Model, HRT 45 min (Day 8)

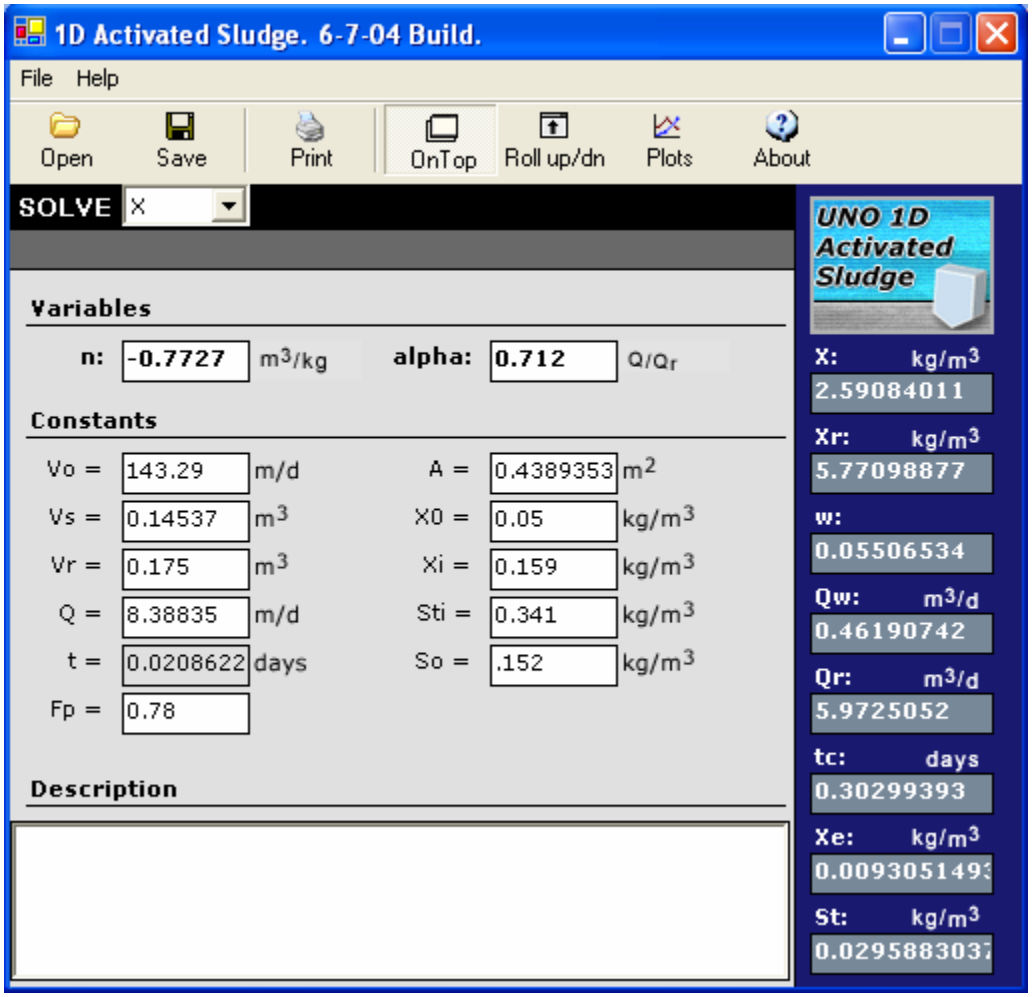

Figure D-9 Data predicted by the UNO 1D Activated Sludge Model, HRT 39 min (Day 1)

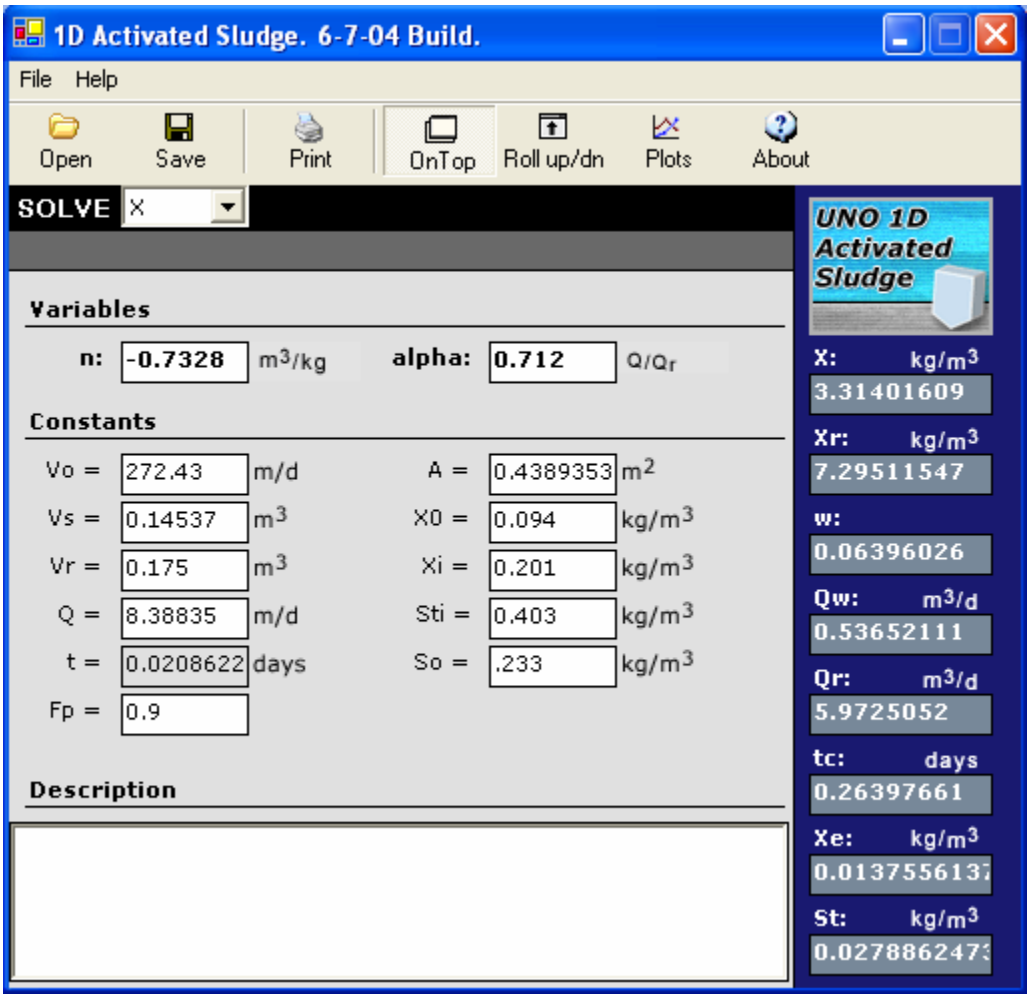

Figure D-10 Data predicted by the UNO 1D Activated Sludge Model, HRT 30 min (Day 2)

