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Title of Thesis — Titre de la thèse

THE INFLUENCE OF DYNAMIC LOADS AND PROCESS VARIABLES ON THE REMOVAL OF SUSPENDED SOLIDS FROM THE ACTIVATED SLUDGE SYSTEM.

University — Université

THE UNIVERSITY OF ALBERTA.

Degree for which thesis was presented — Grade pour lequel cette thèse fut présentée

DOCTOR OF PHILOSOPHY (Ph. D)

Year this degree conferred — Année d'obtention de ce grade

1984.

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THE INFLUENCE OF DYNAMIC LOADS AND PROCESS VARIABLES ON THE REMOVAL OF SUSPENDED SOLIDS FROM THE ACTIVATED SLUDGE SYSTEM

by

David T. Chapman

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE

OF DOCTOR OF PHILOSOPHY

IN

ENVIRONMENTAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING, 1984

THE UNIVERSITY OF ALBERTA

RELEASE FORM

NAME OF AUTHOR David T. Chapman

TITLE OF THESIS THE INFLUENCE OF DYNAMIC LOADS AND PROCESS VARIABLES ON THE REMOVAL OF SUSPENDED SOLIDS FROM THE ACTIVATED SLUDGE SYSTEM

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ABSTRACT

The escape of suspended solids in the effluent from the final settler of the activated sludge system accounts for the majority of the BOD_5 discharged from the system. At present, there is insufficient knowledge to enable the suspended solids concentration to be adequately predicted or controlled. To address this problem, research was conducted to characterize the time-varying response of the settler and to determine which of a number of design and operating variables significantly influence suspended solids removal.

Mixed liquor from an activated sludge plant treating municipal sewage was pumped at varying rates to a 2.4 m diameter settler. The activated sludge plant and test settler were configured so that air flow rate, underflow rate out of the settler, rake speed, feedwell depth, and settler depth could also be varied. The test settler was decoupled from the aeration tanks so that the settler feed flow rate was independent of underflow rate. Sensors interfaced to a minicomputer monitored performance.

Step changes in feed flow rate were applied to the settler with the overflow rate changing between 1.0 and 1.7 $m^3/m^2 \cdot hr$. The time constants for the responses following the step decreases averaged 26 min compared with a value of 17 min for the step increases. A number of responses following the flow increases displayed overshoot which was attributed to the discharge of floating solids. Because of the presence of overshoot, a second-order model was required to predict the settler response for increasing flows. A first-order

model was adequate to describe the response following flow decreases. These results indicate that hydraulic transients have the ability to degrade effluent quality and point to the need to avoid on/off control of large influent pumps.

A first-order model satisfactorily described the changes in effluent suspended solids concentration induced by either increases or decreases in the feed solids concentration in the flow to the settler. The time constant for the changes was approximately five hours. The gain ranged from 5 to 7 mg/L per g/L change in MLSS concentration.

Using regression techniques in conjunction with a two-level factorial design, the influence of a number of design and operating parameters on steady-state removal of suspended solids was investigated. Changes in MLSS concentration, sidewater depth and feed flow rate accounted for 78 percent of the observed variability in effluent suspended solids concentration. Sidewater depth and feed flow rate were interactive; the deterioration accompanying an increase in feed flow rate was less severe at a high sidewater depth than at a low one. Analysis of previously published data collected from settling columns of various heights provided supporting evidence for a depth/flow interaction. Over the ranges investigated, changes in air flow rate, settler underflow rate, rake speed, and feedwell depth had no significant influence on effluent quality. An increase in variability generally accompanied any increase in the mean level of effluent suspended solids concentration.

Based on the experimental results and the published literature, a number of recommendations were proposed. Hydraulic loading criteria should be based on the total flow into the settler rather than on the overflow rate. The total flow into the settler governs the velocity of he vertical roller in the settler and hence the level of turbulence. Accordingly, recycle rate should be minimized subject to the need to prevent thickening failure. Depth is an important design criteria with higher peak inflows to the plant requiring greater settler depths. The MLSS concentration determines the division of construction costs between the aeration tank and the final settler.

ACKNOWLEDGMENTS

The research which formed the subject matter for this thesis was carried out as a co-operative venture between Environment Canada and the University of Alberta. My thanks and appreciation go to the following:

Dr. Daniel W. Smith for program supervision,

Dr. Henryk Melcer and Dr. Bruce Jank for financial support,

Jim Matthews for his invaluable work in operating and maintaining the pilot plant equipment,

Dr. John MacGregor for his advice regarding experimental planning and statistical techniques.

Dr. Grant Fischer, Joe Stephenson, and Dr. Gustaf Olsson for their comments and suggestions regarding clarifier dynamics,

Gordon Speirs for carrying out modifications to the raw sewage screen,

Brian Monaghan and Mark Yendt for developing much of the software used during the research,

Eric Luxon for installing and calibrating on-line sensors,

Ron Gillespie for carrying out equipment modifications, and

Dianne Crabtree for typing the thesis.

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LIST OF SYMBOLS

Conventional Notation:

A = surface area of settler (m^2)

a = empirical constant used in zone settling velocity/concentration equation

a_1, b_1 = regression coefficients for first-order model

C = concentration of tracer

C_b = suspended solids concentration for point "b" on the settling curve (mg/L) (see Figure 2.3)

Ce = effluent suspended solids concentration (mg/L)

Cef = the final steady-state suspended solids concentration

Cei = the initial steady-state suspended solids concentration

Ce_k = effluent suspended solids concentration at kth time interval (mg/L)

Ce_{k+1} = effluent suspended solids concentration at k+1 time interval (mg/L)

$Ce(t)$ = the effluent suspended solids concentreation at time t

$Ce(s)$ = the Laplace transform of the function describing the effluent suspended solids concentration

C_i = suspended solids concentration in zone of uniform settling (mg/L)

C_{in} = suspended solids concentration in inflow to thickener (mg/L)

C_L = suspended solids concentration corresponding to the limiting solids flux (mg/L)

C_L' = dilute blanket concentration (mg/L)

C_0 = initial tracer concentration

C_{sp} = suspended solids concentration corresponding to the state point (mg/L)

C_u = concentration of suspended solids in the underflow from the settler (mg/L)

D = tank diameter (m)

d = axial dispersion index

dia.f = feedwell diameter (m)

FINC = fraction incomplete

f.d. = feedwell depth (m)

G_a = applied solids flux ($kg/m^2 \cdot d$)

G_b = the solids flux for point "b" on the settling curve ($kg/m^2 \cdot d$) (see Figure 2.6a)

G_i = total solids flux at any time t ($kg/m^2 \cdot d$)

G_L = limiting solids flux ($kg/m^2 \cdot d$)

G_{sp} = solids flux corresponding to the state point ($kg/m^2 \cdot d$)

G(s) = transfer function

H_i = the height of the solids/liquid interface in a column after t_i settling (cm)

H_o = the initial height of the solids/liquid interface in a batch settling column (cm)

H_u = the height of the solids/liquid interface corresponding to the desired underflow concentration C_u (cm)

h_B = blanket height above the bottom of the settler (m)

Kp = process gain

k, k^1 = empirical constant used in zone settling velocity/concentration equation

kpf, k'pf,

kn, Kepa = proportionality constants reflecting suspension clarification properties kpf, k^1pf, kn: (m/h); Kepa: (mg/L)/(m/h)

MLSS = the concentration of suspended solids in the mixed liquor (mg/L)

$MLSS_k$ = MLSS concentration at kth time interval (mg/L)

MLSS(s) = Laplace transform of function describing MLSS concentration

$MLSS_{st}$ = set-point MLSS (mg/L)

m^1 = empirical constant used in zone setting velocity/concentration relationships

m = slope of ln (C/C_0) vs t/T plot

N = the number of completely mixed tanks-in-series which describes the hydraulics of a system

Nd = number of data points

n = empirical constant used in zone settling velocity/concentration relationships

np_1 = number of parameters contained in model 1

np_2 = number of parameters contained in model 2

Qa = clarifier feed flow rate, Qi + Qr (m^3/d)

Qa(s) = Laplace transform of function describing feed flow rate

Qi = wastewater flow into plant (m^3/d)

Qr = rate at which sludge is recycled from settler to aeration tank (m^3/d)

Qu	=	settler underflow rate consisting of the sum of recycle and wasting rates (m^3/d)
RRS_1	=	residual sum of squares for model 1
RRS_2	=	residual sum of squares for model 2
RTD	=	residence time distribution
r_{xy}	=	regression coefficient
S	=	settler floor slope
SLR	=	solids loading rate ($kg/m^2 \cdot h$)
SOR	=	surface overflow rate ($m^3/m^2 \cdot h$)
SQIA	=	the Approximate Sludge Quality Index
SRT	=	solids retention time or sludge age
SSV	=	30-minute settled volume (mL/L)
$SSVI_{20}$	=	stirred specific volume index at 20°C (mL/g)
$SSVI_{Te}$	=	stirred specific volume index at T°C (mL/g)
SVI	=	sludge volume index (mL/g)
SWD	=	sidewater depth (m)
s	=	Laplace transform variable
T	=	theoretical detention time, V/Qi (min)
Te	=	temperature (°C)
Ts	=	sampling interval for on-line sensor
$\bar{t}'$	=	centroid of RTD curve; $\Sigma t \cdot C / \Sigma C$ (min)
t_a	=	time to half area (see Figure 2.7)
t_b	=	base time (min) (see Figure 2.7)
t_c	=	base time at $C/2C_0$ (see Figure 2.7)
t	=	time elapsed since beginning of test (min)
t_1	=	for a residence time distribution curve the time to the first occurence of tracer in the effluent (min)

t_p = for a Residence Time Distribution curve, the time to peak value of tracer concentration (min)

t_u = time to reach desired underflow concentration C_u (min)

u = the slope of the Yoshioka line (see Figure 2.2)

V = volume of reactor (m^3)

Vb = "active" or completely-mixed volume (m^3)

V_s/V = ratio of sludge volume to settler volume

$(V_s/V)_{cr}$ = critical ratio of sludge volume to settler volume

v_i = ZSV = zone settling velocity (m/h)

v_L = the zone settling velocity at limiting solids flux (m/h)

v_o = Stokes settling velocity (m/h)

V_r = rake speed (revs/h or m/min)

WLR = weir loading ($m^3/d \cdot m$)

Y = statistic used to judge if addition of parameters to a model is justified

ZSV = v_i = zone settling velocity (m/h)

Greek Notation:

α = Qr/Qi = recycle proportion

β = level of significance

ΔH = settler depth required for sludge storage (m)

ΔT = period for which is overloaded (h)

ζ = damping ratio for a second-order system

θ = normalized time = t/T

σ_θ^2 = the variance of the non-dimensional RTD curve (C/C_0 vs t/T)

σ_t^2	=	the variance of the RTD curve (C vs t)
τ_D	=	lag time (min)
τ	=	process time constant for a first-order system
ω_n	=	natural frequency of oscillations (second-order system)

CHAPTER 1

INTRODUCTION

The activated sludge system consists of an aeration tank followed by a settling basin. For an effluent of acceptable quality to be produced, the settler must remove solids from the process liquid prior to discharge. Secondly, the settler thickens bio-solids. The thickened bio-solids are either pumped back to the aeration basin as return activated sludge or subsequently treated and disposed of as waste activated sludge. During periods of peak flow to the treatment plant, biological solids are shifted from the aeration tank to the settler. There must be sufficient volume in the settling basin to store the solids which are subsequently returned during normal flow conditions. Therefore, the secondary settler in the activated sludge system must be designed and operated to carry out three functions - clarification, thickening and storage.

Sizing of secondary settlers generally involves the use of empirical loading rates developed from past experience. The recycle rate from the settler is most commonly set based on the sludge volume index (SVI) - a batch settling measurement - or in proportion to the rate of inflow into the plant. Recently, a theoretical basis for design and operation has been developed (Dick and Young, 1970). The approach, known as solids flux theory, enables thickening performance to be evaluated using the results from batch settling tests.

Although a consistent approach has been developed to describe the thickening function, clarification is poorly understood.

There is a lack of consensus regarding the relative importance of design and operating variables on clarifier efficiency. There is no widely accepted method for predicting the suspended solids concentration in the effluent from the settler. Consequently, both cost optimization and process performance of the activated sludge system have been limited by the ability to predict or control clarification (Adams and Asano, 1978).

Methods for designing and strategies for controlling secondary settlers are frequently based on the assumption that the process operates at steady-state. In reality, the system is subject to a variety of time-varying disturbances, the most important of which are variations in the rate of influent flow. There is very little information currently available as to the effects of time-varying loadings on operational efficiency.

A research program was undertaken to identify which of a number of design and operating variables significantly influenced the efficiency of clarification. Secondly, an evaluation was carried out of the response of the clarifier to time-varying flows and operating conditions. This thesis is a record of the objectives, methods and results of the research.

CHAPTER 2

LITERATURE REVIEW

An enormous number of papers deal with topics related to clarifier design and operation. This literature review attempts to synthesize the findings of the most important published research in order to identify specific shortcomings in current knowledge. Accordingly, the first two sections of the review are introductory, serving to describe settler operation, list major equipment options and identify methods for characterizing mixed liquor settleability. The following two sections critically examine current design procedures and operating strategies for achieving good settler performance. Techniques for measuring hydraulic efficiency and the effects of density currents and time-varying loads on settler operation are reviewed in the section on hydraulics. Lastly, a description is given of the basic mechanisms involved in bioflocculation, existing clarification models and the influence of design and operating variables on clarification.

2.1 Process Description

Secondary settlers are flow-through units which are generally circular or rectangular. In Germany, however, square units with vertical flow, developed in the 1880's and known as "Dortmund tanks", are still in use (IEC, 1975).

There are five major components to every secondary basin: inlet, outlet, scum collector, sludge collector and sludge return.

The mixed liquor suspension is distributed to the basin through the inlet with the clarified effluent collected by weirs. Floating material is removed by a skimming device and accumulated solids are withdrawn by the sludge collector and return pumps. Table 2.1 outlines the variety of equipment options available for each of the five components.

The majority of sedimentation tanks used in the activated sludge process are circular units with a centre feedwell, peripheral wall-supported weirs and a scraper-type sludge collector. This typical configuration is shown in Figure 2.1. The popularity of circular tanks, frequently referred to as clarifiers, is related to the ease of sludge collection. Rectangular tanks employ either chain and flight or bridge collectors. Chain and flight mechanisms with submerged bearings are difficult and expensive to maintain (Kalbskopf, 1970; McKinney). Travelling bridges, because of travel time, allow uneven accumulation of solids (McKinney). Sestak (1980) found the suspended solids content of the settled sludge to be higher at the ends of the tank requiring that the rate of travel of the bridge be adjustable. The rotating sludge collector used in circular tanks largely overcomes these problems.

Claims have been made that peripheral feed tanks are more efficient than centre feed tanks (Boyle, 1976). To permit scum removal, peripheral feed basins are constrained to use peripheral weirs. The problem of inlet distribution in peripheral feed tanks, especially with highly variable flows, has been raised as a possible disadvantage (McKinney).

ABLE 2.1: Alternative Secondary Settler Configurations
(Adapted from Joint Committee WPCF/ASCE, 1977; Karassik et al., 1976; Metcalf and Eddy, 1979; McKinney; Perkins and Wood, 1979; Task Committee, ASCE, 1979; U.S. EPA, 1975)

---

ank Shapes:

- Circular - Radial Flow
- Rectangular - Horizontal Flow
- Square - Up-Flow "Dortmund" Type

nlet/Outlet:

- Centre-Feed/Peripheral Weir
- Centre-Feed/Radial Wiers
- Peripheral-Feed/Centre Weir
- Peripheral-Feed/Peripheral Weir

utlet:

- Support - Peripheral Support - Single Weir
  - Cantilever Support - Double Weir
- Notches - "Vee" - 60° or 90°
  - Rectangular

ludge Collection:

- Collector:
  - Scraper Type - Chain and Flight
    - Chain
    - Blade - echelon
      - non-echelon
  - Suction Type
- Collector Support:
  - Centre Pier Supported - Bridge is Fixed
  - Bridge Supported - Travelling Bridge
    - Rotating Bridge
    - Centre Gear or Rim Driven
    - ½, Full, 3-Arm or 4-Arm
- Sludge Hopper - Circular Tanks - Concentric
  - Offset
  - Rectangular Tanks - Hopper at Inlet
    - Central Hopper
  - Hopper Outside of Tank: (For Use With Suction Removal on Rectangular Tanks)
- Return Sludge Pumps
  - Centrifugal
  - Air Lift
  - Plunger
  - Progressing Cavity
  - Torque Flow
  - Archimedian Screw Pump

Skimming Devices:

- "Dipping Weir" - Slotted Pipe or Weir
- "Sloping Beach"

Page 6 was removed due to copyright restrictions. The page contained Figure 2.1, obtained from a paper by McKinney, showing a typical circular sedimentation tank.

Scrapers are most commonly used for sludge collection. McKinney has commented on the difficulty of adjusting the head differential on individual nozzles on suction-type collectors. The danger then exists of pulling dilute concentrations of sludge through a hole in the sludge blanket. Hydraulic suction collectors are best used for sludges which denitrify in the clarifier and must, therefore, be removed quickly (McKinney).

Sedimentation tanks are expected to produce an effluent with an average suspended solids concentration in the 20 to 30 mg/L range. However, data collected from a number of activated sludge plants in the U.S. and summarized in Table 2.2, reveal that, for a majority of the plants, the mean effluent suspended solids concentration exceeded 30 mg/L. The facilities were assumed to be operating below design capacity (Committee on Water Pollution Management, ASCE, 1980). Further, from Table 2.2, it appears as if small plants, with a capacity of less than 3800 m^3/day, achieve better solids removal.

With reference to the grand means of Table 2.2, estimates of the contribution of effluent suspended solids to effluent BOD_5 were made using the methods in Appendix A. Roughly, 50 to 90 percent (depending on what assumptions are made with regards to hydraulic and solids retention time) of the BOD_5 in the effluent can be attributed to suspended solids released over the weir. Dick (1970a) has placed the figure at 60 percent. Therefore, for the conventional activated sludge process, the suspended solids which the settler fails to remove constitute the major fraction of BOD_5 discharged by the system.

TABLE 2.2: Conventional Activated Sludge Performance as Measured at Selected Treatment Plants.
(Committee on Water Pollution Management, ASCE, 1980)

Plant Size (m^3/day)	Number of Plants	Effluent Mean BOD_5 (mg/L)	Effluent Mean SS (mg/L)
0 to 3,800	3	13.4	18.1
3,800 to 37,900	8	41.0	42.8
37,900 to 378,500	1	22.0	43.0
	GRAND MEAN	32.5	36.6

The solids concentration in the underflow from secondary clarifiers will be less than 20,000 mg/L or two percent (Joint Committee WPCF/ASCE, 1977). A concentration of 10,000 mg/L is regarded as standard for most clarifiers (McKinney). As with the effluent quality, underflow concentration is greatly influenced by the settling rate of the suspension. Bulking sludges can result in an underflow concentration of less than 2000 mg/L (Pipes, 1979).

Besides the conventional equipment options previously listed, a number of modifications are available to upgrade clarifier performance. Tube settlers and parallel plates (known as "lamellas") improve effluent quality when added to settlers. However, as pointed out by Mendis and Benedek (1980), these shallow sedimentation devices do nothing to improve thickening performance. An air grid or alternative washing device will be required whenever plates or tubes are used (U.S. EPA, 1975). Iron or aluminum salts can improve clarifier performance without harming the microbial population in the aeration tank (IEC, 1975). Another modification to the conventional clarifier is the solids contact unit. Mechanical drive paddles are incorporated within the tank to improve flocculation. Wedge wire settlers have found application in England (U.S. EPA, 1975). A grid constructed of wire which is triangular in section is placed within the tank, between the inlet and outlet.

As can be seen from the preceding process description, a large number of options and modifications are available to the designer concerned with secondary sedimentation. However, as implied in the discussion on effluent and underflow concentrations, efficiency in removing suspended solids is highly dependent on the settling rate of the suspension, as well as the configuration of the tank itself. The following section will, therefore, outline the parameters and tests used to categorize the rate of settling of mixed liquor suspensions.