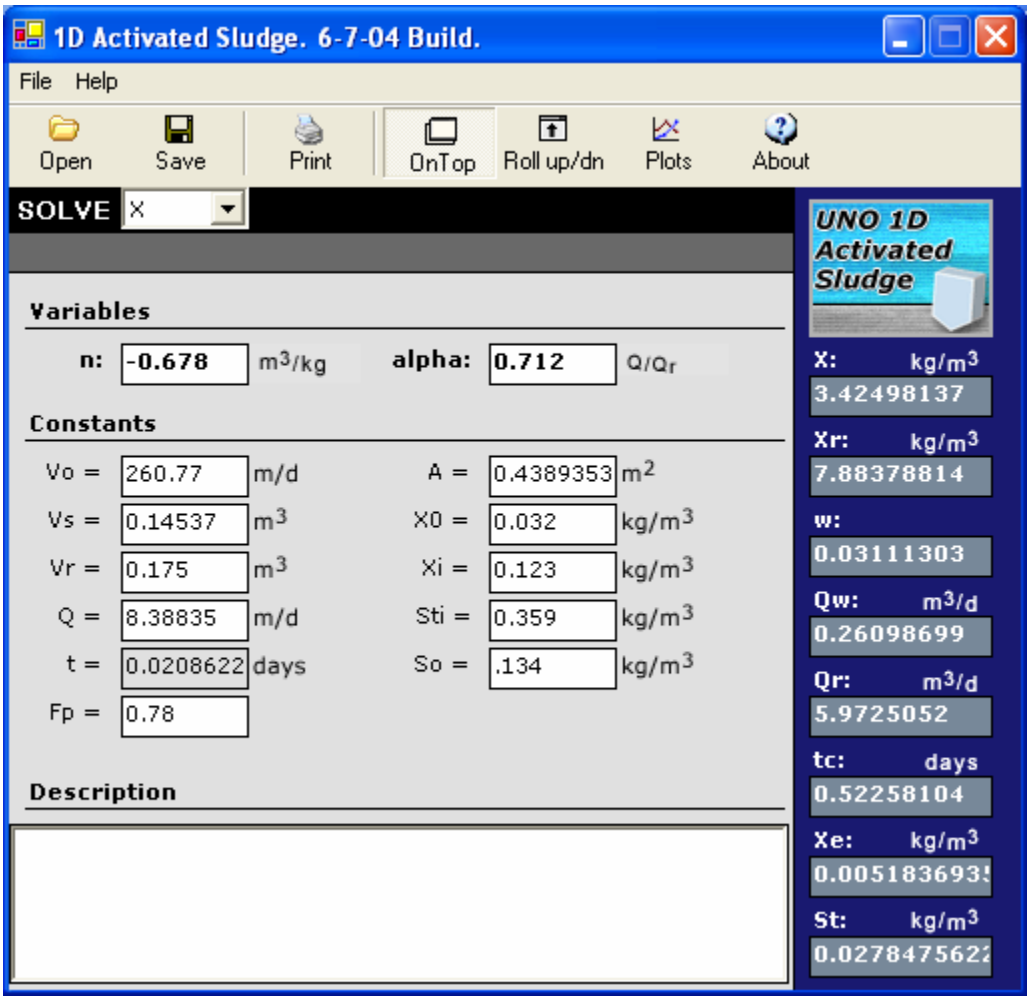

Figure D-11 Data predicted by the UNO 1D Activated Sludge Model, HRT 30 min (Day 3)

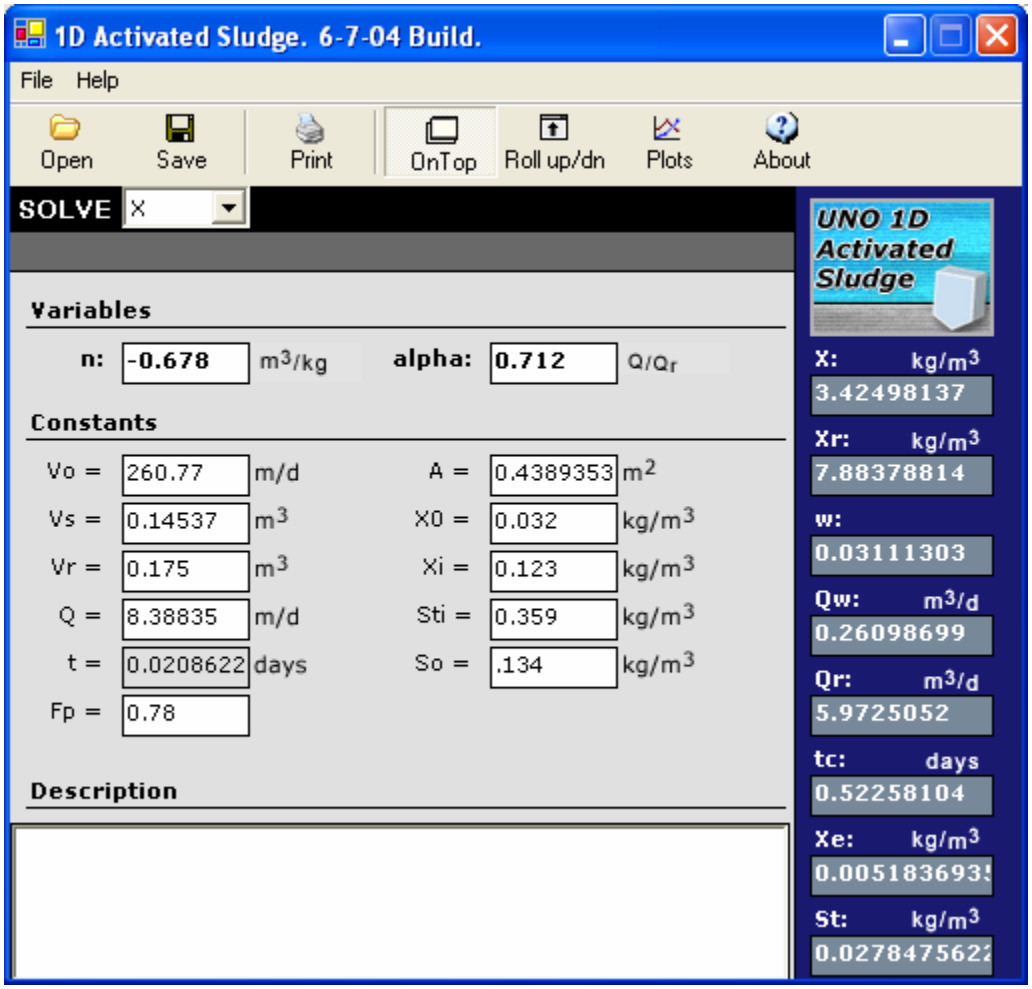

Figure D-12 Data predicted by the UNO 1D Activated Sludge Model, HRT 30 min (Day 4)