2.2 <u>Measurements of Settling</u>

There are three measurements of mixed liquor settleability in common use: settled sludge volume (SSV), zone settling velocity (ZSV) and sludge volume index (SVI). To determine these measurements, a 1000 mL graduated cylinder is filled with mixed liquor and the position of the solid-liquid interface is monitored with time (Standard Methods, 1975). The volume occupied by the solids below the interface is referred to as the settled sludge volume. The slope of the straight-line portion of the interface height versus time curve defines the zone settling velocity (ZSV). The sludge volume index (SVI) or volume per gram of sludge following 30 minutes of settling is obtained from:

$$\text{SVI} = \frac{\text{30 min settled volume (mL/L)} \times 1000 \text{ (mg/g)}}{\text{MLSS (mg/L)}} \tag{1}$$

where: SVI = sludge volume index (mL/g)

MLSS = concentration of suspended solids in the mixed liquor (mg/L)

Owing to the simplicity of the test, the SVI parameter has wide acceptance in design and operation. Unfortunately, as pointed out by Dick and Vesilind (1969), SVI is highly dependent on the concentration of solids in the mixed liquor. As well, SVI does not relate in a consistent manner to either the zone settling velocity or rheological properties such as sludge yield strength or plastic viscosity. Test results vary depending on the cylinder diameter and height, the temperature, and the presence or absence of stirring. Dick and Vesilind (1969) conclude that SVI is best used for in-plant monitoring of settleability and is a poor test for use in research or for plant-to-plant comparisons.

A number of modifications have been proposed to overcome the shortcomings of the standard one-litre batch settling test. Vesilind (1975) suggests the following guidelines for settling tests:

- a cylinder diameter as large as possible with 20 cm a minimum,
- cylinder height equal to the actual tank depth being studied,
- cylinder filled from the bottom, and
- slow stirring of cylinder contents.

The last item, slow stirring, is important as it enhances agglomeration. Without stirring, turbulence is quickly dampened in small diameter cylinders resulting in poor solids agglomeration. Stirring also minimizes chanelling - the tendency of trapped water to flow along the sides of the cylinder - and bridging or arching of solids across the walls of the cylinder. Henry and Salenieks (1980), comparing stirred and unstirred settling tests, found that for moderately bulking sludges, much higher settling velocities were observed for the stirred test. For sludges with extreme bulking, both tests yielded similar settling rates.

White (1975a, 1975b) examined the use of a stirred test to predict maximum loading rates in clarifiers. The settling apparatus consisted of a 10 cm diameter perspex tube, 50 cm in height, which incorporated a 1 rpm stirrer. To avoid concentration effects, the test was carried out at one concentration, 3500 mg/L. Using the resulting values, termed stirred specific volume index (SSVI), White was able to predict solids loading rates which were within 20 percent of observed full-scale rates. The classifications were established based on the SSVI test results and are shown in Table 2.3.

The effect of temperature on the SSVI values was estimated to be (White, 1980):

$$SSVI_{20} = SSVI_{Te} \times 1.04^{(Te-20)} \text{ for Te: 10 - 30°C} \qquad (2)$$

where: $SSVI_{20}$ = stirred specific volume index at 20°C

$SSVI_{Te}$ = stirred specific volume index at Te

Te = temperature (°C)

Reed and Murphy (1969) also studied the effect of temperature on settling rate. Their results indicated that the effect of temperature on zone settling decreases as the concentration of the

TABLE 2.3: Settling Rate Classification Based on SSVI Test (Adapted from White, 1980)

Class	$SSVI_{3.5}$ (mL/g)	ZSV (m/h)
Good Settling	40 - 50	4 - 5
Average Settling	80	1
Poor Settling	120	0.5
Test Results Unreliable*	250	

* Depth of test cylinder inadequate to model full-scale tank.

suspension increases. This phenomena can be explained in terms of the forces which resist the subsidence of particles. For a dilute suspension, fluid drag forces predominate (Shin and Dick, 1975). Therefore, temperature changes affect fluid viscosity and hence the settling rate of the suspension. At a high solids concentration, inter-particle forces predominate (Shin and Dick, 1975). These forces are not influenced by temperature changes and consequently settling rate does not change markedly with temperature changes.

Fitch and Kos (1976) observed that SVI varies with suspension concentration. They estimated that the transition from the dilute to intermediate domain occurs at a settled sludge volume of approximately 300 mL/L and that the variation of SVI with concentration in the intermediate domain could be approximated by a straight line with an ordinate intercept of =600 mL/L. Based on these observations, a new settling parameter, the <u>approximate sludge quality index</u> (SQIA), was presented:

(a) for SSV $\leq$300 mL/L (dilute domain);

$$SQIA = SSV/MLSS \tag{3}$$

where: SQIA = the approximate sludge quality index (mL/g)

SSV = 30-min settled sludge volume (mL/L)

(b) for $300 \text{ mL/L} < SSV < 800 \text{ mL/L}$ (intermediate domain);

$$SQIA = \frac{200 + SSV/3}{MLSS} \qquad (4)$$

Pipes (1977), in commenting on the index, disputed much of the evidence upon which SQIA was based. He maintained that a linear relationship in the intermediate domain was, at best, an approximation and that the intercept of the ordinate was unlikely to be -600 mL/L for all sludges. Further, he stated that the dilute/intermediate transition could be anywhere from 200 to 700 mL/L.

A 2-L beaker (19 cm high × 13 cm outside diameter) has been proposed as an alternative to the 1000 mL cylinder (Nalge). However, experiments carried out at the Wastewater Technology Centre in 1980 revealed that no major advantage was gained in carrying out SVI determinations in a 2-L vessel as compared to the standard 1-L cylinder (Chapman, 1980).

Based upon the preceding discussion, the SSVI test, as proposed by White, appears to be the best of the available settling measurements for research and plant-to-plant comparisons. When using the test, temperature effects must be considered and it should not be used to study a very heavily bulking sludge as depth considerations become important. The standard SVI measurement, because of its simplicity, remains useful for day-to-day monitoring of plant settleability.

Measurements of sludge settling rate, particularly SVI values, have been used to monitor clarifier operation and adjust the recycle flow rate. As will be shown in the following section, batch tests using small diameter cylinders are also finding application in clarifier design.

2.3 Design

There are two approaches to the design of secondary clarifiers. The most common procedure relies on empirical loading rates developed from research and past experience. More recently, a design approach has been developed based on what is known as "solids flux theory".

Whatever the basis, a design procedure must account for up to four zones in the settler. The top of the clarifier contains clear supernatant. In this clarification zone, particles in dilute suspension undergo flocculant (or Class II) settling. The rate of settling is modified by particle growth which is a function of the size range and concentration of the suspension, velocity gradients within the zone, the viscosity of the liquid, the depth of the zone and the overflow rate (Metcalf and Eddy, 1979). For a given surface area of tank, depth (or retention time) becomes important because it governs the time available for particle collisions. In contrast, for Class I settling - discrete particles which do not coalesce during sedimentation - Camp (1953) has demonstrated that the depth of a settler does not influence its efficiency in removing suspended solids.

The transition from clarification to thickening is distinguished by a solids-liquid interface. Below the interface, particles in a suspension of intermediate concentration settle at a uniform rate. As will be discussed later in this section, the rate of zone or Class III settling is a function of the suspension concentration. Below the zone of constant settling, separated by a transition zone, the solids undergo compression (Class IV settling). Particles develop a structure which exhibits compressive strength (Fitch, 1979). No further settling occurs in the compression zone unless additional pressure is exerted (Fitch, 1979).

2.3.1 Empirical Loading Rates. Empirical design criteria, such as those identified in Table 2.4, key in on four parameters: surface overflow rate (SOR), solids loading rate (SLR), tank depth, and weir loading rate (WLR). The surface overflow rate is the plant inflow rate per unit of tank surface area. Based on theoretical arguments, Camp (1953) demonstrated that the surface loading rate for a tank handling discrete non-flocculant particles establishes the settling velocity (v_0) of the slowest particle which is completely removed. For tanks with horizontal flow there will be partial removal for particles with rates less than v_0 while, for vertical flow tanks, no particles with velocities less than v_0 will be removed. Therefore, surface overflow rate is a clarification parameter.

Selection of an appropriate solids load rate - the mass of solids applied per unit area - should result in a tank area which prevents thickening failure. However, as has been discussed by

TABLE 2.4: Design Parameters for Circular Secondary Sedimentation Tanks for the Conventional Activated Sludge Process
(Adapted from: Joint Committee WPCF/ASCE, 1977; Fair et al., 1968; Great Lakes, 1971; Kalbskopf, 1970; Loughton, 1980; Metcalf and Eddy, 1979; McKinney; Perkins and Wood, 1979; Task Committee ASCE, 1979; U.S. EPA, 1975)

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1) Maximum Diameter (D)

a) $D \leq 30$ m (Perkins and Wood, 1979)
b) $D \leq 60$ m (Joint Committee WPCF/ASCE, 1977)
c) $D \leq 61$ m (Task Committee ASCE, 1979)
d) $D \leq 10 \times$ SWD (Metcalf and Eddy, 1979) (SWD = side water depth)

---

2) Hydraulic Loading (or, surface overflow rate, SOR) (SOR = Qi/A)

a) SOR ≤ 1.5 $m^3/m^2/h$ (Perkins and Wood, 1979)
b) Horizontal Flow: SOR ≤ 0.5 to 1.5 $m^3/m^2 \cdot h$ (Kalbskopf, 1970)
Vertical Flow: SOR ≤ 2.5 1.5 $m^3/m^2 \cdot h$ (Kalbskopf, 1970)
c) Average: SOR ≤ 0.7 to 1.4 $m^3/m^2 \cdot h$ (U.S. EPA, 1975)
Peak: SOR ≤ 1.7 to 2.0 $m^3/m^2 \cdot h$ (U.S. EPA, 1975)
d) Peak: $(Qi + Qr)/A \leq 50$ $m^3/m^2 \cdot h$ (Laughton, 1980)

---

3) Solids Loading Rate (SLR) (SLR = Qi × MLSS/A)

a) SLR 6.3 to 6.8 $kg/m^2 \cdot h$ (Joint Committee WPCF/ASCE, 1977)
b) For effluent SS concentration <30 mg/L (Kalbskopf, 1970)
Horizontal Flow:

SVI (mL/g)	SLR Maximum ($kg/m^2/h$)
100	3.5
200	1.1 - 1.3
300	0.8 - 1.1

Vertical Flow: SLR ≤ 7.5 $kg/m^2 \cdot h$ (Kalbskopf, 1970)
c) Average: SLR ≤ 4.1 to 6.1 $kg/m^2 \cdot h$ (U.S. EPA, 1975)
Peak: SLR ≤ 10.2 $kg/m^2 \cdot h$ (U.S. EPA, 1975)

TABLE 2.4 (Cont'd): Design Parameters for Circular Secondary Sedimentation Tanks for the Conventional Activated Sludge Process
(Adapted from: Joint Committee WPCF/ASCE, 1977; Fair *et al*, 1968; Great Lakes, 1971; Kalbskopf, 1970; Loughton, 1980; Metcalf and Eddy, 1979; McKinney; Perkins and Wood, 1979; Task Committee ASCE, 1979; U.S. EPA, 1975)

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4) Theoretical Detention Time (T) or Depth (SWD)

(Detention Time = V/Qi)

a) T $\geq$6 h (Perkins and Wood, 1979)

b) T $\geq$2 h (Task Committee ASCE, 1979)

c) T $\geq$1.5 to 4 (Kalbskopf, 1970)

d) For tank diameter $\leq$30 m   d $\geq$3.3 m (Joint Committee WPCF/ASCE, 1977)

For tank diameter >30 m   d $\geq$4.6 m (Joint Committee WPCF/ASCE, 1977)

e) SWD $\geq$3.0 m (Task Committee ASCE, 1979)

f) SWD $\geq$3.7 to 4.6 m (U.S. EPA, 1975)

g) For weirs at periphery   SWD $\geq$3.7 m (Metcalf and Eddy, 1979)

For weirs otherwise located SWD $\geq$3.1 m (Metcalf and Eddy, 1979)

---

5) Floor Slopes (S)

a) S $\geq$1:2.7 (20°) (Perkins and Wood, 1977)

b) S $\geq$1:12 (4.8°) (Joint Committee WPCF/ASCE, 1977)

c) S $\geq$1:200 to 1:12 (0.3° to 4.8°) (Task Committee ASCE [illegible])

d) S $\geq$1:12.5 to [illegible] (4.6° to 7.[illegible]) (Fair et al., 1968)

---

6) Rake Speed (Vr)

a) Vr $\leq$0.6 to 1.2 m/min (Joint Committee W[illegible])

b) Vr $\leq$2 to 4 revs/h (Task Committ[illegible])

[illegible] Vr [illegible] m/h [illegible]

[illegible]

TABLE 2.4 (Cont'd): Design Parameters for Circular Secondary Sedimentation Tanks for the Conventional Activated Sludge Process
(Adapted from: Joint Committee WPCF/ASCE, 1977; Fair et al., 1968; Great Lakes, 1971; Kalbskopf, 1970; Loughton, 1980; Metcalf and Eddy, 1979; McKinney; Perkins and Wood, 1979; Task Committee ASCE, 1979; U.S. EPA, 1975)

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7) Feed Well (dia. f = feedwell dia; f.d. = feedwell depth)

a) For 100% recirculation:
   dia. f = 0.2 × tank diameter (Joint Committee WPCF/ASCE, 1977)
   f.d. ≤0.55 to 0.65 × SWD (Joint Committee WPCF/ASCE, 1977)
b) dia. f = 0.17 × tank diameter (Task Committee ASCE, 1979)
c) dia. f = 0.15 to 0.20 × tank diameter (Metcalf and Eddy, 1979)
   f.d. ≤1 m (Metcalf and Eddy, 1979)

---

8) Weir Loading (WLR)

a) For Qi ≤3800 m^3/day: WLR ≤124 m^3/day·m (Great Lakes, 1971)
   Qi ≥3800 m^3/day: WLR ≤186 m^3/day·m (Great Lakes, 1971)
b) WLR ≤120 to 480 m^3/day·m (Kalbskopf, 1970)
c) For:

Large Tanks:	Weir in Upturn Zone:	WLR ≤250 m^3/day·m (Metcalf and Eddy, 1979)
	Weir Away from Upturn Zone:	WLR ≤375 m^3/day·m (Metcalf and Eddy, 1979)
Small Tanks:	Average Flow:	WLR ≤125 m^3/day·m (Metcalf and Eddy, 1979)
	Peak:	WLR ≤250 m^3/day·m (Metcalf and Eddy, 1979)

---

where: A = surface area of settler (m^2)
D = tank diameter (m)
dia. f = feedwell diameter (m)
f.d. = feedwell depth (m)
Qi = wastewater flow into plant (m^3/h)
S = floor slope (rise:run)
SLR = solids loading rate (kg/m^2·h)
SOR = surface overflow rate (m^3/m^2·h)
SWD = sidewater depth (m)
T = theoretical detention time (h)
Vr = rake speed (rotation: revs/h; tip speed (m/min))
WLR = weir loading rate (m^3/day·m)

Dick (1970a, 1970b, 1974) and Dick and Young (1972), the performance of a tank with regards to thickening depends on the settling rate of the sludge and the rate of recycle as well as the tank area.

The inadequacy of SLR criteria alone is demonstrated from the data from two treatment plants as shown in Table 2.5. According to the empirical solids loading criteria listed in Table 2.4, the tanks should be experiencing thickening failure. In fact, they operated satisfactorily throughout the period for which the data in Table 2.5 was collected.

As mentioned earlier, depth (or retention time) influences the rate of removal of flocculant particles. Fitch (1958) demonstrated this with batch settling tests and asserted that both depth and overflow rate influenced flocculant clarification. Furthermore, he observed that there was an interaction between the two. Therefore, depth as well as surface overflow rate are empirical criteria aimed at providing adequate clarification. Clarification depth is the distance from the liquid surface to the top of the sludge blanket.

There will also be depth requirements for thickening and to accommodate changes in flow. Dick (1976) maintained that an adequate blanket height is required to prevent the clarified liquid from being withdrawn with the thickened sludge. Anderson (1945) examined the effect of sludge blanket depth on underflow concentration. He found that for a blanket depth of 1.2 to 1.5 m, the underflow concentration was in excess of 20,000 mg/L, while for a blanket depth of 0.3 m, the underflow concentration was reduced to 6000 mg/L.

TABLE 2.5: **Thickening Performance of Two Clarifiers**
(Adapted from Dick and Young, 1972)

	Solids Loading	
Location	Average ($kg/m^2 \cdot h$)	Maximum ($kg/m^2 \cdot h$)
Philadelphia	7.7	14.2
Middlesex	7.6	11.8

Adequate depth must also be provided to store excess sludge which accumulated during high flow periods when the solids feed rate exceeds the underflow removal rate (Metcalf and Eddy, 1979). Therefore, clarifier design must ensure that the tank depth is great enough to provide for clarification, thickening and solids storage. Unfortunately, no allocation of depth according to function is made for the criteria listed in Table 2.4. Criteria providing allocation of depth according to function would be the equivalent of specifying sludge blanket depths for average and peak flow conditions.

With regards to effluent weirs, Anderson (1945) investigated both loading rates and location. The existence of density currents which flow along the tank bottom and are deflected upward by the tank walls lead him to recommend weir loading rates which were related to weir location. Recommended rates are as per item 8c, Table 2.4.

2.3.2 Design Based on Solids Flux Theory. An alternative approach to the design of the tank with regards to thickening is based on "solids flux theory". The theory was first in introduced by Coe and Clevenger (1915) and applied to clarifier design by Dick and others (Dick, 1970a; Dick, 1970b; Dick and Young, 1972; Dick, 1974; Dick, 1976; Keinath *et al.*, 1976a, 1976b). Solids flux can be thought of as the intensity of the "solids rain" experienced at the bottom of the tank. It is the rate per unit of surface area at which solids are transmitted towards the tank bottom. Common units are kg/m^2·day (WPCF, 1976). Solids flux in a clarifier has two components:

settling flux due to particle settling and bulk flux due to the downward movement of the tank contents as the underflow is withdrawn for recycle and wasting.

TOTAL FLUX = SETTLING FLUX + BULK FLUX

or $$G_i = C_i v_i + (Qu \cdot C_i)/A \qquad (5)$$

where: G_i = total solids flux ($kg/m^2 \cdot d$)

C_i = suspended solids concentration in zone of uniform settling (mg/L)

v_i = ZSV = zone settling velocity (m/h)

Qu = underflow rate consisting of the sum of the recycle and wasting rates (m^3/h)

A = surface area of settler (m^3)

The rate of zone settling, v_i, has been found by a number of researchers to be a function of the solids concentration, i.e., $v_i = f(C_i)$ (Richardson and Zaki, 1954; Vesilind, 1968; Dick and Young, 1972). A number of empirical relationships have been developed to describe the velocity/concentration function. Three of the most common are listed in Table 2.6.

Solids flux theory holds that there is a concentration, C_L, which limits the downward transport of solids such that an increase in the rate of applied solids results in an increase in sludge blanket depth and the possibility of thickener failure. To

TABLE 2.6: Relationships Between Settling Velocity and Solids Concentration

Equation	Researcher
$v_i = vo\ (1 - kC_i)^{m^1}$	Richardson and Zaki (1954)
$v_i = vo\ e^{-k^1 Ci}$	Vesilind (1968)
$v_i = aC_i^{-n}$	Dick and Young (1972)

where: vo = Stokes settling velocity.
a, k, k^1, m^1, n = empirical constants,

determine the limiting solids flux associated with concentration C_L, a graphical procedure is employed based on the results of a battery of settling tests. At a variety of concentrations, zone settling velocities (v_i) are determined. Dick and Young (1972) recommend that the concentrations be developed as shown in Table 2.7.

The settling flux ($G_i = v_i C_i$) identified by the tests, when plotted on a graph of flux versus concentration, will produce a curve with characteristic rising and falling limb shown in Figure 2.2. The line from the desired underflow concentration, C_u, which is just tangent to the falling limb of the settling flux curve is the Yoshioka construction (Yoshioka, 1957). The line has a slope of $-$ Qr/A. The intercept of the Yoshioka construction with the abscissa identifies the limiting solids flux G_L. The ordinate of the intersection of the Yoshioka construction with the lower limb of the settling flux curve corresponds to the limiting concentration, C_L. Using the value of G_L, the area (A) required for thickening is determined as follows:

$$\alpha = MLSS/(C_u - MLSS) \tag{6}$$

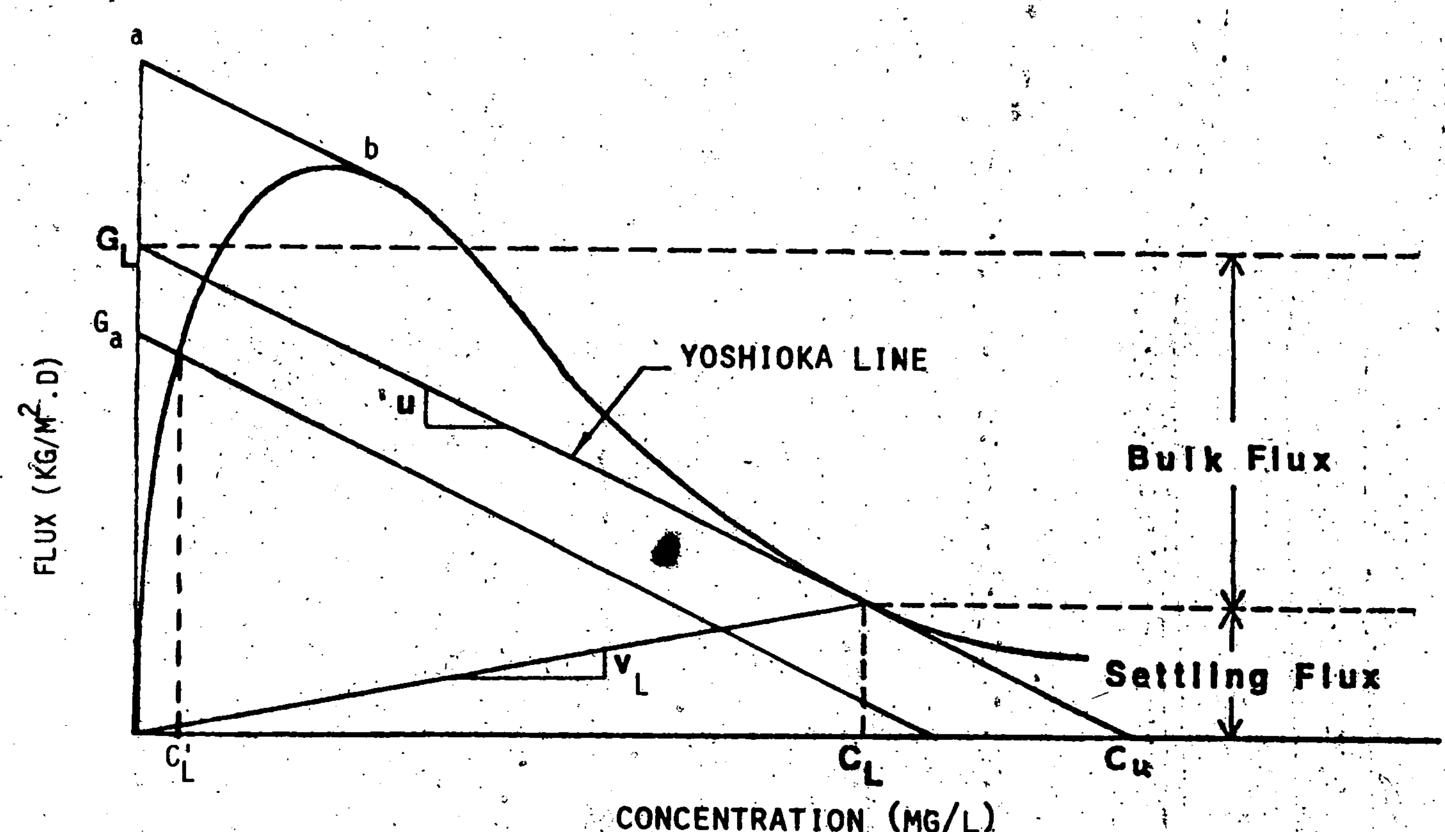
where: α = Qr/Qi = recycle proportion

C_u = concentration of suspended solids in the underflow from the settler (mg/L)

$$A = \frac{(1 + \alpha)\cdot Qi \cdot MLSS}{G_L} \tag{7}$$

TABLE 2.7: Suspension Components for the Development of Batch Flux Curves
(Adapted from Dick and Young, 1972)

Suspension Range	Components
Dilute	MLSS + clarifier effluent
Intermediate	MLSS
	MLSS + clarifier underflow
Concentrate	Clarifier underflow



C_L = suspended solids concentration at limiting solids flux, G_L

G_L = limiting solids flux

C_u = underflow suspended solids concentration

u = slope of Yoshioka Line = Qr/A

v_L = zone settling velocity at limiting solids flux.

G_a = applied solids flux; C_L' = dilute blanket concentration.

Figure 2.2 Batch Settling Flux Procedure.

where: G_L = limiting solids flux (kg/m^2·d)

The area so determined is then compared with the area required for clarification, ususally determined from an empirical overflow rate. The largest area is the required settler surface area.

Keinath et al. (1976a, 1976b) extended the basic solids flux design by plotting additional constraints on the solids flux graph. The graphical area which is bounded by the plotted constraints is the feasible design domain. Additional constraints identified by Keinath were:

a) clarification requirements developed either from overflow rates or empirical models;

b) maximum recycle limits; and

c) minimum underflow concentration limits.

Bisgoni and Dick (1978), in commenting on the constraint procedure, felt that a maximum recycle constraint was unjustified as documented evidence of the effect of recycle on effluent quality was not available. Further, they advised that an additional constraint - a maximum underflow concentration - be added. Their experience indicated that the use of batch flux curves could lead to the prediction of unjustifiably high underflow concentrations.

For an applied solids flux of G_a (Figure 2.2), the concentration C_L' is determined by the intersection of the Yoshioka line with the rising limb of the solids flux curve. This is the concentration of the dilute sludge blanket which, for an underloaded settler, occupies the region from the top of the thick sludge blanket to the feed point (Liquidara and Keinath, 1983). For a given recycle rate, there is a maximum solids flux - identified by line a-b on Figure 2.2 - which can be transmitted by the dilute sludge blanket. If the applied solids loading exceeds this maximum, the dilute blanket propogates to the surface of the settler resulting in a high concentration of solids in the effluent. As a settler which is overloaded with respect to clarification is also overloaded with respect to thickening, the thick blanket also propogates upwards.

Experimental evidence indicates that the effluent suspended solids concentration increases as the dilute blanket concentration increases (Dietz and Keinath, 1982). The degree of flocculation in the settler is established by the detention period of the clear zone above dilute sludge blanket (Dietz and Keinath, 1982). Improved clarification and an increased capacity to withstand clarification failure therefore result from an increase in either the depth of feedwell submergence or the depth of the clarifier itself (Laquidara and Keinath, 1983).

Based on the theory developed by Kynch (1952), attempts were made to simplify the procedure for obtaining the settling flux curve. Kynch stated that the concentration just below the solids/liquid interface could be determined from the shape of the settling curve itself. A tangent to the settling curve at point "b" on Figure 2.3 will have intercepts H_i and t_i. The corresponding concentration and flux will be:

$$C_b = (MLSS \cdot H_o)/H_i \qquad (8)$$

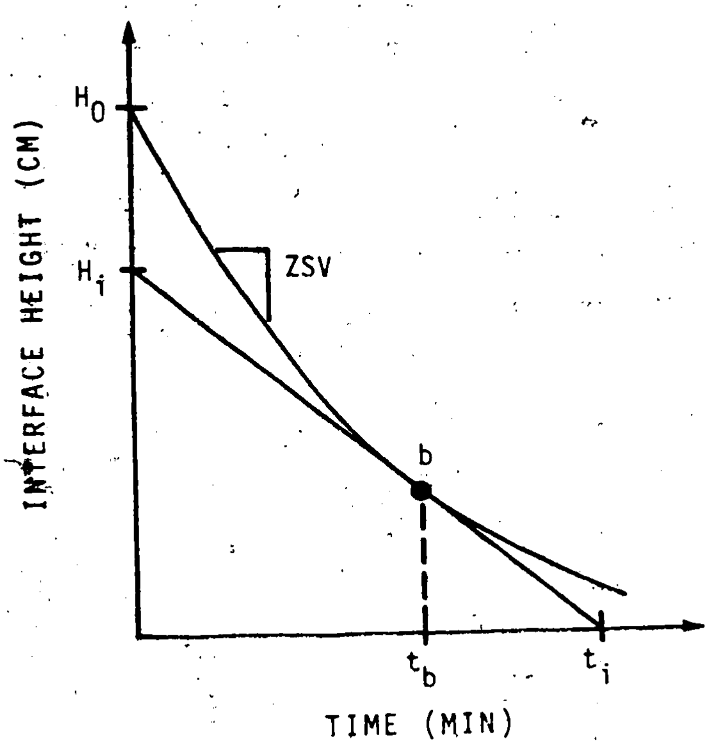


Figure 2.3 Kynch Construction for Determination of Solids Flux Values

and

$$G_b = (MLSS \cdot H_o)/t_i \tag{9}$$

where: G_b = the solids flux for point "b" on the settling curve $(kg/m^3 \cdot d)$

t_i = the X intercept of the tangent to the settling curve at point "b" (min)

As a variety of tangents can be drawn to the interface subsidence curve, a variety of C_b/G_b coordinates can be calculated and thus the settling flux curve determined from a single batch settling test.

Talmage and Fitch (1955) carried the process further and determined the thickening area without developing the solids flux curve at all. The procedure involves selecting the desired underflow concentration and calculating a value, H_u, as follows:

$$H_u = \frac{MLSS \cdot H_o}{C_u} \tag{10}$$

where: H_u = depth corresponding to the desired underflow concentration C_u (cm)

The time, t_u, to reach the desired underflow concentration, C_u, is determined from the tangent to the compression point "C" on the interface subsidence curve. The procedure is as shown on Figure 2.4. The required area is then:

$$A = \frac{Qi \cdot t_u}{H_o} \qquad (11)$$

Unfortunately, as reported by Vesilind (1975), research has shown that Kynch's theory does not apply to highly compressible materials. As activated sludge falls into this category, design must be based on a battery of tests.

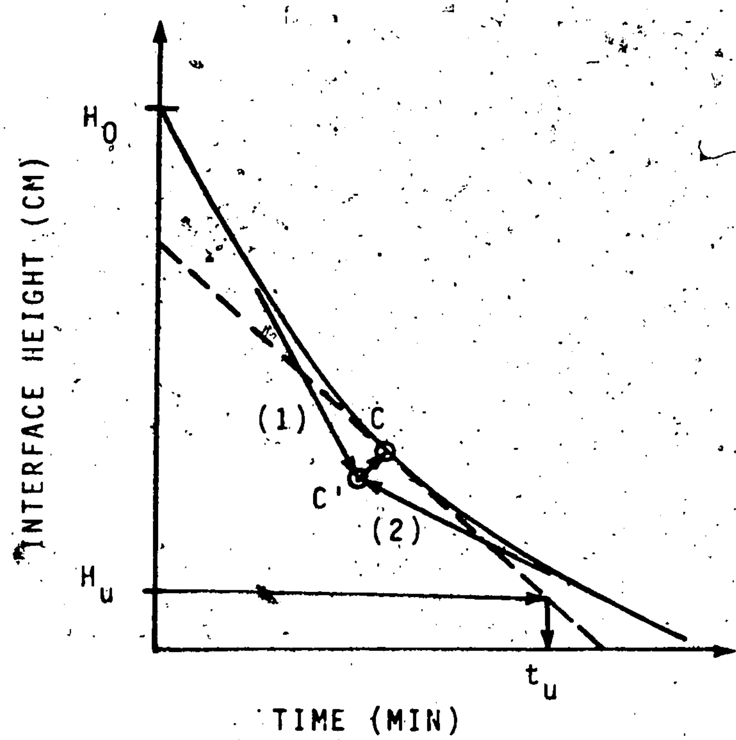
Solids flux theory also provides insight into the depth required for sludge storage. For an overloaded condition - the applied flux (G_a) exceeding the limiting flux (G_L) - Keinath et al. (1976b) determined the depth of sludge storage (ΔH) required by calculating the difference between G_L and G_a, multiplying by the time (ΔT) for which the overloading condition applied and dividing by the limiting concentration (C_L):

$$\Delta H = \frac{(G_L - G_a) \cdot \Delta T}{C_L} \qquad (12)$$

where: ΔH = settler depth required for sludge storage (m)

ΔT = period for which settler is overloaded (h)

G_a = applied solids flux (kg/m[illegible]



Procedure:

a.) tangent to upper limb, (1)
b.) tangent to lower limb, (2)
c.) bisect angle at C'
d.) tangent at C
e.) select t_u corresponding to required H_u.

Figure 2.4 Talmage and Fitch Construction for Determining Thickener Area.

Metcalf and Eddy (1979) recommend a similar procedure but used the average of the feed and underflow concentrations:

$$\Delta H = \frac{(G_L - G_a)\cdot\Delta T}{(MLSS + C_u)/2} \tag{13}$$

Despite the advantages derived by using solids flux theory in design, the method is still approximate. Conditions in the clarifier will differ to a degree from the ideal conditions assumed in theory. In particular, solids flux theory assumes that:

a) solids are uniformly applied over the cross-section of the tank (Dick and Young, 1972);

b) sludge removal produces a uniform downward velocity (Dick and Young, 1972); and that

c) settling rates in full-scale units can be determined by using batch settling tests (Fitch, 1979).

As has been shown, settler design involves the correct dimensioning of the tank to ensure adequate area and depth for thickening, clarification, and sludge storage. Besides being influenced by settler design, the performance of the activated sludge system depends on settler operation under field conditions. The designer must, therefore, also pay adequate attention to the dynamic behaviour and the controllability of the clarifier. These issues are further explored in the next section.

2.4 Operation

The mode of operating the activated sludge system will either directly influence clarifier performance by changing flux conditions within the settler or indirectly influence it by changing the characteristics of the suspension which is fed to the clarifier. Of the four activated sludge control options - air flow control, recycle rate control, wasting rate control, and incremental feeding (or step-feed) - the last three have a direct effect on settler performance and will be examined in further detail.

Whatever the control strategy employed, it must lead to a net benefit in total system performance. System performance is evaluated with regards to four objectives:

a) prevention of system failure;

b) minimization of discharge of BOD;

c) minimization of operating costs; and

d) minimization of sludge processing disposal costs (Keinath et al., 1979)

A number of characteristics of the activated sludge process and of wastewaters in general make evaluation of control strategies difficult. As pointed out by Olsson (1976), the response of the activated sludge system to operating changes is strongly non-linear. A second problem is that the recycle sludge line introduces a process feedback loop which makes the system difficult to model and understand (Tracy and Keinath, 1974; Andrews, 1975; Olsson, 1976). For instance, a change in recycle rate may improve conditions with respect to solids flux in the settler, yet degrade the clarification characteristics of the biological suspension. Changes in system operation introduce transitory effects which are superimposed on dynamics already created by variations in wastewater temperature, flow rate, and composition (Bisgoni and Dick, 1978).

2.4.1 <u>Recycle Control</u>

a) <u>Constant Recycle</u>

The recycle rate will either be set at a constant value or vary in proportion to the rate of flow of sewage entering the aeration tank. For constant recycle, the sludge blanket will rise and fall throughout the day as the plant inflow varies. Recommended recycle rates are as shown in Table 2.8.

McKinney has stated that a poorer effluent quality results if a recycle rate is used which is in excess of that necessary to provide adequate distance between the weirs of the settler and the sludge blanket during the highest flows associated with the diurnal cycle. Keinath <u>et al</u>. (1979) supported the view that high recycle rates degrade effluent quality. Observations made by Sorensen (1979; 1980) based on pilot plant operation indicated that effluent quality was not particularly affected by the recycle rate. Using a fractional factorial design, Tuntoolavest <u>et al</u>. (1980) examined the effect of a number of variables, including recycle rate, on flocculation in the aeration basin. Their experimental results demonstrated that the effect of recycle was interactive, i.e., the effect of recycle depended on the levels of the other variables. At low air flow rates in the aeration basin, increasing sludge return reduced

TABLE 2.8: Typical Recycle Flow Rate Percentages
(After Tsugita *et al.*, 1977)
Recycle rate as a percentage of influent flow

Type of process	Average (%)	Lower Limit (%)	Upper Limit (%)
Conventional	30	15	75
High Rate	20	10	50
Step-Feed	50	20	75
Contact Stabilization	100	50	150
Extended Aeration	100	50	150

effluent solids. It was postulated that an increase in recycle resulted in a greater number of preformed flocs in the aeration basin to which colloids could adhere. At high air flow rates, an increase in recycle resulted in a deterioration of effluent quality. It is likely that high air flow rates create floc disruption and increasing the floc concentration in the aeration tank by increasing recycle merely produces a greater number of primary particles. These particles have a maximum dimension of between 0.5 μ and 5 μ and are not settleable.

With regards to underflow concentration, solids flux theory and more particularly "the state point concept" can be used to predict the underflow concentration and control recycle (Keinath et al., 1979). The state point is the intersection of the Yoshioka line with a line having a slope of Qi/A and an intercept of zero. The coordinates of the state point will therefore be:

$$G_{sp} = \frac{Qi \cdot MLSS}{A} \tag{14}$$

and

$$C_{sp} = MLSS \tag{15}$$

where: G_{sp} = solids flux corresponding to state point ($kg/m^2 \cdot d$)

C_{sp} = concentration corresponding to state point (mg/L)

Increasing the recycle rate is the equivalent of a clockwise rotation of the Yoshioka line about the state point, as shown in Figure 2.5.

The underflow concentration corresponding with each of the recycle rates will decrease as the recycle rate is increased.

If the Yoshioka line cuts the flux curve, as is the case for the dashed line with slope U_4 in Figure 2.5, then the settler is overloaded with respect to thickening. An appropriate response to an overloaded settler is to increase the recycle rate so that the Yoshioka line no longer cuts the batch flux curve. Using the graphical representation, an increase in influent flow rate is represented by an anti-clockwise rotation about the origin of the line with slope Qi/A.

An alternative and equivalent procedure to graphical examination is a numerical procedure based on Dick's velocity/concentration relationship of Table 2.6. For a given settler area (A) and state-point MLSS concentration, the following trial-and-error procedure establishes the recycle rate which results in critical loading:

STEP 1: Select a value of recycle, $\alpha = Qr/Qi$ (16)

STEP 2: From a battery of tests, select the batch settling coefficients a, n

STEP 3: Determine the limiting solids flux (G_L):

$$C_L = \left[\frac{a(n-1)A}{\alpha Qi}\right]^{1/n} \qquad (17)$$

Page 41 was removed due to copyright restrictions. The page contained Figure 2.5, obtained from a report by Keinath *et al.* (1979), showing a batch settling curve, the "state point", and lines representing various recycle rates.

where: C_L = suspended solids concentration at limiting solids flux (mg/L)

$$V_L = aC_L^{-n} \tag{18}$$

where: V_L = the zone settling velocity at limited solids flux (m/h)

$$G_L = C_L \cdot V_L \tag{19}$$

STEP 4: For a Yoshioka line corresponding to a recycle rate of α and tangent to the solids flux curve, the underflow concentration is:

$$C_u = \frac{G_L \cdot A}{\alpha Qi} \tag{20}$$

STEP 5: Determine the MLSS' concentration corresponding to the selected recycle and based on a solids balance around the aeration tank:

$$MLSS' = C_u \cdot \left(\frac{\alpha}{1+\alpha}\right) \tag{21}$$

If MLSS' equals the state-point MLSS concentration, the critical recycle rate has been found. If not, then a new value for α is selected and the procedure repeated.

The above procedure will be particularly useful in conjunction with a computer.