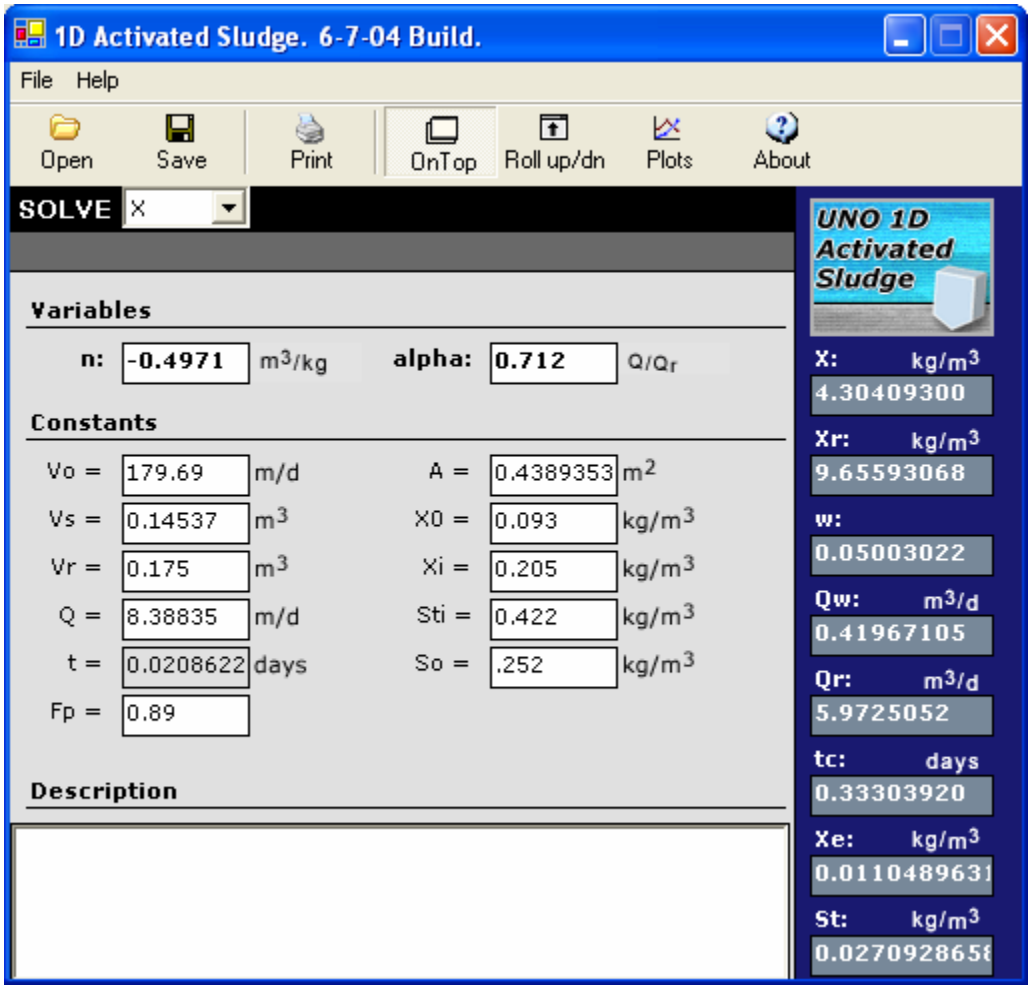

Figure D-12 Data predicted by the UNO 1D Activated Sludge Model, HRT 30 min (Day 4)

APPENDIX E

Table E-1 Settling Velocity Calculations, HRT 45 min (Day 1)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	1978	68.4	1.978
800 sludge / 200 water	1587.5	93.6	1.5875
700 sludge / 300 water	1382.5	118.08	1.3825
600 sludge / 400 water	1182.5	136.8	1.1825
500 sludge / 500 water	1022.5	165.6	1.0225

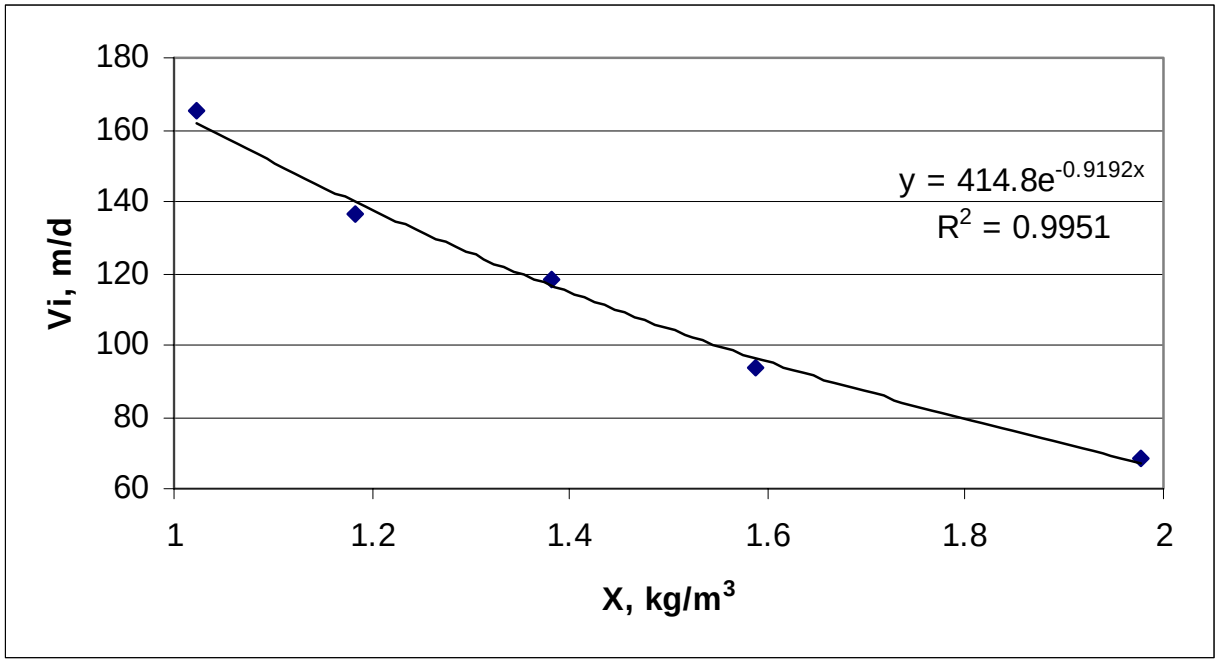

Figure E-1 Field Data and Fitted Exponential Equation for Zone Settling

Table E-2 Settling Velocity Calculations, HRT 45 min (Day 2)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	2376	63.36	2.376
800 sludge / 200 water	1892.5	84.96	1.8925
700 sludge / 300 water	1660	113.76	1.66
600 sludge / 400 water	1412.5	115.2	1.4125
500 sludge / 500 water	1180	172.8	1.18