Dick and Young (1972), using this method, were able to obtain a comparison of predicted and observed underflow and MLSS concentrations. For two pilot plants examined, the ranges were -10 to +20 percent and -20 to +20 percent. White (1975a) found he was able to predict underflow concentrations to within ±20 percent of observed values. Other studies by Schaffner and Pipes (1978), Hibberd and Jones (1974) and Munch and Fitzpatrick (1978) were much less successful in applying flux procedures to predict underflow concentrations.

Traditionally, recycle rate has been set based on the sludge volume index (SVI). Using the SVI values, the maximum underflow concentration, Cu, is calculated as follows (Dick, 1976; Schaffner and Pipes, 1978):

$$C_u = 10^6/SVI \tag{22}$$

Based on a mass balance of solids entering and leaving the aeration tank and neglecting solids in the feed from the primary settler, the recycle rate required is:

$$\alpha = Qr/Qi = \frac{MLSS_{st}}{C_u - MLSS_{st}} \tag{23}$$

where: $MLSS_{st}$ = setpoint MLSS

The plant operator can check the accumulation of solids in the settler using the previously determined values and the measured influent rate, Qi (Schaffner and Pipes, 1978):

$$\text{Accumulation} = G_a A - Q_i C_u \qquad (24)$$

Pipes (1979), in examining this procedure, analyzed data from a large number of plants which were experiencing bulking. He found, that for two-thirds of plants, the underflow concentration from the settler exceeded 10^6/SVI. Dick (1976) has also commented on the inadequacy of SVI as a predictor of underflow concentration.

b) Proportional Recycle

As an alternative to constant recycle, proportional recycle is employed. Proportional recycle is used to establish food to micro-organism (F:M) control and, thereby, maintain a constant growth rate in the aeration tank (Buhr, 1975). The recycle rate can be set based on a number of variables including:

i) Influent flow rate (Tsugita *et al.*, 1977).

ii) Influent total organic carbon (TOC) (Olsson, 1976).

iii) The difference between influent and effluent TOC per unit of volatile MLSS (Flanagan, 1975).

iv) The respiration rates of the recycle and mixed liquor sludges (Flanagan, 1975).

v) The solids concentration of the wastewater entering the aeration tank (Pearse, 1942).

Cashion et al. (1979) used a mathematical model and pilot-scale facility to examine the F:M strategy. They found that effective F:M control could not be applied by varying the recycle flow alone and that biosolids storage in the recycle loop was required. Even with biosolids storage, F:M control, although decreasing the variability of soluble TOC in the effluent, increased particulate TOC due to the introduction of hydraulic transients.

Using dynamic models and computer simulations, Busby and Andrews (1975) determined that system performance, as measured by the kilograms of BOD discharged per day in the effluent, could be improved by 10 percent using proportional recycle. Further, the simulations indicated that increasing the recycle ratio increases the MLVSS concentration in the aeration tank and hence improves BOD removal. As increasing the recycle rate leads to more solids in the effluent from the settler there is, therefore, a recycle ratio which optimizes system performance. This optimum recycle ratio was thought to be a function of the sludge settling characteristics and the detention time in the aeration tank (Busby and Andrews, 1975).

Disadvantages of proportional recycle control include the need for variable speed control for the recycle pumps (McKinney) and the increased possibility of solids washout (Tsugita et al., 1977). Additional operator attention to the recycle rate will also be required. A manual prepared for the U.S. EPA (Tsugita et al., 1977) recommends that the operator check and [illegible] two hours for proportional control compared with once every eight-hour shift for constant recycle.

2.4.2 Sludge Blanket Control. The depth of the sludge blanket in the clarifier can be used to either control the recycle rate or, using a constant recycle rate, to control sludge wasting (Sorensen, 1979). Sorensen (1979) examined the latter strategy using two pilot plants in parallel. Strong evidence indicated that the maintenance of a high blanket produced a better quality of effluent. Sorensen's results, comparing automatic blanket control with manual operation, are shown in Figure 2.6.

Clough (1974) and Hill (1974) have also commented on the improvement of effluent quality obtained when the incoming clarifier flow is discharged below the sludge blanket. Resch (1980) found that the efficiency of vertical-flow Dortmund tanks improved as the depth of the blanket above the feedwell outlet increased. He felt that because such tanks could maintain a good effluent quality at higher loadings in comparison to conventional settlers, they should be considered for use in middle to large-sized works. However, evidence from the thickening of coal refuse suggests that the introduction of feed below the blanket is detrimental to the solids concentration in the underflow (Keleghan, 1980).

Use is made of the sludge blanket in vertical-flow settlers used for the treatment of potable water. Gregory (1979) maintains that the mechanisms of clarification in such systems are: floccula- [illegible]

Page 47 was removed due to copyright restrictions. The page contained Figure 2.6, obtained from a paper by Sorensen (19̄9), showing comparison of tw effluents, one effluent fr ... rolled on the basi f sludge blanket ... ond ... ll of 2 to 3 m are necessary for good stability of operation. Deterioration of blanket stability can occur because of poor distribution of inlet flow. To maximize flocculation in blankets, Degremont have developed a pulsated blanket settler (Bebin and Jacquart, 1978). A feed chamber incorporating a vacuum which is alternatively applied and broken is responsible for the pulses within the bed.

Olsson (1976), though noting the possibility of improving effluent suspended solids concentration with a deep blanket, felt that wasting based on blanket depth may be a poor strategy. He reasoned that, based on this strategy, wasting will occur when the blanket is at its highest and consequently, the inflow and recycle rate (assuming proportional recycle) are also at their highest. The resulting sludge will, therefore, occupy a large volume and have a dilute concentration. Secondly, as the wasting rate is typically three percent of the recycle rate, any wasting strategy aimed at improving settler performance is likely to have limited "control authority". McKinney noted that wasting will be controlled by the operational requirements of the sludge processing train and not by settler requirements.

A number of devices are available to determine blanket depth when employing a blanket strategy or checking for overloading. The devices include:

a) a series of sampling pumps with inlet lines mounted at various depths (Tsugita et al., 1977),

b) electronic sensors based on the measurement of light attenuation between source and photocell detector which may either be portable (Engler, 1978), fixed, or automatic (Eur-Control, Partech),

c) sensors based on the measurement of the attenuation and scattering of ultrasonic waves between the transmitting and receiving elements (Neo Nishi Hara),

d) a sight glass with a light source at the lower end (Tsugita _et al._, 1977),

e) a core sampler consisting of a plexiglass tube with ball valve which is seated when the sampler strikes the tank bottom (Sherman, 1980),

f) an infrared blanket sensor (Flanagan, 1975).

Walton (1974), using a photocell device, measured variations in blanket depth of a factor of two over the basin area.

2.4.3 Step-Feed Control. A specialized activated sludge configuration allows for the addition of influent feed from the primary settler basin along the length of the aeration basin. The concept, known as "step-feed", originated with Gould (Torpey, 1948). A constant recycle rate with sludge returned to the head end of the tank is used with the system. For an overloaded settler, the control action involves shifting the addition of influent feed towards the outlet end of the aeration tank. As a consequence, biosolids are stored in the aeration tank near the inlet end while the flow leaving the aeration tank contains a reduced solids concentration. The net effect is to reduce the solids loading applied to the settler. Using step-feed, Torpey and Chasick (1955) achieved average suspended solids concentrations at three New York treatment plants of 6, 19 and 17 mg/L.

Busby and Andrews (1975), using computer simulations, examined the utility of various control strategies in preventing thickening failure when the system was undergoing sludge "bulking". Greater benefits were derived from the step-feed strategy than either wasting rate or recycle rate based on sludge blanket height. In addition, step-feed was thought to be valuable in preventing upsets caused by shock organic overloads or toxic slugs.

Many of the problems in understanding and controlling activated sludge operation arise because of the unique hydraulic conditions in the settler. Density currents, which influence performance, occur in all secondary clarifiers. Clarifiers do not operate at steady-state but are subject to a number of time-varying disturbances. As well, the occurrence of other mechanisms besides settling - mechanisms such as flocculation and entrapment - make it difficult to relate hydraulic efficiency to removal efficiency. These points are further discussed in the following section.

2.5 Hydraulic Considerations

There is general acceptance that settler performance is dependent on the hydraulic characteristics of the tank. A substantial body of literature exists which describes the development of techniques for evaluating hydraulic charactertistics.

2.5.1 <u>Evaluation Techniques</u>. Historically, basin hydraulics have been studied by measuring the dilution of a tracer as it passes from inlet to point of discharge. Tracers used include dyes, salts, and radioactive isotopes. The method of addition will either be continuous, for determining the direction of flows and the location of stagnant areas or instantaneous, for the development of residence time distribution (RTD) curves (Mazurczyk <u>et al</u>., 1980).

Based on the evaluation of a residence time distribution curve, any reactor can be classified into one of three hydraulic regimes: plug flow, complete mix or arbitrary flow. For a plug flow reactor, the tracer passes as a slug through the system with no dilution. In contrast, for the complete mix system, the tracer is instantaneously mixed throughout the tank volume. The resulting concentration of tracer in the system effluent will exhibit exponential decay:

$$C = C_0 \ EXP \ (-t/T) \tag{25}$$

where: C = concentration of tracer

C_0 = initial tracer concentration; the mass of tracer added divided by the tank volume

t = time after tracer addition

T = theoretical retention time; V/Q

All settling tanks exhibit characteristics of the arbitrary flow regime which is intermediate between plug flow and complete mix. Deviation from plug flow in settlers can be attributed to the existence of stagnant zones, small-scale eddies and large recirculation patterns (Villemonte et al., 1966). Hydraulic efficiency for arbitrary flow systems is characterized by a number of parameters measured from the RTD curve, which is generally graphed in a non-dimensional format (C/C_0 versus t/T). Some of these parameters are indicated in Table 2.9 for the RTD curve of Figure 2.7.

Alternatively, RTD data can be used to fit parameters to various flow models. Common models include the dispersion, N-tanks-in-series and dead space models. The dispersion model was developed from a mass balance of tracer entering and leaving an element of flow and, assuming that the mechanism of tracer mixing is similar to that for diffusion of matter on a microscopic scale, by applying Fick's second Law (Krenkel, 1962). The dispersion model contains two parameters - the dispersion index (d) and the average flow-through time, $\bar{t}$. Levenspiel (1962) developed a procedure for estimating these parameters based on a statistical analysis of the RTD curve. Using Levenspiel's procedure, mean flow-through time is related to the centroid of the RTD curve as follows:

$$\bar{t} = \frac{\Sigma t \cdot C}{\Sigma C} \qquad (26)$$

where: C = concentration of dye in the reactor effluent collected at time t following tracer injection.

TABLE 2.9: Parameters for Characterizing Hydraulic Efficiencies from Residence Time Distribution Curves
(Adapted from: Dague and Baumann, 1961; Levenspiel, 1962; Villemonte *et al.*, 1966; Villemonte and Rohlich, 1962)

Parameter	Estimate of:
$T_i = t_i/T$	Severe short-circuiting
$T_p = t_p/T$	Stagnant zones
$T_c = t_c/T$	Amount of eddying
$T_b = t_b/T$	Turbulence and large scale recirculation
$\bar{t}/T$	Deadspace
$(\bar{t} - t_p)/\bar{t}$	Index of short-circuiting
t_a/T	Flow through efficiency

where:
- t_a = time to half area; $A_1 = A_2$
- t_b = as defined by Figure 2.7
- t_c = as defined by Figure 2.7
- t_i = first visual trace time
- t_p = time to peak concentration
- $\bar{t}$ = centroid of RTD curve; $\bar{t} = \Sigma t \cdot C/\Sigma C$
- T = theoretical retention time; $T = V/Qi$

Page 54 was removed due to copyright restrictions. The page contained Figure 2.7, obtained from a paper by Villemonte *et al.* (1966), showing a tracer washout curve with the parameters used to characterize it.

The variance of the RTD curve is calculated according to the following formulae:

$$\sigma_t^2 = \frac{\Sigma t^2 C}{\Sigma C} - \bar{t}^2 \qquad (27)$$

$$\sigma_\theta^2 = \left(\frac{\sigma_t}{\bar{t}}\right)^2 \qquad (28)$$

where: σ_t^2 = the variance of the RTD curve (C vs t)

σ_θ^2 = the variance of the non-dimensional RTD curve (C/C_0 vs t/T)

The variance of the non-dimensional RTD curve is related to the dispersion index for one of three possible system configurations - open, closed and open/closed (Levenspiel, 1962). Commonly, the configuration will be closed as the tracer is injected into and collected from pipes entering and leaving the reactor. For a closed vessel the following equation applied:

$$\sigma_\theta^2 = 2d - 2d^2 \, (1 - \mathrm{EXP}\,(-1/d)) \qquad (29)$$

The above equation is solved by cut-and-try methods to obtain the dispersion index (d).

Another model commonly used to characterize flow in reactors is the N tanks-in-series model. The reactor is assumed to be represented by a series of "N" completely mixed vessels. The predicted concentration is then:

$$C = C_0 \cdot \frac{N^N}{(N-1)!} \cdot \theta^{N-1} \cdot EXP(-N\theta) \qquad (30)$$

where: C_0 = initial concentration

N = number of completely mixed tanks of equal volume

θ = normalized time = $t/(V/Q)$

As the value for N approaches infinity, the reactor assumes plug flow conditions. Estimates of N can be obtained using the variance of the the non-dimensional RTD curve as follows:

$$N = 1/\sigma_\theta^2 \qquad (31)$$

Alternatively, the value for N can be estimated using a procedure known as nonlinear least squares. The technique consists of a computer search for estimates of the model parameters with the search terminating when the residual sum of squares - the sum of squares of the differences between the observed and predicted values - has reached a minimum (Wolfe, 1965; Thackston et al., 1967). Details of the procedure as applied to the N-tanks model are contained in a recent paper (Chapman, 1982).

A model presented by Levenspiel (1962) enables the existence of unmixed volumes in a reactor to be estimated. The model visualizes the contents of the reactor to be divided into two zones - a zone of "active" mixing where the tracer is instantaneously and completely dispersed and a zone of "dead" space into which the tracer cannot penetrate. For such a reactor, the RTD curve resembles that of a reactor with a much smaller volume. The appropriate model is as follows:

$$C = C_0 \cdot \left(\frac{V}{Vb}\right) \cdot EXP\left(-\frac{V}{Vb} \cdot \frac{t}{T}\right) \tag{32}$$

where: V = total reactor volume

Vb = "active" volume

The slope (m) of the $\ln(C/C_0)$ versus t/T plot provides an estimate of the fraction of the volume which is active,

$$V/Vb = -m \tag{33}$$

Attempts have been made to use residence time distribution curves in conjunction with data from settling columns to design settlers (Silveston, 1969). However, a number of researchers have questioned the effectiveness of tracer studies in evaluating settler performance (McKinney; Fiedler and Fitch, 1959; Bayley and Adams, 1964; Hall, 1966; Kalbskopf, 1966; Silveston et al., 1979; Mazurczyk et al., 1980). Mazurczyk et al. (1980) for instance, were unable to establish any direct correlation between dispersion curve characteristics and performances as measured by effluent suspended solids concentrations. Bayley and Adams (1964) found that, although baffling improved the hydraulic efficiency of a settler, no corresponding improvement in solids removal resulted. Kalbskopf (1966) observed that agitation in the inlet zone actually improved solids removal. It was therefore concluded by a number of researchers that the effect of flocculation on solids removal was not accounted for in tracer study results (Bayley and Adams, 1964; Hall, 1966; Kalbskopf, 1966).

A second major problem with tracer studies is the non-reproducibility of residence time distribution curves (Bayley and Adams, 1964; Bruce, 1964; Silveston et al., 1979). As stated by Hall (1966): "...a given pattern is not fully reproducible, though the broad pattern may be reproduced." Silveston et al. (1979) found that variations in RTD curves could be attributed to small changes in tracer density, injection time and temperature from test to test.

Besides conventional tracer studies, a number of other techniques have been used to evaluate settler performance. Sludge flow within the clarifier has been monitored by using a tracer which is absorbed onto a portion of MLSS entering the tank (Katz and Geinopolos, 1964; Mazurczyk et al., 1980). Velocity profiles can be established using current meters or floats with vanes set for the depth at which velocity is to be measured (Anderson, 1945).

Identifying the solids profile and sampling the effluent at several points along the weir are techniques which have been successfully employed (Anderson, 1945; Mazurczyk et al., 1980).

2.5.2 Density Currents. The most prominent hydraulic feature of secondary settlers is a vertical roll induced by density effects associated with the incoming flow. The existence of density currents in secondary settlers was first identified by Anderson (1945) at Chicago and subsequently confirmed by others (Sawyer, 1956; Hamlin, 1966; Annis, 1979; Crosby and Bender, 1980).

From the numerous studies, a characteristic flow pattern emerges:

a) the incoming flow plunges from the feedwell until it is deflected laterally by the sludge blanket,

b) the flow then moves as a thin layer across the top of the blanket until turned upwards towards the weirs by the tank sidewalls,

c) a countercurrent is induced in the top of the tank such that up to two thirds of the tank volume is moving back towards the inlet (Anderson, 1945).

Based on this characteristic pattern, Collins and Crosby (1980) subdivided settler volume into four zones: inlet zone, horizontal zone, inactive zone and upflow zone. The depth of the horizontal zone is dependent on the settling rate of the incoming suspension (Mazurczyk et al., 1980; Collins and Crosby, 1980). The depth [illegible] zone decreases as the settling rate decreases.

With respect to the movement of solids below the horizontal flow zone, Anderson (1945) found no perceptible velocity in the sludge blanket. Mazurcyzk _et al_. (1980) established that the solids were carried along the horizontal flow zone above the sludge blanket interface. Eventually, the solids recirculated downward and inward toward the inlet structure and sludge draw-off.

The precise effect of density currents on solids removal is unknown. Potential benefits of density currents include increased flocculation and increased movement of solids into the quiescent blanket (Mazurczyk _et al_., 1980). Alternatively, Sawyer (1956) expressed the opinion that the extreme turbulence created at the feedwell by the plunging flow reduced the concentration at which thickened sludge is removed. The velocity of the current above the blanket increases the possibility of scour. In this regard, Clements (1969) reported that a velocity of 20 mm/sec will resuspend activated sludge flocs.

2.5.3 Transient Loadings. In addition to density currents, the presence of transient loadings in the activated sludge system influences removal efficiency. Almost all sewage treatment plants experience large variations in influent wastewater composition, concentration and flow rate. Typically, flow into municipal activated sludge plants varies from about 50 percent to 150 percent of the daily mean flow (Eckhoff and Jenkins, 1966). Variations in flow are also increased within the plants as influent, sludge wasting and a variety of [illegible] turned on [illegible] off.

A complete assessment of clarification performance therefore requires that the "dynamic" response of the clarifier to time-varying loads be established. Such information is required in order to evaluate steady-state design criteria, determine appropriate sampling frequencies for data loggers and sensors monitoring effluent quality, and for developing control strategies and optimization techniques. For instance, Lech <u>et al</u>. (1978), having examined the efficacy of a number of control strategies using computer simulations, concluded that the choices for measured and controlled variables depended heavily upon the dynamic behaviour exhibited by the settler. At a workshop on automatic control, Andrews (1975) also commented on the need for an improved clarifier model so that computer control strategies could be subsequently explored.

Although a number of studies have examined the effect of transient loadings on activated sludge, they have generally concentrated on soluble BOD removal in the aeration basin. Only a very few researchers have examined the influence of transients on clarification efficiency. Shih (1976) used time-series analysis to analyze data collected from three full-size activated sludge plants. He found that as MLSS concentrations were negatively correlated with influent flow rates, no significant correlations were found relating plant influent flow rate to total effluent BOD_5. From the results, it was concluded that as sewage treatment plants have internal compensating capacity to dampen out diurnal variations, no hour-to-hour control was needed.

Porta <u>et</u> al. (1980) observed that effluent from a full-scale clarifier deteriorated due to significant hourly variations in influent flow. The source of the influent variations was traced to the action of influent pumps. Some dampening of the surges occurred before they entered the final clarifiers. By implementing measures to control the surges, the city of Detroit was able to forego the addition of four clarifiers at cost savings estimated to be $18 million.