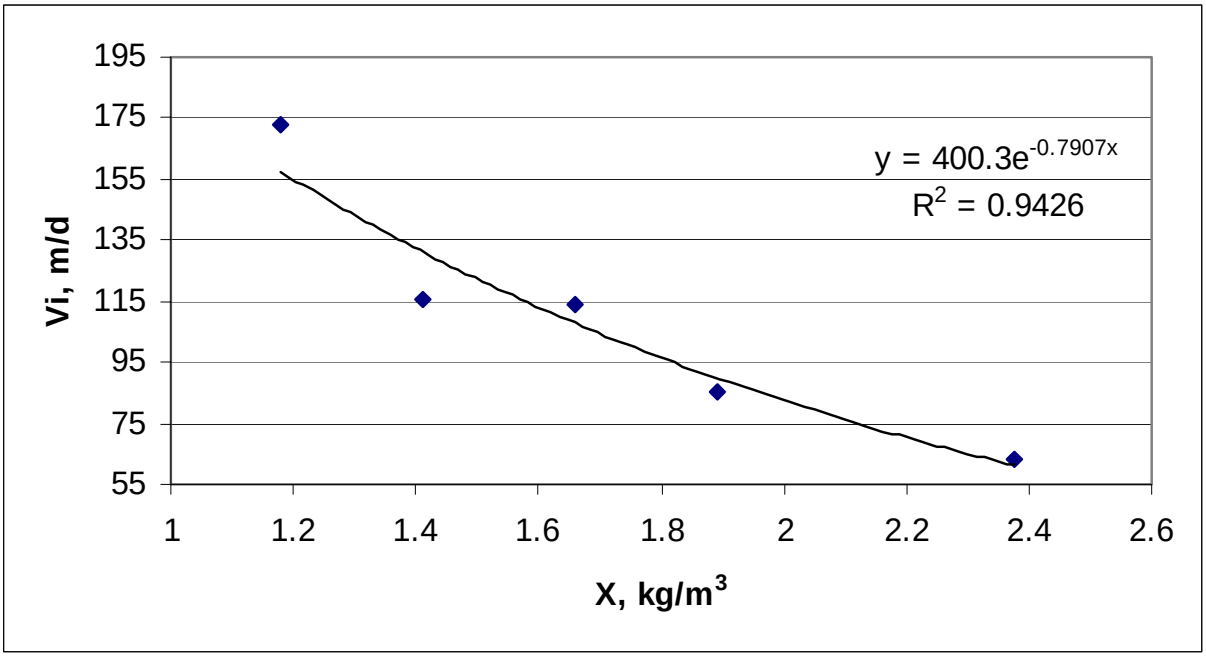

Figure E-2 Field Data and Fitted Exponential Equation for Zone Settling

Table E-3 Settling Velocity Calculations, HRT 45 min (Day 3)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	2470	53.28	2.47
800 sludge / 200 water	1972.5	64.8	1.9725
600 sludge / 400 water	1495	115.2	1.495
500 sludge / 500 water	1240	172.8	1.24
400 sludge / 400 water	992.5	172.8	0.9925

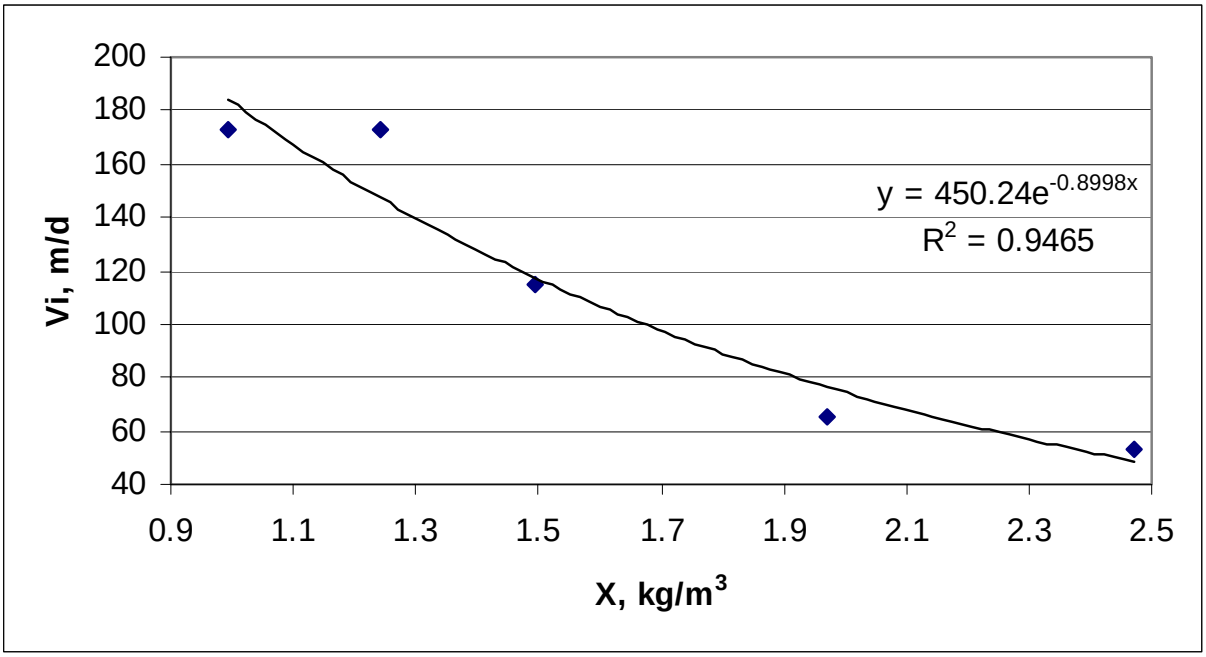

Figure E-3 Field Data and Fitted Exponential Equation for Zone Settling

Table E-4 Settling Velocity Calculations, HRT 45 min (Day 4)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	2836	43.92	2.836
800 sludge / 200 water	2250	51.12	2.25
700 sludge / 300 water	1972.5	74.28	1.9725
600 sludge / 400 water	1720	100.8	1.72
500 sludge / 500 water	1327.5	129.6	1.3275

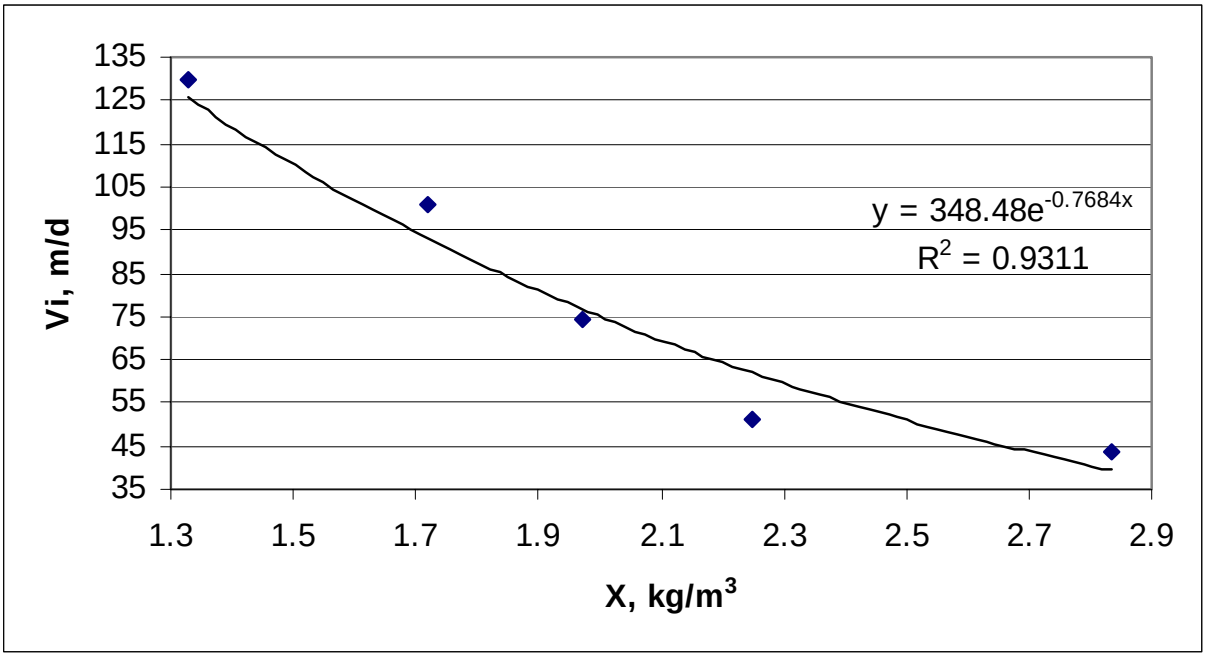

Figure E-4 Field Data and Fitted Exponential Equation for Zone Settling

Table E-5 Settling Velocity Calculations, HRT 45 min (Day 5)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	3014	48.96	3.014
800 sludge / 200 water	2420	60.96	2.42
700 sludge / 300 water	2130	81.6	2.13
600 sludge / 400 water	1802.5	108	1.8025
500 sludge / 500 water	1485	129.6	1.485