Collins and Crosby (1980) have also identified a number of characteristics of the transient performance of secondary settlers. They found that turbulence, though quickly induced, dampens out slowly. Secondly, the movement of the rake arm in the clarifier was observed to result in a trailing wave of solids, 90° out of phase with the rake.

In conclusion, this section on hydraulics has reviewed the use of tracer studies in evaluating settlers and the effects of density currents and transient loadings on clarificiation efficiency. Although tracers have long been applied to the evaluation of settler hydraulics, the evidence suggests that there is no direct correlation between characteristics of RTD curves and suspended solids removal. Flocculation, an important mechanism in solids removal, is poorly estimated by conventional tracer techniques. The existence of a vertical roll above the sludge blanket is well documented. These currents increase mixing in settlers. Transient loadings are created by variations in the influent received at the plant and the operation of pumps within the plant. The small amount of research devoted to the influence of transients on clarifier efficiency indicates that, although there is a certain amount of dampening within the plant, transients can lead to a significant deterioration in effluent quality.

2.6 Clarification

2.6.1 Bioflocculation. A number of questions arise as to the nature of solids removal in the secondary settler. In particular, what are the fundamental mechanisms involved in the flocculation of activated sludge particles? Secondly, given the flocculant nature of activated sludge suspension and the non-ideal hydraulic characteristics of the settler, how can clarifier performance be estimated? Bioflocculation and the modelling of clarification will therefore be the next topics examined.

Activated sludge microorganisms are mainly gram negative bacteria which carry a net negative charge (Tenney and Verhoff, 1973; Brown and Lester, 1980). The cells are therefore hydrophilic in solution and with a typical size of 1 to 100 μ fall into the supracolloidal range as defined by Table 2.10. Bacteria will tend to form a stable colloidal suspension because of the repulsive forces developed by their similar charge.

Flocs, or packets of bacteria, have been visualized as consisting of a "backbone" of filaments to which zoogleal bacteria become attached (Parker et al., 1972; Palm et al., 1978). The "glue" holding the similarly charged cells together consists of long-chain polymers (O'Melia, 1970; Tenney and Verhoff, 1973). Excessive growth of the filaments into the bulk fluid results in a sludge which settles and compacts poorly (Palm et al., 1978; Sezgin et al., 1978). A

TABLE 2.10: Size Classification of Solids
(After Rickert and Hunter, 1972)

Fraction	Ideal Size Limits
Settleable	100 μ
Supracolloidal	1 to 100 μ
Colloidal	1 mμ to 1μ
Soluble	1 mμ

variety of nutrient and environmental conditions have been identified as possible causes for the shift from zoogleal to filamentous microbial populations. Potential causes include process loading intensities (Logan and Budd, 1956), specific combinations of substrate loading and dissolved oxygen (Palm *et al.*, 1978), aeration tank configuration (Tomlinson, 1976), and biological solids retention time (Bisgoni and Lawrence, 1971). Logan and Budd (1956) observed a lag period of four days between a change in environmental conditions in the aeration tank and a corresponding change in SVI.

The clarity of the supernatant is thought to be dependent on the presence of naturally-produced polymers (O'Melia, 1970; Tenney and Verhoff, 1973). These polymers are anionic and consist mainly of polysaccharides with some protein and nucleic acids (Brown and Lester, 1980). Bacteria are flocculated through the mechanism of polymer bridging which is presented schematically in Figure 2.8. The polymer is adsorbed at the surface of the bacteria, with the remainder of the molecule extending into the solution. If the polymer length is sufficient to bridge the minimum distance established by repulsive forces and a second particle becomes attached, a bacteria-polymer-bacteria complex results. The attachment of the polymer to the cell surface is by means of ion exchange and is reversible (Tenney and Verhoff, 1973). There is therefore an optimum polymer dose, with over-dosing resulting in all sites on the cell becoming covered with polymers (see Reaction 4, Figure 2.8).

Page 66 was removed due to copyright restrictions. The page contained Figure 2.8, obtained from a paper by O'Melia, showing schematically the steps occurring during polymer bridging.

The degree of bioflocculation is dependent on pH, the concentration of the cells, and the bacterial growth phase (Tenney and Verhoff, 1973). As shown in Figure 2.9, it is only during declining or endogenous growth that the ratio of polymer to cell mass is sufficient to ensure good flocculation. Tenney and Verhoff (1973) postulated that exocellular polymers are added to the solution during cell lysis. The presence of divalent metal ions - Ca^{++} and Mg^{++} - enhances the ability of the anionic polmers to adsorb the negatively charged cells (O'Melia, 1970).

The intensity of agitation of the bulk fluid also influences the flocculation of cells. Parker et al. (1972) determined that conditions in activated sludge aeration tanks are conducive to floc breakup rather than floc aggregation. The turbulence in the aeration tank disrupts the floc by either fracturing the filament "backbone" (floc splitting) or shearing the polymer-cell bond (surface particle erosion). The resulting size distribution of particles in the aeration tank is bimodal, reflecting the two mechanisms of floc breakup. "Primary particles" due to surface erosion have dimensions in the 0.5 to 5 μ range. Floc sizes, in the 50 to 500 μ range, are established by floc splitting. Tuntoolavest et al. (1980) found that the effectiveness of the final settler is greatly influenced by the concentration of non-settleable primary particles.

2.6.2 Clarification Models. Conditions in the activated sludge aeration tank will determine the clarification, settling, and compaction characteristics of the mixed liquor which is introduced into the

Page 68 was removed due to copyright restrictions. The page contained Figure 2.9, obtained from a paper by Tenney and Verhoff (1973), showing the experimentally obtained relationship between microbial growth, polymer production, and turbidity.

settler. For a given suspension quality, a knowledge of how various settler parameters affect the concentration of solids in the effluent is required for good clarifier design and operation. A number of models are currently available for evaluating settler performance in terms of the effluent suspended solids concentration.

Pflanz (1969) carried out a two-year study of secondary sedimentation on one circular and two rectangular full-size tanks. The effect of overflow rate (OR), influent solids concentration (MLSS), mixed liquor settleability as measured by the sludge volume index (SVI), temperature and wind on the concentration of solids in the clarified effluent was studied. Based on Pflanz's work, a number of other researchers have developed models for clarification in secondary settlers. Ghobrial (1978) and Busby and Andrews (1975) in their papers present a model of the following form:

$$Ce = kpf \cdot \frac{(Qi + Qr)}{A} \cdot MLSS \qquad (34)$$

where: Qi = rate of plant inflow

Qr = rate of settler recycle

Ce = effluent suspended solids concentration (mg/L)

kpf = Pflanz proportionality constant

Also based on Pflanz's work, Keinath *et al.* (1976a) developed a slightly different equation:

$$Ce = 4.5 + k'pf \cdot (Q[illegible]) \cdot MLSS \qquad (35)$$

where: k'pf = modified Pflanz proportionality constant

A modification to the equations developed directly from the Pflanz research was proposed by Ghobrial (1978) based on his studies of a bench-scale activated sludge system. These studies indicated that the sludge blanket level in the clarifier has a pronounced effect on the efficiency of the tank when the blanket rises above a specified limit. His modification assumes the following form:

$$Ce = kn \cdot \frac{(Qi + Qr)}{A} \cdot MLSS \cdot \frac{V_s/V}{(V_s/V)_{cr}} \qquad (36)$$

where: kn = Ghobrial constant (h/m)

V_s/V = ratio of sludge volume to settler volume

$(V_s/V)_{cr}$ = critical ratio of sludge volume to settler volume

The critical sludge blanket height depends on tank configuration, inflow and outflow arrangements, bottom slope and a number of other factors. Unfortunately, although a value of $(V_s/V)_{cr}$ is presented for the test apparatus, no method was given for its computation for other applications.

An EPA study carried out by Rex Chainbelt Inc. (Agnew, 1972) resulted in two models. One was developed from a multiple regression analysis of results from a full size tank:

$$Ce = 18.2 + Kepa\ (Qi/A) - 0.0033\ MLSS \qquad (37)$$

where: Kepa = EPA constant

As the above equation had a low coefficient of determination, it was concluded from the study that the effluent suspended solids concentration cannot be estimated from parameters commonly used to operate wastewater treatment plants.

Tuntoolavest, *et al.* (1980) carried out a fractional factorial experiment on a pilot plant facility which was supplied with a synthetic wastewater. MLSS concentration, air flow rate, and recycle rate were studied to determine how flocculation in the aeration tank influenced suspended solids removal. The results supported Parker's contention that the environment in the aeration tank is floc disruptive (Parker *et al.*, 1972). Secondly, variables were found to be interactive; that is, the magnitude of the effect of one variable depends on the levels of the other variables. The single most important variable or interaction was found to be the air flow rate/MLSS concentration interaction, with 87 percent of the variability in settler effluent suspended solids concentration accounted for by changes in this interaction alone.

The effect of two settler variables, overflow rate and detention time, was also examined by Tuntoolavest *et al.* (1980). Columns of varying height were used to vary detention time for a given overflow rate. The simplest equation which expresses the effect of these two variables and MLSS concentration is:

$$Ce = 0.01345 \cdot MLSS - 0.0024 \cdot MLSS \cdot (V/Qi) + 0.00389 \cdot MLSS \cdot (Qi/A) - 6.51 \qquad (38)$$

The coefficient of multiple determination for this equation is 0.82. In other words, 82 percent of the variability in effluent suspended solids concentration could be explained by the combined influence of the above variables. A number of other equations of varying complexity were developed from a step-wise regression analysis.

Polta and Stulc (1979) examined the effect of automatic sludge blanket control in influencing overflow and underflow solids concentrations from a gravity thickener. Variables monitored besides blanket height (h_B) included: inflow rate (Qi), inflow solids concentration (C_i) and temperature. Blanket height and inflow solids concentration were found to be the most significant variables. The following two equations employing these two variables were developed from a linear regression analysis:

<u>Manual control</u>:

$$Ce = -508 - 520\, h_B + 0.761 C_{in} \qquad (39)$$

where: C_{in} = suspended solids concentration in inflow to thickener (mg/L)

h_B = blanket height (m)

Automatic control:

$$Ce = -302 - 660\ h_B + 0.453C_{in} \tag{40}$$

The coefficient of multiple determination for the first equation (manual control) was 0.72 with 0.50 for the second (automatic control).

2.6.3 Clarification Variables. The models described in the preceding paragraphs and summarized in Table 2.11 employ a total of five variables. An understanding of the relative importance of the variables was gained by performing a sensitivity analysis. For the initial conditions of Table 2.12a, reflecting "average" design and operating conditions, the percentage change in effluent suspended solids concentration was determined for a given change in the value of the variable used in the models. The responses for a +20 percent change in variable level are presented in Table 2.12b with results for a -20 percent change presented in Table 2.12c.

A surprising result of the sensitivity anlaysis is the relative importance of MLSS concentration for all of the models examined. The influence of this variable is most pronounced for the Tuntoolavest et al. (1980) and Polta/Stulc (1978) models and, with the exception of

TABLE 2.11: **Operating and Design Parameters for the Activated Sludge Settler which are used in Modelling Clarification**

Model	Variable					Comments
	Qi/A	V/Q	MLSS	h_B	Qr/A	
Busby and Andrews (1975)	✓		✓		✓	Based on Pflanz's full-scale study
Ghobrial (1978)	✓	✓	✓	✓	✓	Pflanz's work and columns
Agnew (1972)	✓		✓			Full-scale clarifier
Tuntoolavest et al. (1980)	✓	✓	✓			Pilot plant and columns
Polta and Stulc (1979)			✓	✓		Full-scale thickener

TABLE 2.12: Sensitivity Analysis

a) Initial Conditions for Average Design and Operation

Variable	Value	Comment
MLSS	2250 mg/L	Average for range, Table 10-4 (Metcalf and Eddy, 1979)
Qr/Q_i	30%	From Table 2.8 of this work
Qi/A	1.017 $m^3/m^2 \cdot h$	Mid-range, Table 7.3 (U.S. EPA, 1975)
h = SWD	4.1 m	Mid-range, Table 7.3 (U.S. EPA, 1975)
h/(Qi/A)	4 h	–
h_B	1.5 m	Mid-range, Figure 25 (Polta and Stulc, 1979)

b) Response in Effluent Suspended Solids Concentration for a 20 Percent Increase in Variable Level

Model	Change Effluent SS Concentration				
	Qi/A (+20%)	V/Q (+20%)	MLSS (+20%)	Qr/A (+20%)	h_B (+20%)
Busby and Andrews (1975)	+15%	–	+20%	+5%	–
Ghobrial (1978)	+15%	17%	+20%	+5%	+20%
Agnew (1972)	+9%	–	-8%		–
Tuntoolavest et al. (1980)	+17%	43%	+59%	.	–
Polta and Stulc (1979)			+84%	–	-39%

[illegible] Response in Effluent Suspended Solids Concentration for a [illegible] Variable Level

Model	Change Effluent SS Concentration				
	Qi/A (-20%)	V/Q (-20%)	MLSS (-20%)	Qr/A (-20%)	h_B (-20%)
[illegible]usby and Andre[illegible]	16%	–	-20%	-5%	–
[illegible]hobrial (19[illegible])	15%	[illegible]25%	20%	5%	-20%
[illegible]gnew (1972)	-8%	.	+8%	.	–
[illegible] et al. (1[illegible])	7[illegible]	[illegible]4[illegible]	32%		...
[illegible] (1[illegible])			84%		+39%

the Agnew equation, an increase in MLSS concentration is accompanied by an increase in effluent suspended solids concentration.

Support for the importance of MLSS concentration as a clarification parameter also comes from the work of Tebbutt and Christoulas (1975) on primary sedimentation. They concluded that the effect of the influent SS concentration on solids removal was considerably more important than was the overflow rate. The effect of influent SS concentration was attributed to its influence on feed density and the degree of flocculation within the settler. The Purdue researchers (Tuntoolavest _et al_., 1980) commented that, as a proportion of the mixed liquor suspension consisted of primary particles, increasing the MLSS concentration simply increases the concentration of non-settleable particles introduced into the settler.

Compared to MLSS concentration, overflow rate was a less important variable. For the Ghobrial study, the influences of overflow rate and detention time were the same. Tuntoolavest _et al_. (1980) concluded that the two variables affected clarification about equally. Tebbutt (1969) observed that the removal of solids from crude sewage was virtually independent of the surface overflow rate in the traditional design range. Fitch (1957) examined the effect of detention time and overflow rate on solids removal in a settling column. The results shown in Figure [illegible].10 indicate that, as the curves for the values of percent removal are more [illegible] horizontal than vertical [illegible]

Page 77 was removed due to copyright restrictions. The page contained Figure 2.10, obtained from a paper by Fitch (1957), showing the experimentally obtained relationship between average solids removal, detention time, and overflow rate.

For the two models incorporating blanket height, the sensitivity analysis revealed that this parameter is relatively important. However, the response in the sensitivity analysis is seen to be in the opposite direction for the two models. It is possible that for a particular settler an optimum blanket height exists such that raising or lowering the height away from the optimum will result in deterioration of effluent quality.

Besides recycle rate which was determined to be unimportant for all of the models analyzed, other variables identified as possibly influencing clarification include: temperature, wind and rake speed. Mazurczyk <u>et</u> <u>al</u>. (1980) reported that the incoming flow could rise toward the surface of the settler provided that this flow was 0.7°C warmer than the tank contents. Pflanz (1969) observed that a significant deterioration in removal efficiency resulted for a settler operated in the 2 to 3°C range as compared to operation in the 13 to 15°C range. As well, it was noted that a wind velocity of 6 m/s produced a suspended solids concentration of 113 mg/L in the lee-side weir while the weather-side concentration was 13 mg/L. Wind-introduced seiches (or level oscillations) at both the tank surface and blanket interface, exert a possible detrimental influence on the settler (Crosby and Bender, 1980). Finally, Crosby and Bender (1980) determined that removal efficiency in activated sludge settlers is adversely [illegible] (Table [illegible]) commonly used in practice.

2.7 Summary

The cost and efficiency of the activated sludge system are profoundly influenced by the efficiency of solids removal. A majority of the BOD_5 discharged from the conventional activated sludge system results from the loss of solids over the weirs of the settler. A large number of conventional activated sludge plants produce an effluent containing a concentration of suspended solids greater than 30 mg/L. Requirements for very low effluent suspended solids concentrations have generally been met by adding granular filters or polishing lagoons to the activated sludge system (Bush and Irvine, 1980). The processing of solids wasted from the underflow of the secondary settler, along with primary solids, accounts for 50 percent of the total treatment operating costs.

A number of factors influence the performance of the settler in capturing and concentrating bio-solids and these are summarized in Table 2.13. Suspension characteristics are determined by the composition and flow characteristics of the wastewater and the environment in the aeration tank. Settling and compaction rates are governed by the relative proportion of filamentous to zoogleal bacteria in the floc. The relative rate of growth of the two types of bacteria is related to the level of dissolved oxygen, the rate of substrate loading and possibly the aeration tank configuration.

Flocculation occurring in the aeration tank and settler depends on naturally produced exo-polymers which bridge the distance between adjacent negatively charged cells. The rate of polymer production is related to the growth phase of the microorganisms and hence

TABLE 2.13: Summary of Factors Affecting Clarification and Thickening in Circular Settlers

WASTEWATER CHARACTERISTICS/ATMOSPHERIC CONDITIONS:
Established by:
a) Wastewater composition and physical characteristics:
• organic strength, pH, temperature and ionic concentration
b) Wastewater flow rate:
• rates; duration and intensity
c) Atmospheric conditions:
• wind velocity and direction; ambient air temperature

SUSPENSION PROPERTIES:	
(Given: wastewater characteristics/atmospheric conditions)	
Established by:	**Controlled by:**
a) Concentration of exo-polymer	SRT
b) Concentration of non-settleable dispersed solids.	Air flow rate and type of diffusers
	Recycle rate
c) Total solids concentration (MLSS)	Hydraulic retention time of aeration basin
d) Physiological state of microorganisms	MLSS concentration
• growth rate	Flow mode (plug/complete mix)
• proportion of filamentous to zoogleal bacteria	

Continued..../

TABLE 2.13 (Cont'd): **Summary of Factors Affecting Clarification and Thickening in Circular Settlers**

SETTLER PERFORMANCE:	
(Given: wastewater characteristics/atmospheric conditions and suspension properties)	
<u>Established by:</u>	<u>Controlled by:</u>
a) Tank configuration	Settler diameter and depth
b) Rate of loading	Diameter and depth
c) Configuration and effectiveness of sludge collection equipment	Weir location and loading
	Overflow rate
	Detention time
d) Settler operation	Recycle rate/blanket height
	Rake speed

the solids retention time (SRT) of the system. In the aeration tanks, the air flow rate necessary to maintain a minimum dissolved oxygen concentration results in a level of turbulence which limits floc size and produces non-settleable particles.

The efficiency of a clarifier in removing and concentrating solids will depend on the nature of the solids introduced and the settler characteristics as established by tank configuration, the effectiveness of the sludge removal equipment, methods of operation, and the rate of loading. Empirical loading rates, most commonly overflow rate and detention period, have traditionally been used to establish acceptable removal efficiencies. Settler performance is most commonly evaluated on the basis of hydraulic efficiency as measured by tracer techniques. Research indicates that these methods are inadequate as they are not based on the clarification and thickening properties of the mixed liquor. Furthermore, they underestimate flocculation as a removal mechanism. The existence of vertical currents within the settler and transients within the plant result in hydraulic conditions which are far from ideal. The currents increase mixing. Transients arising from variations in influent and the operations within the plant can result in a deterioration of effluent quality.

An alternative to empirical loading criteria is provided by the solids flux theory. Evidence suggests that the accuracy of the solids flux method in predicting thickening performance will be in the ±20 percent range. In contrast, clarification remains poorly understood. In particular, it appears that the role of MLSS concentration and sludge blanket depth have been underestimated in comparison to overflow rate and detention time. The MLSS concentration determines the concentration of non-settleable primary particles entering the settler, the degree of flocculation and the magnitude of density currents within the settler. The sludge blanket provides a quiescent zone for sedimentation and, if the blanket is above the feedwell outlet, enhances flocculation.

While the literature provides some insight into the effects of process and design variables on clarification, no clear conclusions can be reached concerning the relative magnitudes of their effects. Secondly, the dynamics of solids removal in the settler remains a relatively unexplored area.