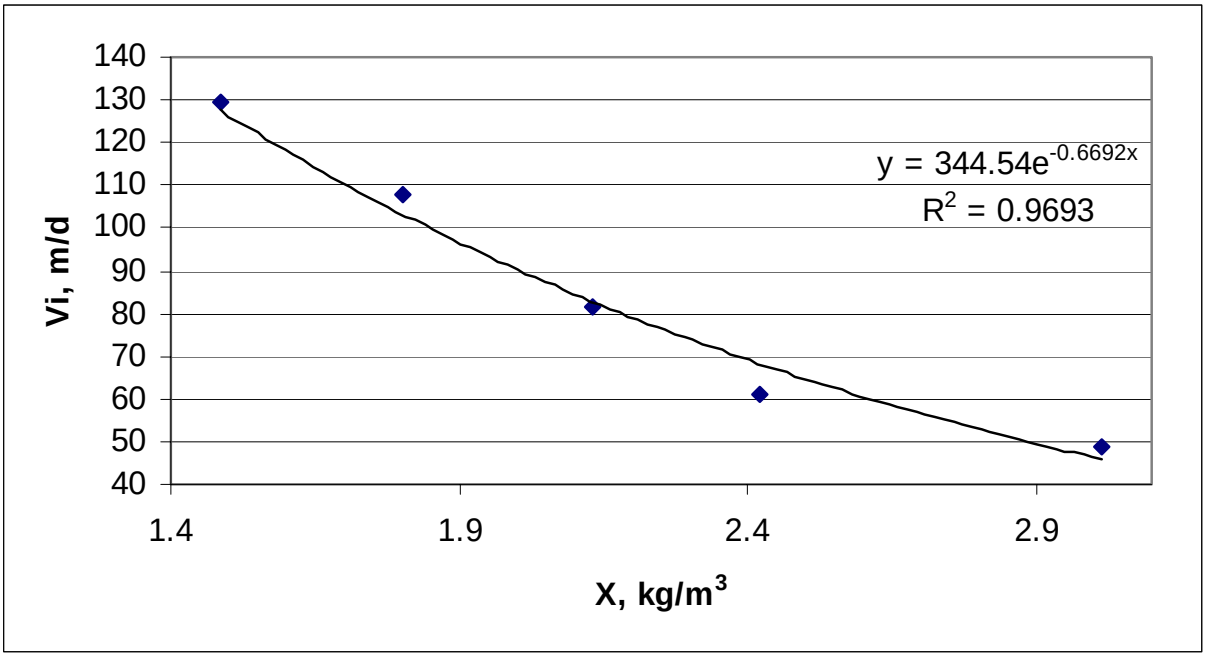

Figure E-5 Field Data and Fitted Exponential Equation for Zone Settling

Table E-6 Settling Velocity Calculations, HRT 45 min (Day 6)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	3242.5	31.68	3.2425
800 sludge / 200 water	2519.167	38.88	2.51916667
600 sludge / 400 water	1975	56.4	1.975
400 sludge / 600 water	1272.5	100.8	1.2725
300 sludge / 700 water	980	129.6	0.98

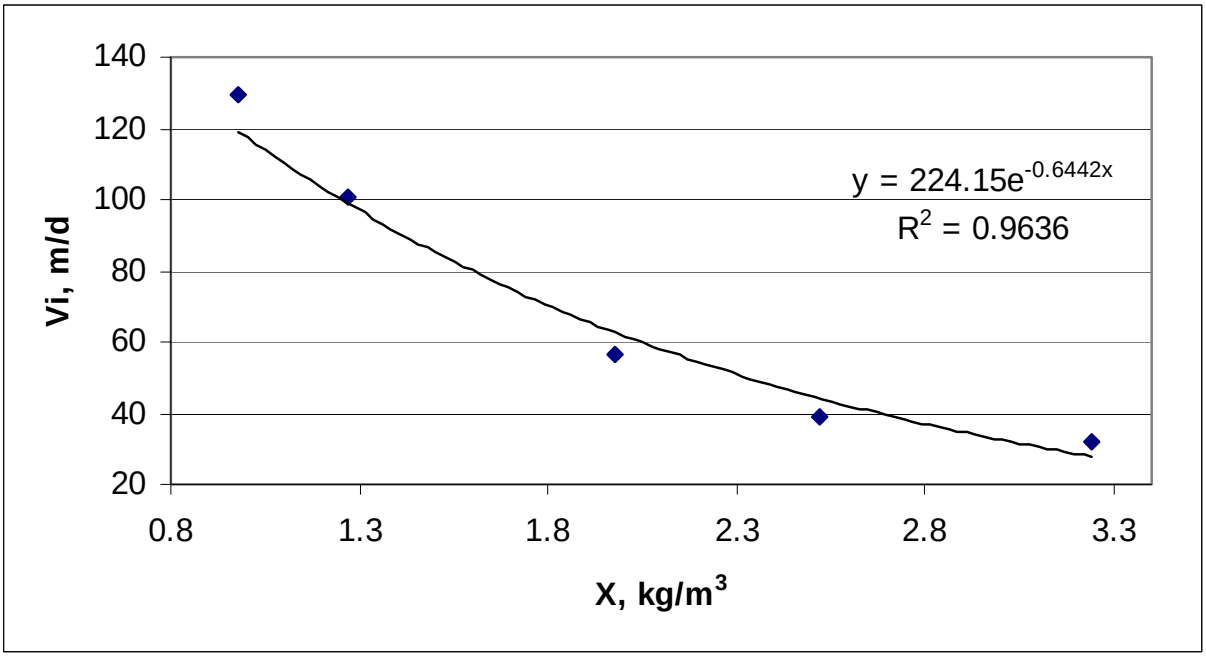

Figure E-6 Field Data and Fitted Exponential Equation for Zone Settling

Table E-7 Settling Velocity Calculations, HRT 45 min (Day 7)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	3348	51.84	3.348
800 sludge / 200 water	2595	54	2.595
600 sludge / 400 water	1997.5	89.28	1.9975
500 sludge / 500 water	1672.5	115.2	1.6725
400 sludge / 600 water	1330	158.4	1.33