CHAPTER 3

RESEARCH OBJECTIVES AND EXPERIMENTAL PLAN

3.1 Research Objectives

For the activated sludge system, the concentration of suspended solids in the effluent from the final settler is the major determinant of process efficiency. At present, there is insufficient knowledge to enable clarification efficiency to be either predicted or controlled. In particular, as revealed in the literature, confusion exists as to the importance of numerous design parameters and operating strategies. The influence of time-varying loadings on clarification has not been fully investigated.

As indicated by Table 2.13, a large number of variables were identified as potentially influencing clarification efficiency. In order to develop an experimental program which would contribute to maximizing process efficiency, attention was focussed on those variables which could be controlled by selecting appropriate design levels or instituting control loops. Parameters, such as the temperature of the raw sewage entering the plant, are of interest in order to predict process performance but do not enable removal efficiency to be maximized as their level cannot be selected at the design stage or economically controlled. Based on available experimental equipment, a total of seven "design and operating" variables were selected for investigation. A list of these variables is contained in Table 3.1. The table also indicates previous studies which make reference to the variable and the mechanisms postulated to account for the observed

TABLE 3.1: Variables Selected for Investigation

Variable	Researcher(s)	Postulated Mechanism(s)
1) MLSS	Pflanz (1969) Tuntoolavest et al. (1980)	• Changes solids loading to clarifier • Changes concentration of dispersed solids entering clarifier
2) Feed flow rate	Hazen (1904) Camp (1953)	• Establishes settling velocity of slowest particle to be completely removed
3) Recycle rate	Tuntoolavest et al. (1980)	• Influences flocculation and particle shear in the aeration basin
4) Speed of sludge scraper	Crosby and Bender (1980)	• Stirs and resuspends solids from sludge blanket
5) Air flow rate	Parker et al. (1972) Tuntoolavest et al. (1980)	• Influences floc shear changing number of dispersed solids entering settler
6) Feedwell depth	Keinath et al. (1976b) Hill (1974)	• Influences flocculation and entrapment in sludge blanket
7) Sidewater depth	Fitch (1969) Parker (1982)	• Influences flocculation

changes in clarifier efficiency.

Having selected variables which could be investigated using available equipment, a series of experiments were designed to study both the steady-state and dynamic performance of clarification. More specifically, the objectives of the research program were:

1) To evaluate the response of the clarifier to time-varying changes in hydraulic flows and operating conditions,
2) To determine which of the seven design and operating variables significantly influence the efficiency of suspended solids removal during steady-state operation,
3) To quantify the results using statistical design and analysis techniques,
4) To compare the observed results with those obtained from previously published research, and
5) To establish the implications of both the dynamic and steady-state results to activated sludge design and operation.

3.2 Experimental Plan

General Plan: In order to meet the outlined objectives, experiments were performed on a test clarifier modified to receive mixed liquor from Pilot Plant #1 at Environment Canada's Wastewater Technology Centre in Burlington, Ontario. The [illegible] were instrumented with a computer providing for automatic data logging and control. Information from the sensors was supplemented with data collected manually from settling tests and gravimetric solids analyses.

The experimental program was carried out in three phases. Initially, preliminary information was collected to establish operating and monitoring procedures and to plan the subsequent phases. The second phase was devoted to examining the dynamics of clarification using step tests. The influence of design and operating variables on clarifier efficiency was evaluated using a series of factorial experiments. Following analyses of results and comparison with other research, implications were drawn regarding design and operating practices.

The experimental plan which was followed in each of the three phases is discussed in more detail in the following sections.

Phase I - Preliminary

Initially, work was carried out with the aim of making the system fully operational, collecting some preliminary hydraulic information and calibrating instrumentation. Prior to seeding the plant, software was modified and a data base for on-line information created.

To seed the system, waste activated sludge was trucked from the Burlington Skyway Treatment Plant and added to the aeration basin. With liquid in the process, the PI flow controllers were tuned. A solids wasting schedule was instituted, establishing a solids retention time in the three to five-day range. On-line sensors were calibrated and a manual testing routine implemented.

Phase II - Dynamics

Secondary clarifiers are designed on the assumption that the process operates at steady-state. In reality, this is not true. The activated sludge system is subject to a variety of time-varying disturbances including uncontrolled variations in flow, sewage composition, and weather conditions. Variations in influent flow rate with time have a significant impact on process performance. They are induced by diurnal fluctuations in domestic water demand, the infiltration of water into sewers during storms, and industrial discharges which change from shift to shift. Flow variations are also induced within treatment plants as influent, wasting and recycle pumps are turned on or off.

To obtain a complete picture of the solids removal process, it is desirable to establish the response of the clarifier to changes in loading and, in particular, to changes in hydraulic loading. Such information is useful in assessing and modifying design criteria and in developing operating strategies. For the experimental program, a knowledge of the dynamic [illegible] of the [illegible] was required to plan the steady state portion of the [illegible]. The length of time required to complete [illegible] steady state run and the amount of [illegible] required to change from one [illegible] depends [illegible] how [illegible] the [illegible]

The dynamics of a process can be identified by modelling from first principles or experimentally by introducing disturbances into the system and measuring the response. The lack of agreement in the literature regarding the mechanisms of clarification dictated that an experimental approach be used to investigate the dynamics of clarification. Disturbances introduced into the system can be sine waves, ramps, steps or pulses. Step testing was identified as the most direct method, yielding results which are easy to analyze (Luyben, 1973).

Step changes were made to the rate of flow into the clarifier by changing the setpoint in the digital control loop of the feed-flow pump. A step size of 40 L/min was used, representing a 40 percent change in the hydraulic loading rate. During the test, the feed flow rate to the settler and the suspended solids concentration in the effluent and underflow were monitored using on-line sensors. The data collected from the sensors was stored in disc files on the HP-1000 for later plotting. After steady-state conditions were reached following a step increase in flow rate, a step decrease of equal magnitude was introduced.

A major objective of the step testing was to identify the dominant time constant (τ) of the settler with respect to clarification. The dominant time constant establishes the "time-scale" of the process and is measured in order to design control algorithms and to select sampling times (Edgar, 1979).

For a first order system, or for a system with a response which [illegible] be approximated [illegible] a first-order linear differential equation - the rate at which the process changes from one steady-state condition to another depends on the time constant. For the test clarifier, the time constant was estimated by measuring from the plots of the response the length of time required for the process to reach 39, 63 and 78 percent of the total change in steady-state values. The length of times corresponded to a specified fraction of the dominant time constant (39 percent, 0.5 τ; 63 percent, 1.0 τ; and 78 percent, 1.5 τ). A single estimate for the time constant was then obtained by averaging the three values.

The order of the dynamics was estimated by using a linear least squares technique. The method, described in detail by Astrom and Eykhoff (1971), is based on fitting models to the observed data. The models were obtained by writing, as discrete-time equations, the differential equations postulated to represent the dynamics of the system. The order of the process was therefore tested by increasing the order, starting at a first-order model, and using a sequential F-test to determine if the improved fit was significant. The method was employed for the clarifier study because it was relatively simple to employ and some of the software needed to carry out the analysis already existed.

One major disadvantage of the method is that misleading results can arise if the residuals [illegible] the differences between observed and predicted values [illegible] are correlated (Astrom and Eykhoff, 1971). In order to minimize this potential problem, care was taken to select data points [illegible] in the analysis that were separated [illegible] [illegible] The length [illegible] was [illegible] results from the repeated least squares analyses of data from one of the runs. The interval in each analysis was increased until the method provided consistent results with respect to the order of the system.

Phase III - Steady-State

The most important questions to be addressed by Phase III of the research included:

- Do any of the seven variables influence effluent quality significantly?
- Which variables are the most important?
- What is the nature of the relationship between the important variables and effluent quality?

A number of difficulties had to be overcome if satisfactory answers were to be provided to these questions. For instance, the number of variables included in the research was large, requiring potentially a large experimental program. Not all of the variables which could influence effluent suspended solids concentration could be controlled or, in some cases, be measured. Changes in the strength and direction of the wind, liquid temperatures, and wastewater strength and composition were sources of experimental variability or noise. Some evidence from previous clarifier research indicated that relationships concerning removal efficiency were complex - that is, the influence of variables was neither linear nor additive.

A rigorous approach to these and other research difficulties has been [illegible] by Box and co-workers using statistical design techniques (Box et al., 1978; Box and Hunter, 1961). In particular, an experimental design procedure was selected which required that each of the variables under investigation be maintained at one of two levels during the course of an experimental run. Such "two-level" designs could be carried out in a sequence of moderately-sized sets of experiments requiring relatively few runs per set (Box et al., 1978). A major advantage of such an experimental strategy was that information gained in one set directly influenced the choice of experiments in the next. Randomization of the run order within a set of experiments ensured that there was a sound basis for deducing causality (Box et al., 1978).

For seven variables, a sequence of designs requiring eight runs per design was identified as an efficient arrangement for screening variables (Box and Hunter, 1961). A complete factorial involving seven variables would require a total of 2^7 or 128 runs. Eight runs represented 1/16 of the total number, therefore an eight-run design was designated as a 2^{7-4} ($1/16 \times 2^7$) design. Each of the variables was either set at a high ("+") or a low ("-") level for a particular run. For a set of eight runs, each of the variables was maintained at a "+" level for four runs and a "-" level for four runs. As considerable physical effort was required to raise and lower the weir in the test clarifier, each design was blocked about sidewater depth. That is, the four runs at which sidewater depth was at a "-" level were conducted one after another, the weir position lowered and the four [illegible] then carried out.

The original 2^{7-4} design was further modified by adding a number of runs for which the variables were set at centre-point levels midway between the "+" and "-" levels. The centre-point designation was "0". As the levels of MLSS concentration were controlled during the factorial by shifting the points at which influent and recycle sludge were added to the aeration tank, the additional "0" runs were included to re-equilibrate the system following the changes in feed or recycle distribution. As well, the centre-point runs identified time trends in the data arising from changes in influent composition or shifts in other uncontrolled variables.

Variables were assigned the levels listed in Table 3.2. The feed flow and recycle rates were selected so that the corresponding overflow rates covered the range of values contained in existing design guidelines. Given the levels listed in Table 3.2, the overflow rate ranged from a low of 0.68 $m^3/m^2 \cdot h$ (for a "-" feed flow rate and a "+" feed flow rate) to a high of 2.03 $m^3/m^2 \cdot h$ (for a "+" feed flow rate and a "-" recycle rate). The U.S. Environmental Protection Agency (1975) recommends that for the conventional activated sludge system the average overflow rate be in the range of from 0.70 to 1.36 $m^3/m^2 \cdot h$ with peak flow rates in the 1.70 to 2.04 $m^3/m^2 \cdot h$ range. Similarly, rake speeds varied from 2 to 8 rev/h while the recommended range was from 2 to 4 rev/h (Task Committee ASCE, 1979).

For the other variables used in the two-level factorial, levels were selected on the basis of preliminary experimentation so as to produce as large a change as possible from the "-" to the "+" levels. The magnitude of the change in levels was limited by the size

TABLE 3.2 Levels for the Screening Factorial

Abbreviation	Variable	Level "-"	Level "0"	Level "+"
ML	MLSS	Qfeed: "C" Qras: "A"	Qfeed: "A" Qras: "A"	Qfeed: "A" Qras: "C"
FD	Feedwell Depth	0.81 m	1.27 m	1.72 m
RS	Rake Speed	2 rev/h	5 rev/h	8 rev/h
FF	Feed Flow	100 L/min	120 L/min	140 L/min
UF	Underflow	20 L/min	40 L/min	60 L/min
SD	Sidewater Depth	1.48 m	N/A	1.94 m
AF	Air Flow	51.5 L/s	59.0 L/s	66.5 L/s

of the existing equipment and the need to maintain a viable treatment system.

A list of the levels at which each of the variables was maintained was the design matrix. The design matrix for the first fraction (also known as the principal fraction) is given in Table 3.3. Upon completion of the runs for the principal fraction, any ambiguities were resolved by adding complementary eight-run sets. Designs for these sets were generated by switching the signs on one or all of the columns of the principal fraction (Box and Hunter, 1961). Combining all the data from the sequence of factorials, conclusions could then be drawn regarding the importance of the variables and the nature of their influence on effluent clarity.

TABLE 3.3 Design Matrix for Principal Fraction

Run Number	Variable Level*						
	ML	FD	RS	FF	UF	SD	AF
1	−	−	−	+	+	+	−
2	+	−	−	−	−	+	+
3	−	+	−	−	+	−	+
4	+	+	−	+	−	−	−
5	−	−	+	+	−	−	+
6	+	−	+	−	+	−	−
7	−	+	+	−	−	+	−
8	+	+	+	+	+	+	+

* Table 3.2 identified the variables associated with the listed abbreviations.

CHAPTER 4

MATERIALS AND METHODS

The research program required that step and ramp inputs be applied to the clarifier and that seven variables be controlled at pre-selected levels. Modifications were therefore made to a test clarifier. The clarifier was positioned next to a large pilot-scale activated sludge system which provided mixed liquor. Sensors interfaced to a real-time computer continuously monitored performance.

The equipment and procedures used in the research are described in more detail in the following sections.

4.1 Test Clarifier

The test clarifier was a 2.4 m diameter settler modified to enable sidewater depth, feedwell depth and rake speed to be varied.

Provision for control of sidewater depth was made by constructing a moveable weir. The existing weir was cut away from the tank wall, a plate welded on the back, and the resulting box supported by hangers and bolts from the top of the tank. A flexible hose connected the outlet from the weir box to the tank outlet.

Extensions were designed for the inlet feedwell so that feedwell depth could be increased in increments of 15 cm. A variable speed motor was used to drive the sludge scraper arms. Rake speed could be controlled from 0.0 to 12.0 rev/h.

Additional details of the settler are presented in Figure 4.1. Prior to the previously described modifications, Heinke *et al*. (1977) determined the hydraulic characteristics of the test settler for various overflow-rates.

4.2 Pilot Plant

An activated sludge pilot plant supplied the test clarifier with mixed liquor. The plant, located at Environment Canada's Wastewater Technology Centre, in Burlington, Ontario, consisted of a Smith and Loveless extended aeration plant and pumphouse.

To ensure that results from the research had the widest applicability, the pilot plant was operated in the conventional mode. Baffles were installed in existing wall supports, proportioning the aeration tank into three compartments of equal size. Mixing characteristics were therefore changed from complete mix to three CSTRs in series. Flow rates were also selected so that the hydraulic retention time of aeration tanks was in the range typical for conventional activated sludge systems. Operating characteristics of the pilot plant were as shown in Table 4.1.

The pilot plant was seeded with waste activated sludge trucked from the Burlington Skyway Sewage Treatment Plant. A low sludge age was selected in an attempt to avoid nitrification in the aeration tank. Subsequent denitrification in the settler would result in floating sludge, increasing the solids concentration in the settler overflow. Direct wasting of mixed liquor from the aeration tanks was used to control the SRT.

Initially, degritted sewage pumped from the Burlington Skyway Sewage Treatment Plant was added directly to the aeration

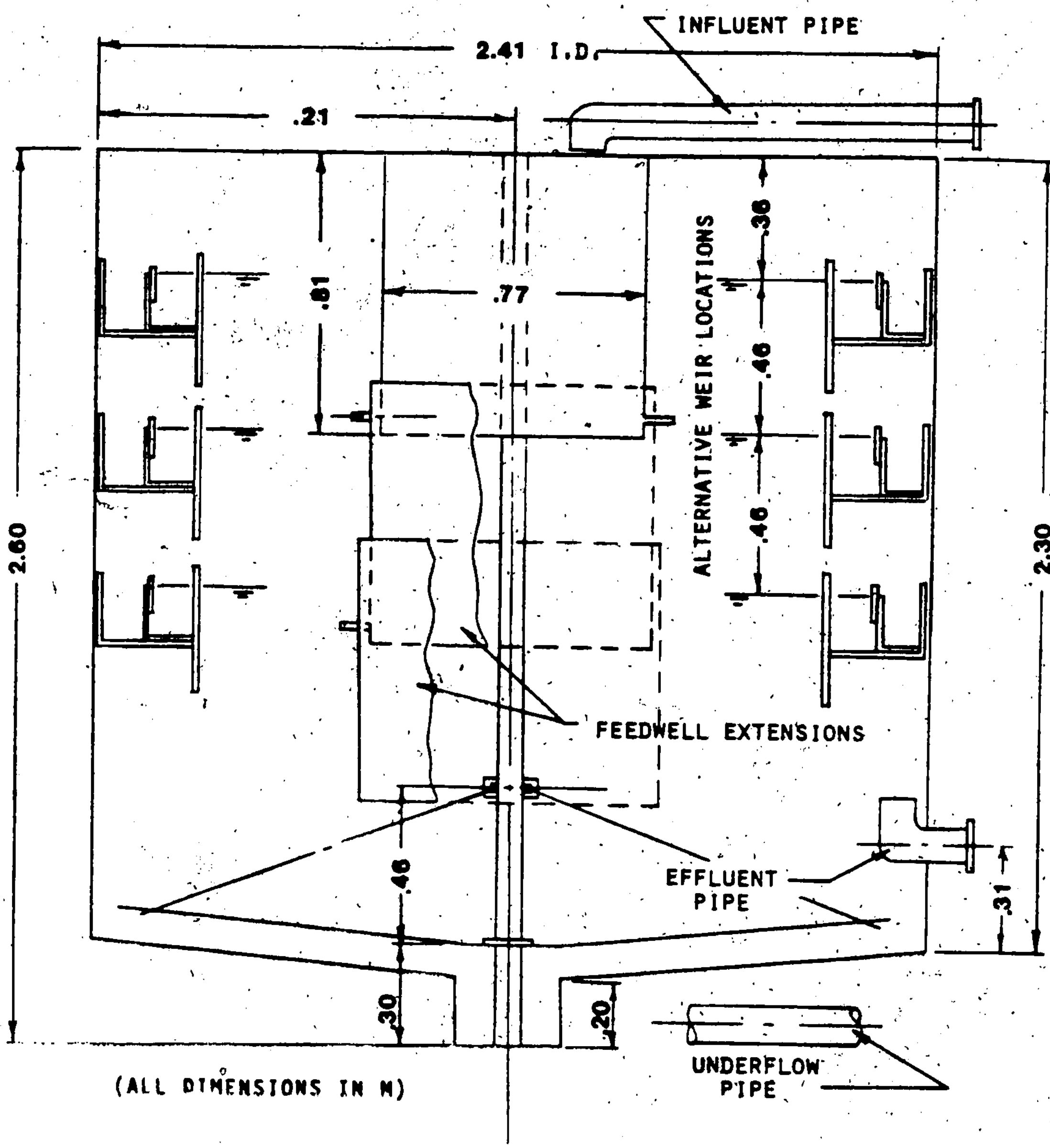


Figure 4.1 Cross-Section of Test Clarifier.

TABLE 4.1: Design and Operational Data for Pilot Plant #1

Description	Level
Influent flow	262 m^3/d
Recycle flow (100%)	262 m^3/d
Aeration tank volume	66.3 m^3
Hydraulic retention time	6.1 h
MLSS concentration	2000 mg/L
Setpoint SRT (sludge age)	3 d
Diffusers	Keen "Aircomb" (2.5 m submergence)

tanks. That is, there was no primary sedimentation. As a consequence, during plant shakedown, control valves became clogged with stringy material. To overcome the problem, the influent was passed through a vertical screen before being added to the aeration tanks. The screen, manufactured by Dorr-Oliver, contained a wedge-wire surface with 900 μm openings. Approximately 10 to 15 percent of the influent solids, including the coarse material which clogged the valves, was removed by the screen. Modification and installation details have been given by Speirs (1981). Kronis and Tonelli (1980) have evaluated the effectiveness of such devices in screening combined sewage.