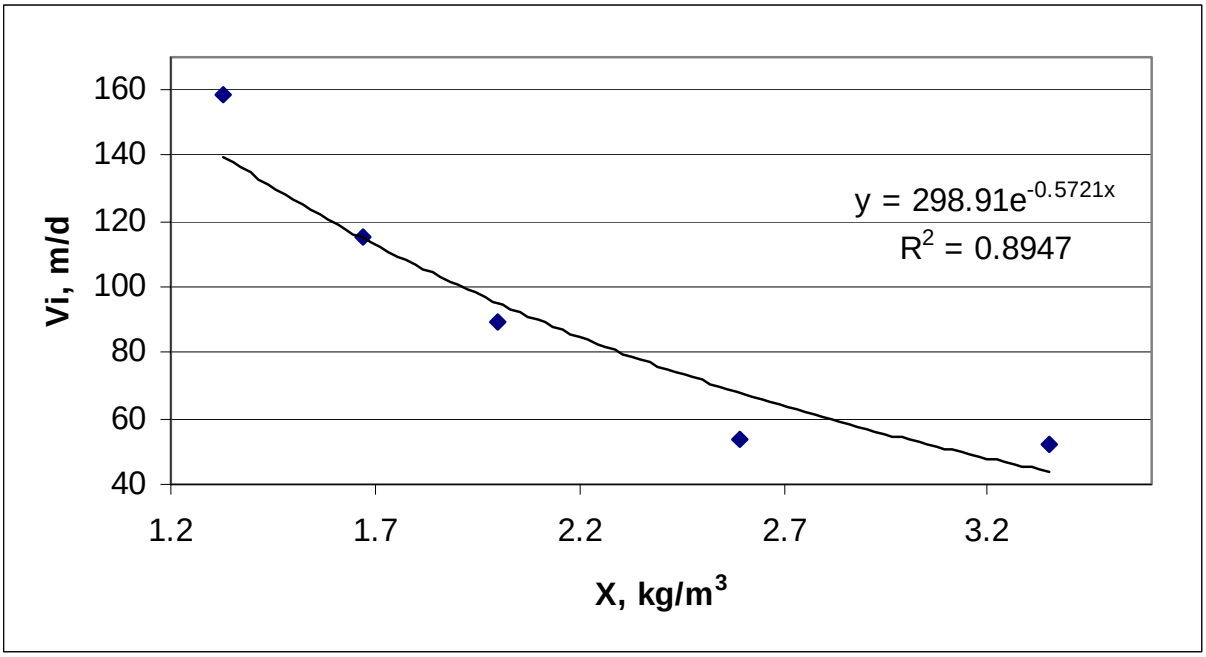

Figure E-7 Field Data and Fitted Exponential Equation for Zone Settling

Table E-8 Settling Velocity Calculations, HRT 45 min (Day 8)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	4645	38.16	4.645
800 sludge / 200 water	3705	44.64	3.705
600 sludge / 400 water	2702.5	54	2.7025
400 sludge / 600 water	1780	97.2	1.78
300 sludge / 700 water	1330	158.4	1.33

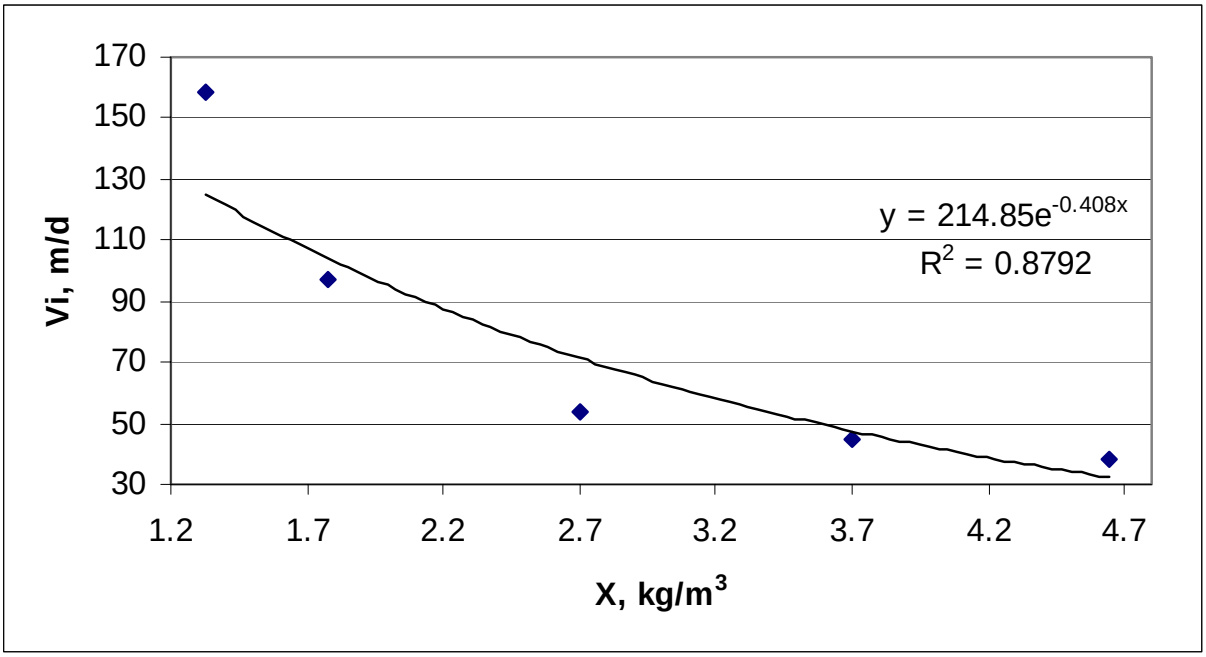

Figure E-8 Field Data and Fitted Exponential Equation for Zone Settling

Table E-9 Settling Velocity Calculations, HRT 30 min (Day 1)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	2022	30.72	2.022
800 sludge / 200 water	1622.5	42.48	1.6225
700 sludge / 300 water	1212.5	50.4	1.2125
600 sludge / 400 water	1012.5	61.2	1.0125
500 sludge / 500 water	805	86.4	0.805

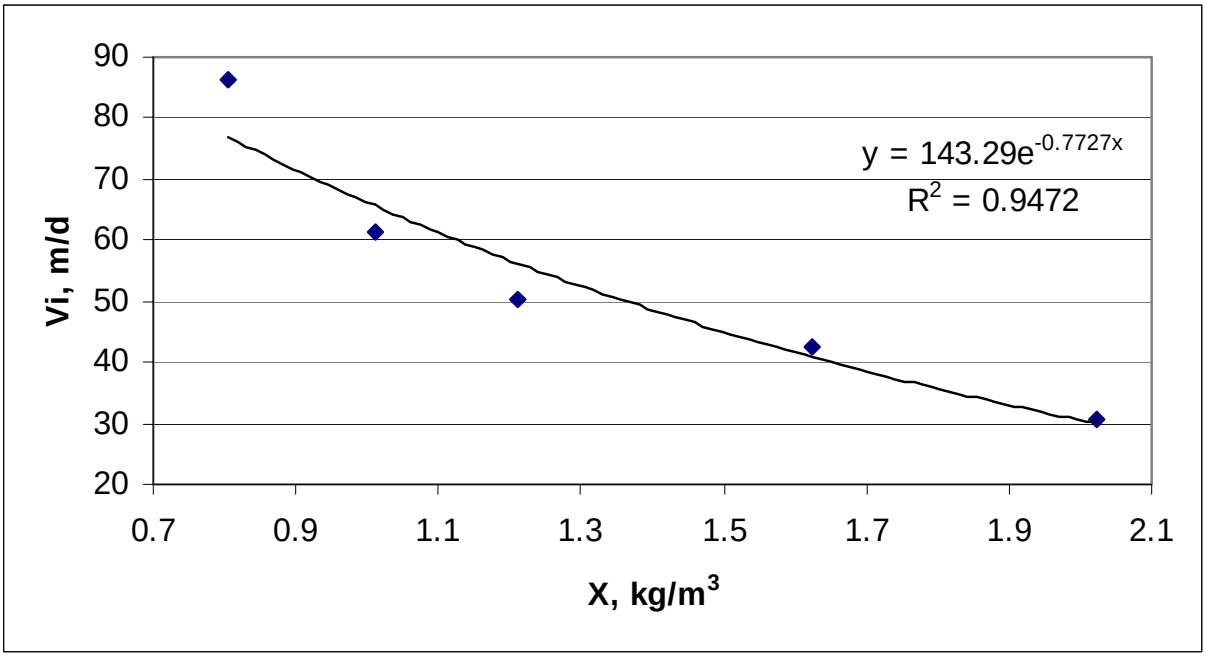

Figure E-9 Field Data and Fitted Exponential Equation for Zone Settling

Table E-10 Settling Velocity Calculations, HRT 30 min (Day 2)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	2600	42.48	2.6
800 sludge / 200 water	2075	57.6	2.075
600 sludge / 400 water	1562.5	81.36	1.5625
500 sludge / 500 water	1295	103.2	1.295
400 sludge / 600 water	1037.5	136.8	1.0375

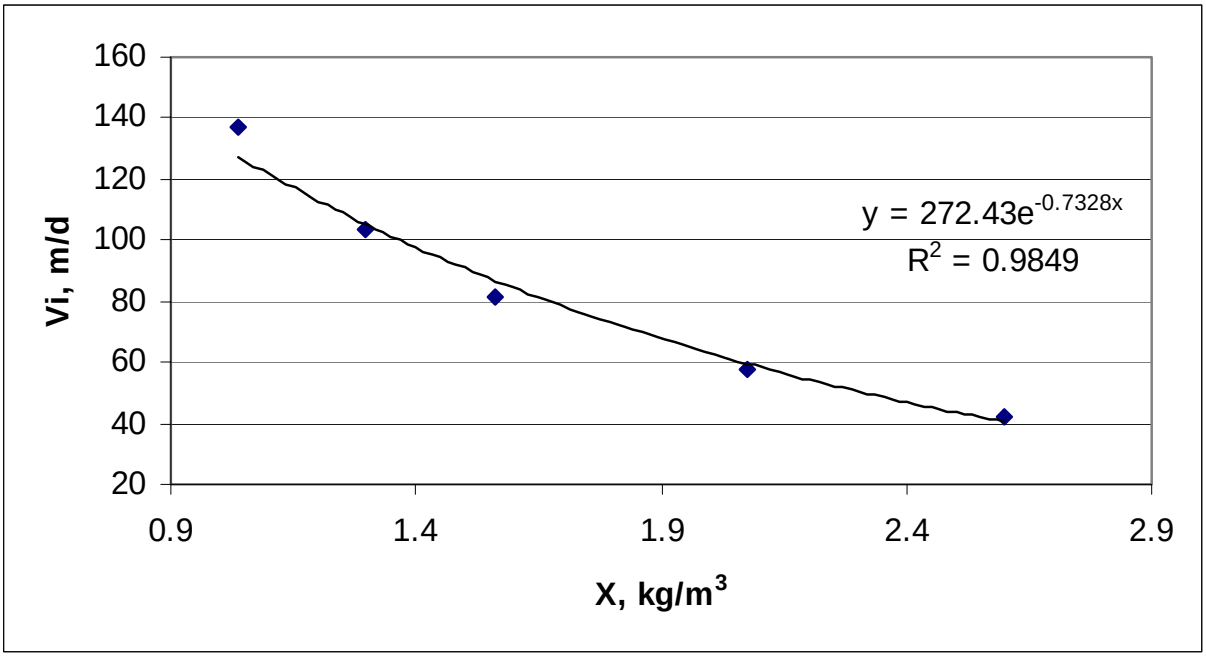

Figure E-10 Field Data and Fitted Exponential Equation for Zone Settling

Table E-11 Settling Velocity Calculations, HRT 30 min (Day 3)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	3093	35.28	3.093
800 sludge / 200 water	2470	43.2	2.47
600 sludge / 400 water	1855	67.68	1.855
500 sludge / 500 water	1540	100.8	1.54
400 sludge / 600 water	1240	115.2	1.24

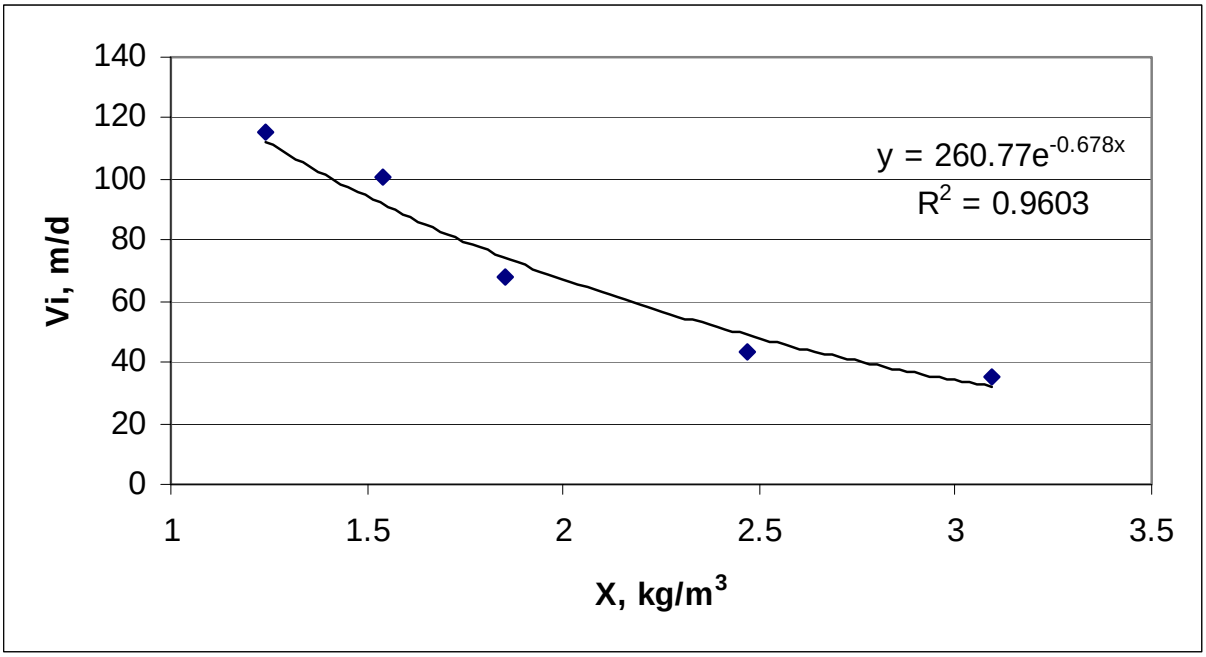

Figure E-11 Field Data and Fitted Exponential Equation for Zone Settling

Table E-12 Settling Velocity Calculations, HRT 30 min (Day 4)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	3153.5	45.24	3.1535
800 sludge / 200 water	2520	58.08	2.52
600 sludge / 400 water	1900	72	1.9
500 sludge / 500 water	1572.5	94.8	1.5725
400 sludge / 600 water	1255	115.2	1.255