To exercise control over the rate at which mixed liquor was transferred from the aeration basin of the activated sludge plant to the test settler, a pump with automatic flow control was installed. A potential disadvantage of this arrangement was that the pump might shear the flocs contained in the mixed liquor. The effect of air flow rate on floc shear would then be masked. A progressive-cavity pump was selected to reduce the potential for floc shear. The pump consists of a helical rotor placed in a housing or stator which itself has an internal thread. Cavities are defined by the rotor and stator. As the rotor turns, axial flow develops with the enclosed cavities containing entrapped fluid moving continuously from inlet to outlet. Low inernal velocities are developed by progressive-cavity pumps (Czarnecki and Lippincott, 1976) Floc shear was accordingly minimized during the transfer of mixed liquor.

A feedback loop is created in the activated sludge system by the recycle of underflow solids from the settler to the aeration tank. Due to the strong interaction between the two units, the process is difficult to understand and model (Olsson, 1976). The test clarifier and pilot plant were configured in order to minimize the feedback loop between the test clarifier and the aeration tanks. Adding the test clarifier to Pilot Plant #1 resulted in a system with three aeration compartments and two final settlers - the previously described test clarifier and a "system" clarifier consisting of a hopper-bottomed tank built integrally to the aeration tanks. As shown in Figure 4.2, the underflow lines from the "system" and test clarifiers were joined, forming a single "system recycle" line. During the experimental program, the flow in this line was maintained at a constant rate by controlling the system recycle pump (Pump #1 in Figure 4.2). The underflow rate from the test clarifier, controlled by Pump #2 (Figure 4.2) was varied in accordance with the experimental design. With this arrangement, the rate of flow of sludge into the aeration basin from the two settlers was held constant while variations were made in the underflow from the test clarifier. For the combinations of feed flow rate and recycle rate applied to the test clarifier during the factorial experiments, Table [illegible] lists the corresponding flow rates in the other lines in the plant.

With the aeration basin baffled into three compartments, piping was configured to allow MLSS to be manipulated. When operating normally, the pilot plant [illegible] and recycle were added to the head end of the [illegible] For [illegible] MLSS [illegible], the

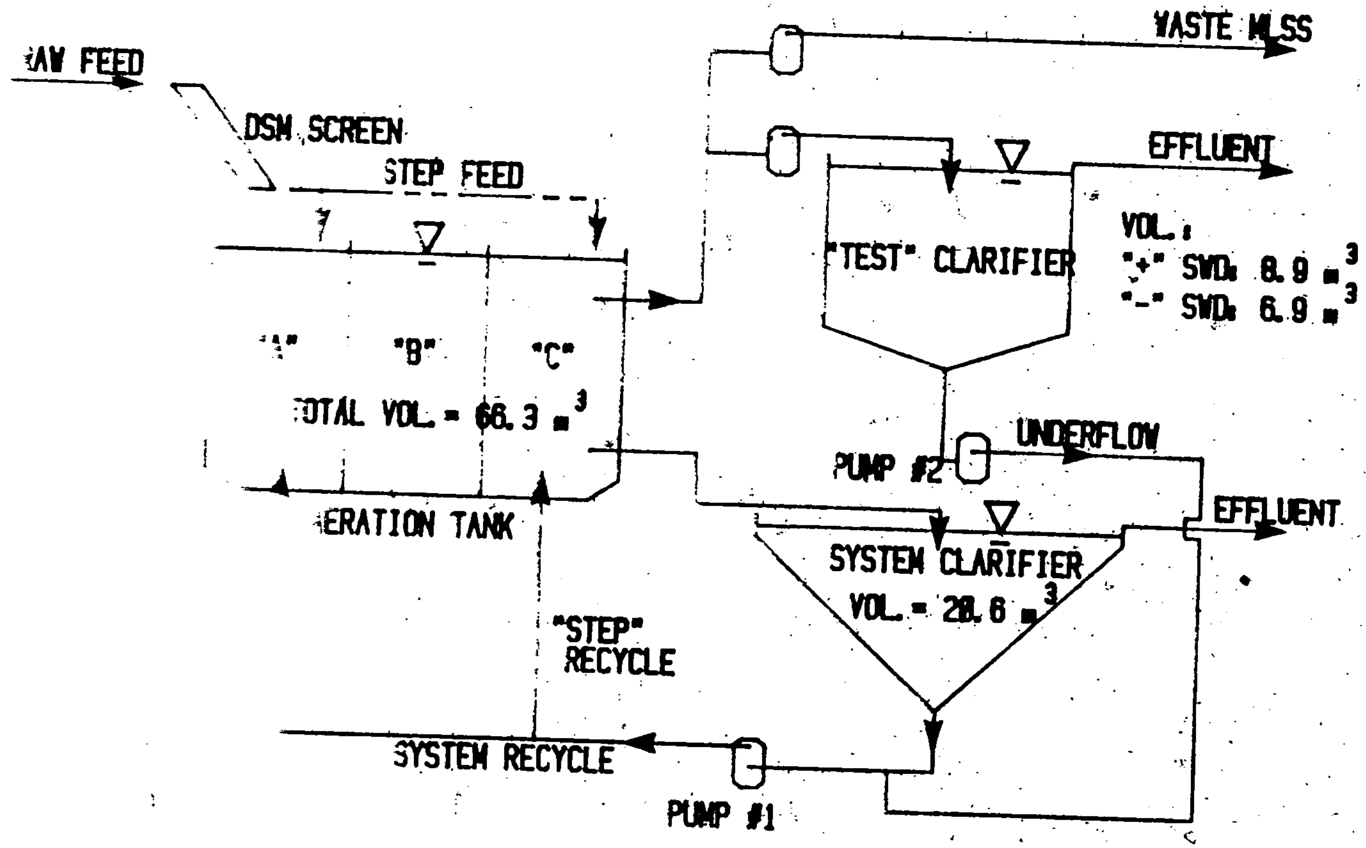


Figure 4.2 Research Facility Flowsheet.

TABLE 4.2: Flow Rates in Test Clarifier and Pilot Plant During Steady-State Expermentation (in L/min).

Test Clarifier		Raw Sewage	System Recycle	MLSS Wastage*	System Clarifier	
FF	UF				FF	UF
140.(+)	60.(+)	140.	140.	15.	125.	80.
140.(+)	20.(-)	140.	140.	15.	125.	120.
100.(-)	60.(+)	140.	140.	15.	165.	80.
100.(-)	20.(-)	140.	140.	15.	165.	120.

* Based on a three-day SRT.

influent was added to the last of the three compartments ("C"), with the recycle sludge line emptying into the first compartment. The "+" MLSS level was obtained by reversing the arrangement, i.e., recycle into compartment "C" and inflow to "A".

4.3 On-line Monitoring and Control

a) Computer Hardware: Data acquisition and process control were implemented using on-line sensors and a real time computer. The computer was a Hewlett-Packard 1000 21MX-E-Series with the RTE-IVB operating system. It had 160 k-words of memory with 120 M-bytes of disc storage. An HP-2240 front-end microprocessor interfaced the sensors to the minicomputer and provided signal conditioning and optical isolation. Computer peripherals included five CRTs, a magnetic tape station, two plotters, and a line printer.

b) Sensors: A variety of sensors were interfaced to the HP-1000. A brief description of each of the sensors is given in Table 4.3 with locations given in Figure 4.3. The turbidimeters, solids meters, and dissolved oxygen sensors had been previously used on another pilot plant at the Wastewater Technology Centre. An evaluation of their performance was conducted by Stephenson *et al.* (1981).

A Partech ASLD 2000 Mark V sludge blanket monitor was mounted on the test clarifier. As supplied, the unit measured sludge blanket height by unwinding a probe from a cable drum until the solids-liquid interface was encountered. The monitor was modified by connecting digital output signal lines from the front-end of the

TABLE 4.3: Process Equipment for Clarifier Research (Refer to Figure 4.3)

Designation	Function	Description
	PUMPS	
	Pilot Plant #1 recycle	2 in dia WEMCO
	Test clarifier ML feed	2L8 Moyno located outside of pump-house
	Test clarifier underflow	2 in dia WEMCO with throttling valve
	Wasting control for Pilot Plant #1	2L4 Moyno
	FLOW METERS	
F1	Pilot Plant #1 inflow	1 in dia Brooks mag meter
F2	Pilot Plant #1 recycle	2 in dia Brooks mag meter
F3	Test clarifier ML feed	1 in dia Brooks mag meter
F4	Test clarifier underflow	1 in dia Brooks mag meter
F5	Wasting control for Pilot Plant #1	1 in dia Brooks mag meter
F6	Total air to aeration tanks	Foxboro Vortex meter
	SENSORS	
S1	Solids probe for #3 aeration tank	Clam Monitek
S2	Solids probe for test clarifier underflow	Clam Monitek
S3	Turbidimeter for test clarifier effluent	Lisle-Metrix
S4	Turbidimeter for Pilot Plant #1 effluent	Lisle-Metrix
D1	DO probe for aeration tank "A"	pHOX
D2	DO probe for aeration tank "B"	pHOX
D3	DO probe for aeration tank "C"	pHOX
D4	"Float" DO probe for checking	Zullig
T1	Temperature probe for aeration tank "C"	RTD detector
T2	Temperature probe for test clarifier	RTD detector
SB1	Sludge blanket monitor	Partech ASLD 2000

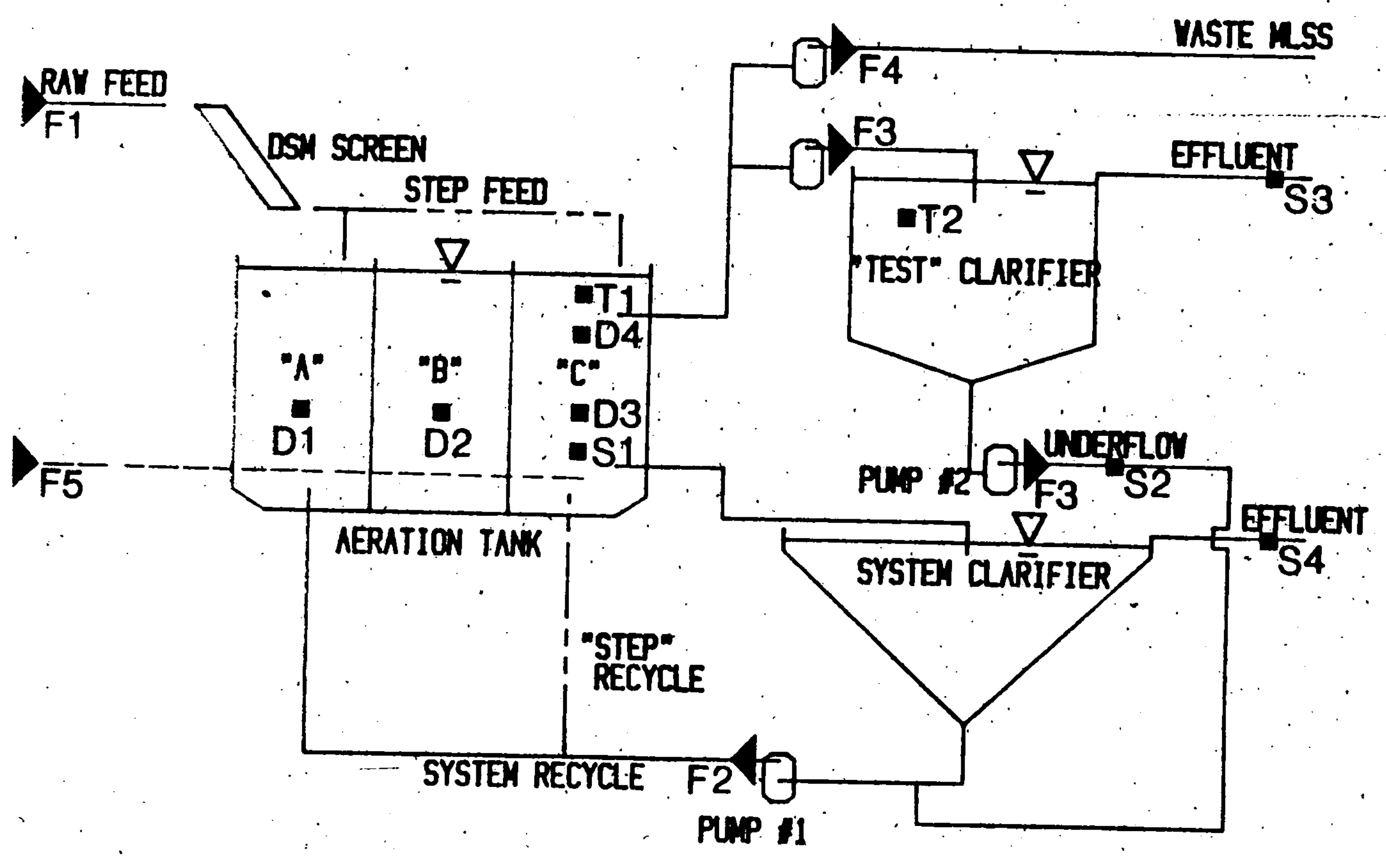


Figure 4.3 Schematic Showing Location of Sensors and Flow Meters.

computer to "make-and-break" relay switches located in the electronic control box of the unit. The motor controlling the cable drum could then be turned on or off and the probe raised or lowered. The modifications to the sludge blanket detector permitted solids profiles - height versus concentration data - to be collected automatically.

The transmitters with local indicators for the sensors were located in the pumphouse. They were linked to a terminal box which in turn was connected via two coaxial cables to the front-end of the computer located in the main building at the Wastewater Technology Centre. Digital displays for all process flows were located in a panel box in the pumphouse, allowing the pilot plant operator to monitor the process conveniently.

Sensors were calibrated by comparing sensor outputs to the results of measurements using standard procedures. For instance, for the solids sensor, over the range of the instrument, paired data was collected consisting of gravimetrically determined suspended solids concentrations and corresponding integer numbers representing the sensor signal as registered by the computer. Regression techniques were used to develop an empirical relationship between the standard and corresponding sensor output. The resulting equation (or equations) was entered into the computer which then converted the sensor signal to the appropriate engineering units. Additional details of the regression technique are given by Speirs et al. (1983).

c) Final Control Elements: Air and liquid flows were controlled using the computer. For the plant inflow, test clarifier recycle and system recycle lines, automatic valves were the final control elements. A positioner and variable speed motor controlled the rate at which mixed liquor was pumped from aeration compartment "C" to the test clarifier. A Parajust AC frequency controller was connected to the motor driving the Roots positive displacement blower. Motor speed was therefore varied in order to satisfy the required air flow levels.

Digital PI control algorithms were used to keep flows at the desired levels. Controller settings were originally selected using the Ziegler-Nichols method with fine tuning based on engineering judgement (Coughanowr and Koppel, 1965).

4.4 Sampling and Testing

In addition to on-line monitoring, a number of analytical tests were performed on samples collected from the system. The results for such "off-line" testing supplemented the on-line data to provide a complete description of plant operation and established the accuracy of the sensors and therefore the need for the recalibration or repair.

Composite samples were obtained of raw sewage before screening and effluents from both the test and system clarifier. Grab samples were collected of mixed liquor from aeration compartment "C" and return sludge from the system return sludge line. Table 4.4 indicates which tests were performed on the samples as well as the frequency of testing.

To reduce the time required for suspended solids determinations, samples were dried in a microwave (Campbell and Crescuolo, 1982). The measurement of settling rates employed a column with a

TABLE 4.4: Sampling and Testing Menu

Sample	Type	Measurement	Frequency of Measurement
Influent	Composite	• pH	2
		• Suspended Solids	1
		• Volatile Suspended Solids	2
		• Ammonia	4
		• Unfiltered BOD_5	4
		• Filtered BOD_5	4
Mixed Liquor	Grab	• Oxygen Uptake Rate	2
		• Suspended Solids	1
		• Volatile Suspended Solids	2
		• Stirred Sludge Volume Index (White, 1965b)	3
		• Zone Settling Velocity	3
		• Supernatant Suspended Solids (see text)	5
Return Activated Sludge	Grab	• Suspended Solids	1
		• Volatile Suspended Solids	2
Test Effluent		• Suspended Solids	1
		• Volatile Suspended Solids	2
System Effluent	Composite	• pH	2
		• Suspended Solids	1
		• Volatile Suspended Solids	2
		• Ammonia	4
		• Unfiltered BOD_5	4
		• Filtred BOD_5	4

Frequency: 1: Everyday.
2: Everyday, except weekends and holidays.
3: Two or three times per week during factorals.
4: Occasional.
5: Two or three times per week, first factorial only.

volume of 3.1 L, stirred at 1 rpm. An estimate of the concentration of dispersed solids contained in the mixed liquor and supplied to the test clarifier was estimated by sampling the supernatant in the column. Following 30 min of settling, a sample of supernatant was siphoned off and the concentration of suspended solids determined. On-line dissolved oxygen measurements were compared to readings obtained on a portable YSI dissolved oxygen meter.

4.5 Software

Flows were controlled, alarm conditions monitored, and data collected and analyzed under the supervision of software executed on the minicomputer. A portion of the software developed for the project was devoted to controlling liquid flows and blower output on the basis of digital PI control algorithms. An alarm was sounded and a message sent to the system CRT if flows were found to be outside of pre-set limits or a technician had forgotten to re-install a sensor following calibration. Another set of programs sampled the sensors and stored the data in disc files. Sensor data had one of two forms - averaged data or "dynamic" data. Day-to-day monitoring of all of the sensors resulted in data collected at 5 sec intervals and stored as 15-min averages. The "dynamic data" was collected from the sensors during step testing and was not averaged. A third group of programs was devoted to the analysis and presentation of results either as print-outs or plots.

All of the programs, written in FORTRAN '77, were documented. Program listings, sample outputs, instructions and an index were placed in a folder located in the computer room. All of the 16 programs developed for the research were documented in this way.

4.6 Manpower

The scope of the research and the complexity of the process equipment required the skills of a number of people. The individuals who contributed to the research and their duties during the program are outlined in Table 4.5. Routine plant maintenance and sampling was performed by the pilot plant operator on an 8:00 to 4:00 basis, Monday to Friday. "Call-ins", resulting from alarms, were handled as required. On weekends and holidays, a roster indicated which of a number of operators carried out the necessary sampling and testing for this and other projects at the Wastewater Technology Centre.

4.7 Experimental Procedures

a) Clarifier Dynamics: Step and ramp inputs were applied to the clarifier as the effluent suspended solids concentration was monitored with time. Clarifier response was tracked using the dynamic data acquisition program. The program sampled outputs from three of the sensors and stored the data in a disc file along with "time stamps", that is, the times at which the sensors were sampled. The frequency of sampling was specified by the user when the program was run.

Step changes in flow were made by changing the setpoints in the PI control loops. For ramp changes in flows, the control loop was notified that it was to read setpoint values from a disc file at

TABLE 4.5: Clarifier Research Personnel and Duties

Individual	Commitment	Major Duties
Dave Chapman	Full-time	• Project management • Experimental planning • Data analysis and presentation
Gordon Speirs	Occasional	• Modification and testing of DSM screen • DO sensor calibration • Tuning of air flow loop
Jim Matthews	Full-time	• Plant operation • Mechanical repairs • Plant modifications • Sensor calibration
Eric Luxon	Occasional	• Sensor installation and interfacing • Sensor "troubleshooting"
Brian Trapp	Half-time	• Sampling and analytical testing • Maintenance of composite samplers
Brian Monaghan	Occasional	• Software and data-base management • Software development
Mark Yendt	Occasional	• Software development
Ron Gillespie	Occasional	• Selection of airflow monitoring and control equipment • Pilot plant modifications

specified intervals. The list of setpoints in the file comprised the ramping function.

b) Steady-State Factorials: In all, three fractions of the design factorial were completed. Each fraction consisted of eight factorial ("+/-") runs and a number of centre-point ("0") runs.

Several days were required to complete one run. On the first day of the run, the clarifier was drained, washed down and any necessary changes made to the feedwell depth, weir position, and rake speed. The clarifier was then filled with the flows set at the required design levels. Influent feed and recycle were shifted to match experimental design requirements. The 15-min averages were plotted each morning, providing a pictorial diary of plant operation. A set of the plots for one of the days of a run (Factorial Run #10) has been photoreduced and is presented as Figure 4.4.

The flow-through vial in the test clarifier turbidimeter was cleaned each morning to remove condensation and deposited solids. Data collected from the morning following the "change-over" to the start of the next "change-over" was averaged to obtain the clarifier response.