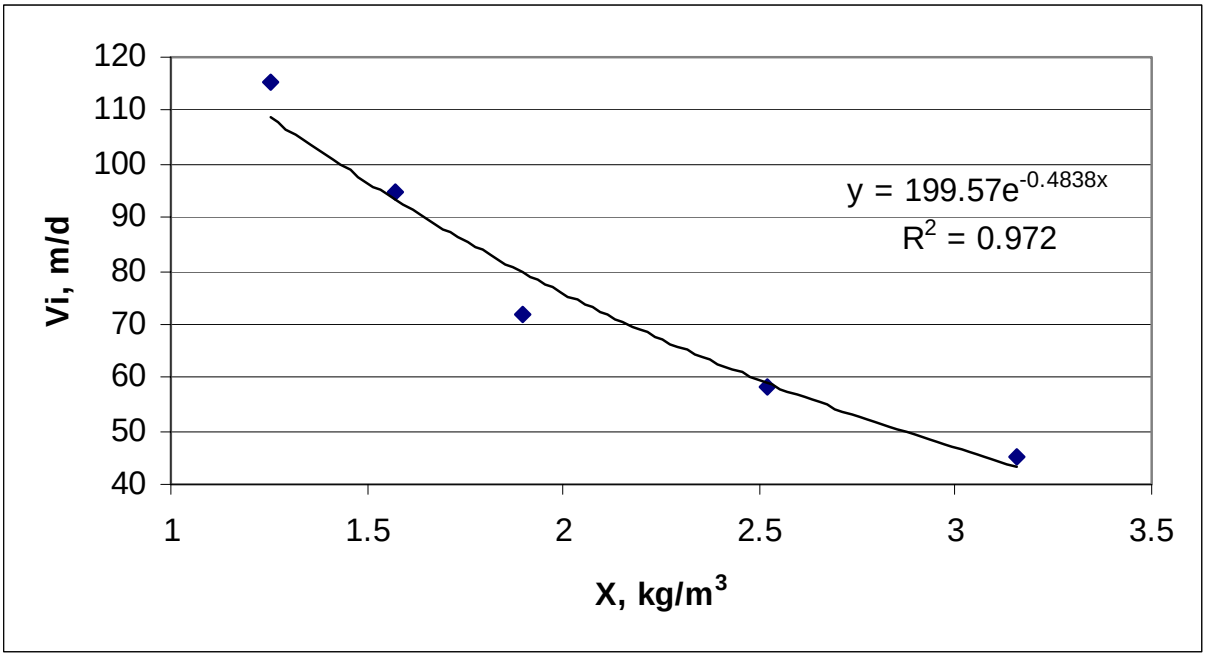

Figure E-12 Field Data and Fitted Exponential Equation for Zone Settling

Table E-13 Settling Velocity Calculations, HRT 30 min (Day 5)

Dilutions	Xi (mg/L)	Vi(m/d)	X (kg/m3)
MLSS	3515	33.12	3.515
800 sludge / 200 water	2810	43.2	2.81
600 sludge / 400 water	2107.5	57.6	2.1075
500 sludge / 500 water	1752.5	72	1.7525
400 sludge / 600 water	1405	99.36	1.405

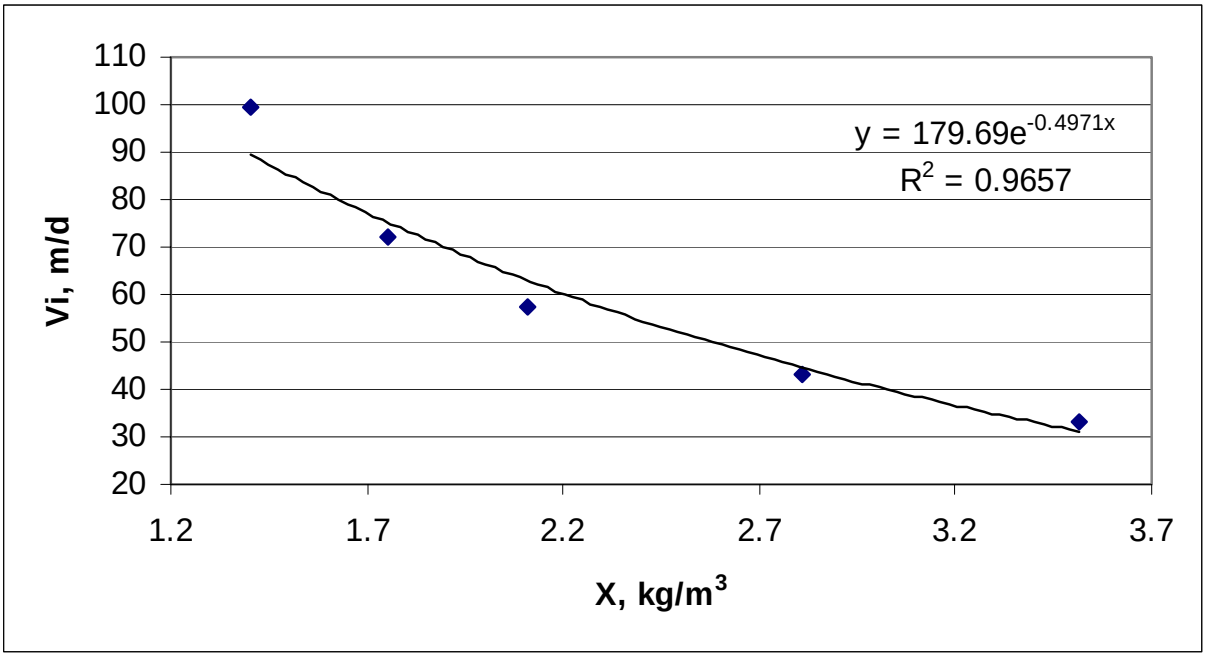

Figure E-13 Field Data and Fitted Exponential Equation for Zone Settling

VITA

Jose Angel Rojas was born in Maracaibo, Venezuela, on September 09, 1978. In 1995, he graduated from Antonio Rosmini High School in Maracaibo. Later on, he graduated from Universidad Santa Maria in March 2002, obtaining a degree of Bachelor of Sciences in Civil Engineering. In summer 2003, he started at the University of New Orleans, pursing a Master's of Science in Environmental Engineering.

During graduate school, he was a Graduate Research Assistant for two years at the Urban Waste Management and Research Center in the Department of Civil and Environmental Engineering working for Dr. Enrique J. La Motta. His academic emphasis is focused in the area of water and wastewater treatment processes in the environmental field.