For the first fraction, design levels were maintained for two days following the change-over. For centre-point runs, the length of run was one day. Each factorial run was followed by a centre-point run. Following the completion of the runs for a fraction, sensors were recalibrated and major repairs carried out.

For the second and third fractions, the length of the runs and the number of centre-point runs were changed. Two factorial runs

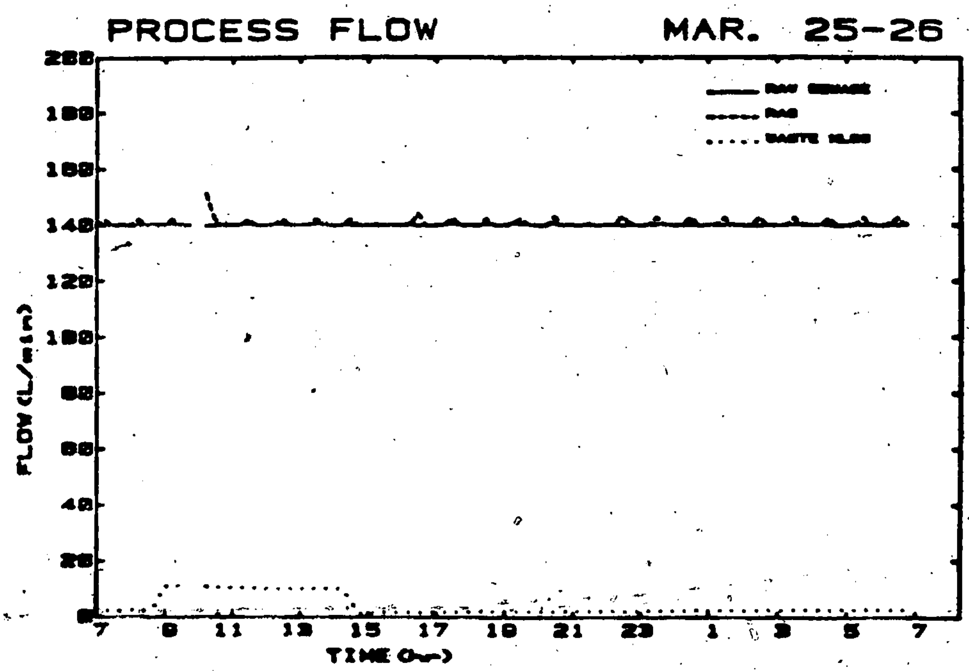


Figure 4.4 (a) Process Liquid Flow Rates (15-min. Averages)

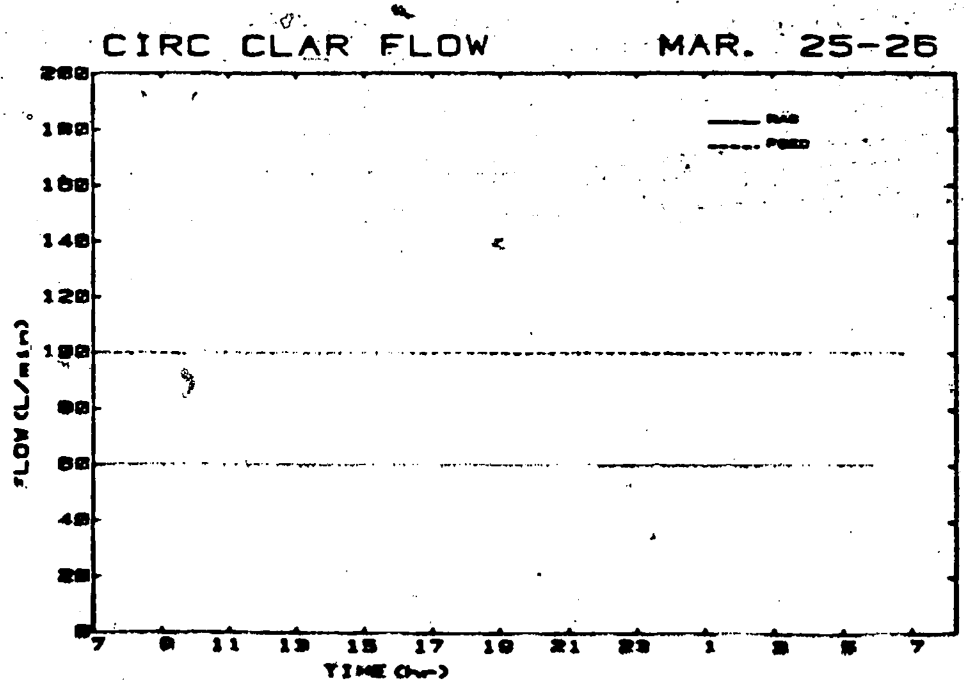


Figure 4.4 (b) "Test" Clarifier Flow Rates (15-min. Averages)

Figure 4.4 Example of Daily Plots of On-Line Data.

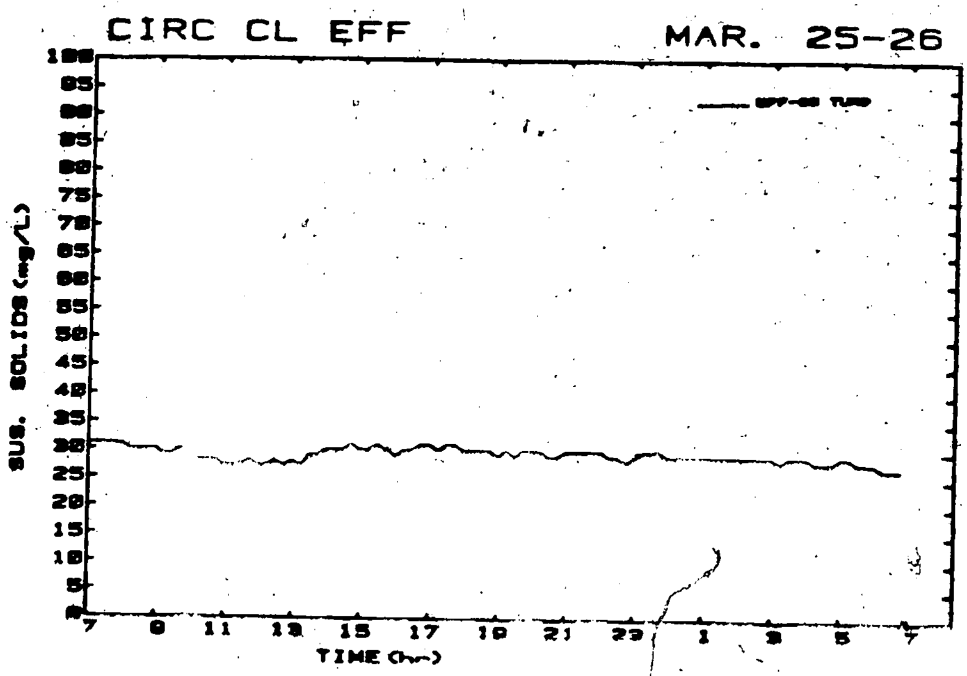
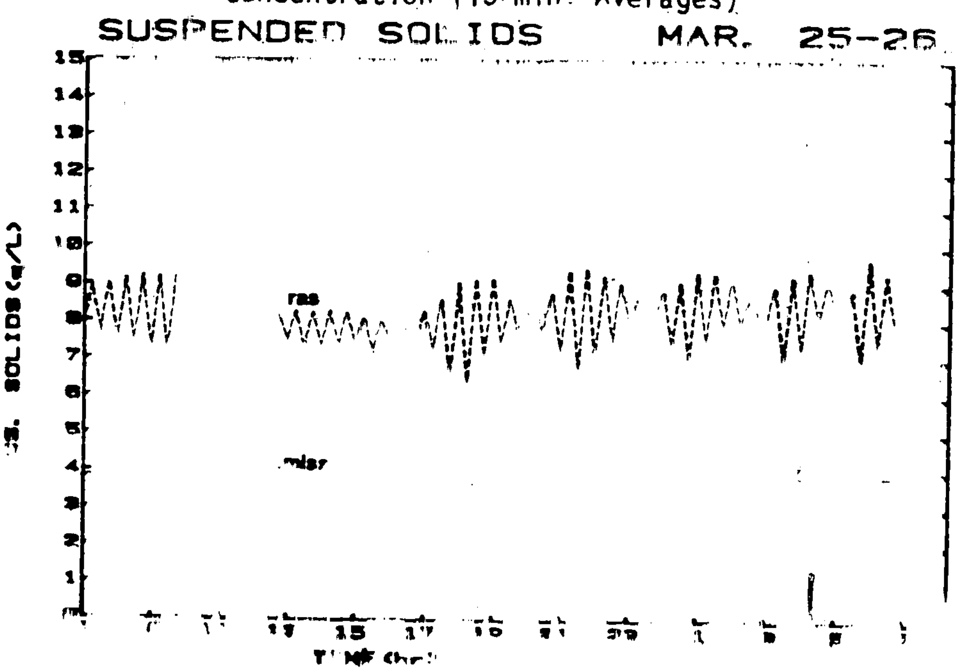


Figure 4.4 (c) "Test" Clarifier Effluent Suspended Solids Concentration (15-min. Averages)



MLSS a d Ret rn Act ated Sl dge Susp... ...min. A...

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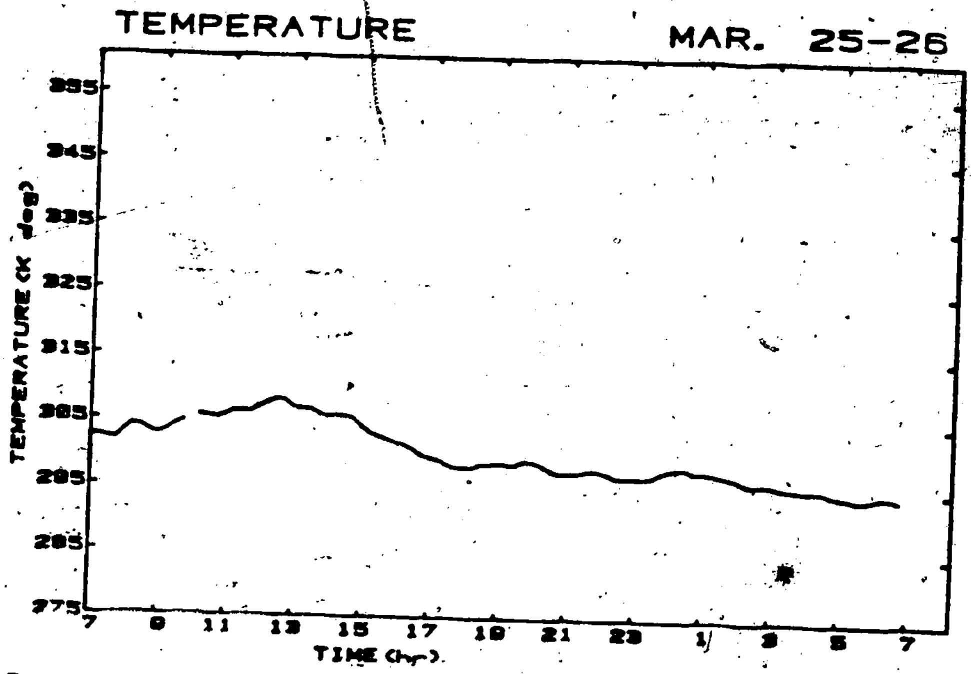


Figure 4.4 (e) Process Air Temperature (15-min. Averages)

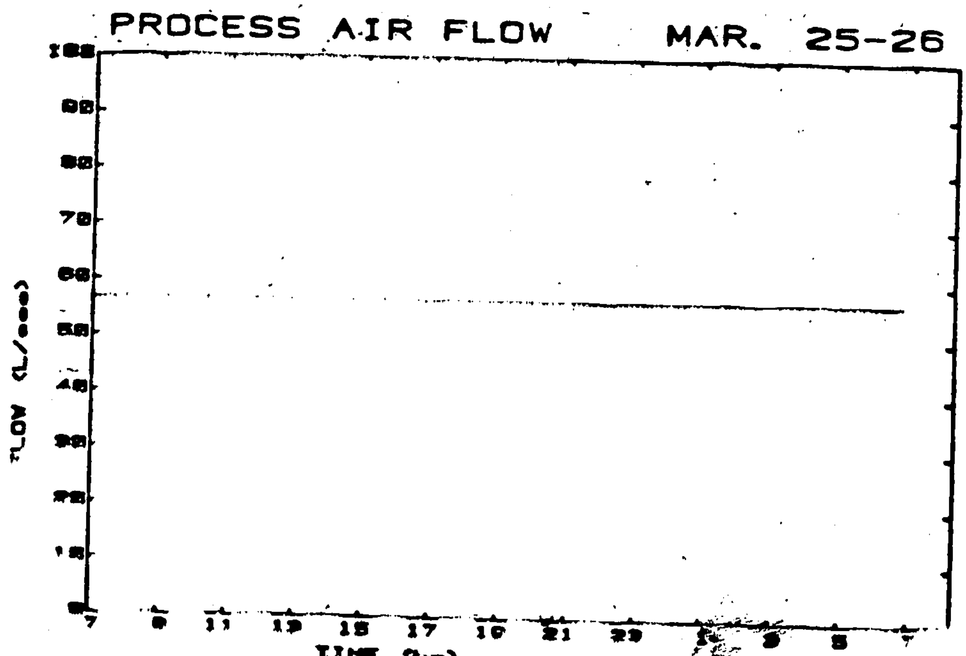


Figure 4.4 (f) Process Air Flow Rate (15-min. Averages)

Figure 4.4 Example of Daily Plots of On Line Data (Con't)

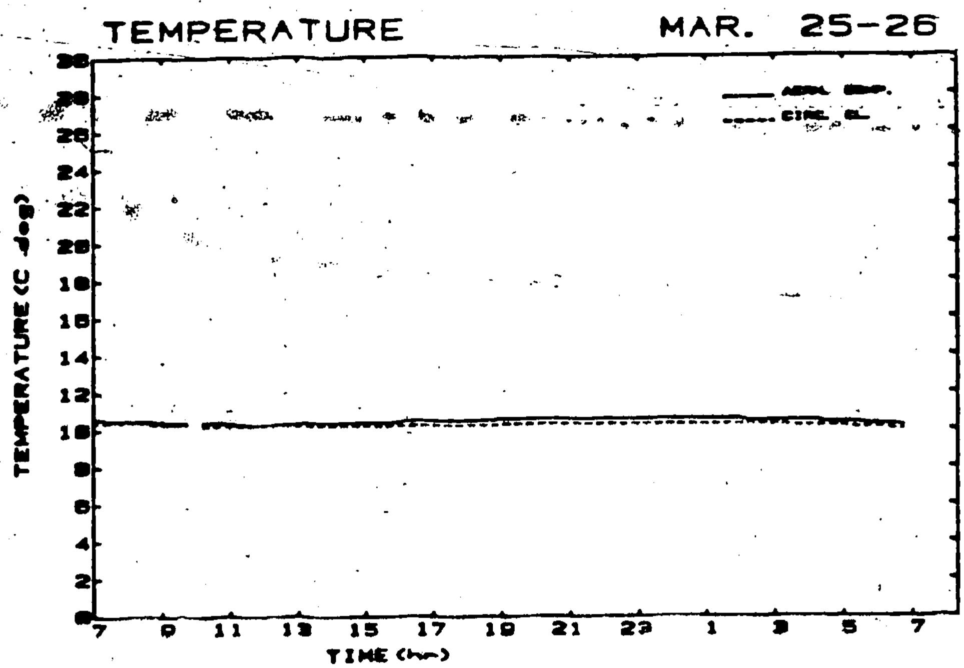


Figure 4.4 (g) Process Liquid Temperatures (15-min: Averages)

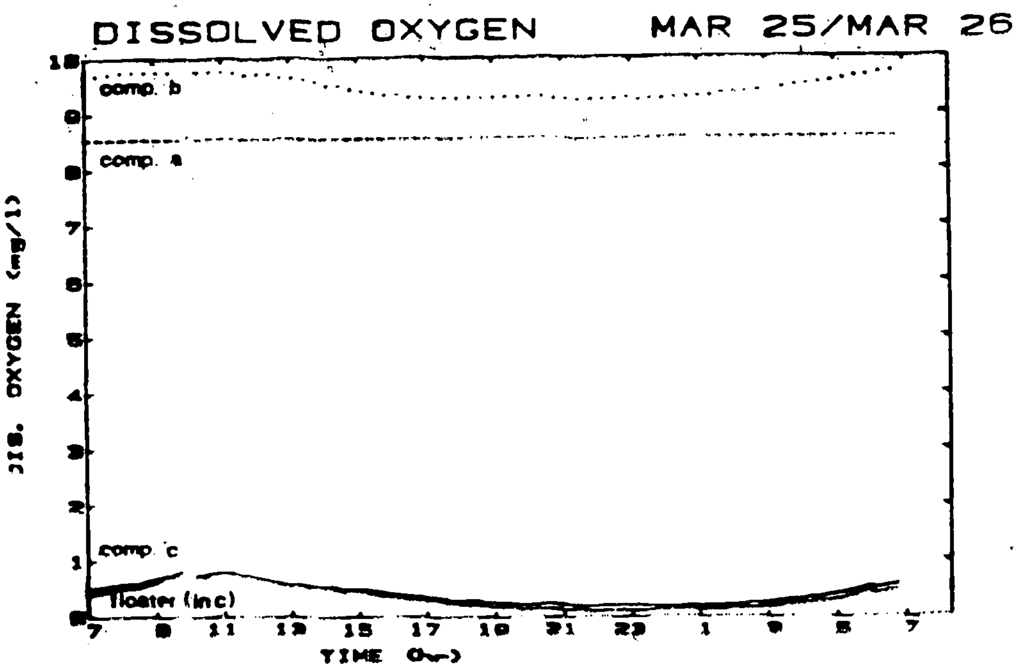


Figure 4.4 (h) Dissolved Oxygen Concentrations in the Three Aeration Tanks (15-min Averages)

Figure 4.4 Example of Daily Plots of On-Line Data (Con't) of one-day duration were completed on Tuesday and Thursday of each week, with centre-point levels in operation over the weekend. This schedule had the advantage of shortening the length of time to complete the runs for a fraction. Also, results collected over the weekend tended to be less reliable and less typical than those collected during the week. During the week, analytical tests were conducted by one technician and the pilot plant operator was on hand to catch any operational problems. During the weekend one of a number of operators carried out the testing and the pilot plant operator was only present if called in by an alarm condition. Further, it was observed that on weekends and holidays, the organic strength of the sewage entering the plant was very much weaker in comparison to the strength during the week. Because of these reasons, as far as was possible, factorial runs were carried out during the week.

c) Work Schedule: Using the equipment and procedures outlined in the previous sections, data was collected from May 1981 until July 1982. Table 4.6 presents a brief description and schedule of the completed experimental work.

TABLE 4.6: A Description and Schedule of Research Completed

Activity	Dates	Work Completed
Dye Testing	12/8 - 19/8/81	• 8 runs at different levels of feed flow rate, sidewater depth and underflow rate
Dynamic Test		
Series #: 1	25/8 - 13/9/81	• Step changes in feed flow and underflow rates
2	1/11 - 5/11/81	• Step changes in feed flow rate
3	23/2 - 20/4/82	• Step and ramp changes in feed flow rate
4	5/5 -14/5/82	• Changes in MLSS, air flow and rake speed
5	28/6 - 1/7/82	• Step and ramp changes in feed flow rate
Steady-State Testing		
Factorial #: 1	3/11 - 22/12/81	• 8 factorial and 7 centre-point runs
2	23/3 - 21/4/82	• 8 factorial and 5 centre-point runs
3	16/5 - 28/6/82	• 9 factorial and 6 centre-point runs

CHAPTER 5

RESULTS AND DISCUSSION

5.1 Off-Line Data

Off-line data collected from the pilot plant provided background information concerning wastewater strength and the overall efficiency of treatment. Appendix C contains plots of the most important measurements with mean and standard deviations indicated.

Table 5.1 summarizes the off-line data with respect to the composition of the influent wastewater and presents a classification system developed by Metcalf and Eddy (1979). The table shows, that on average, the strength of the sewage entering the pilot plant could be classified as "weak".

As indicated by the plot in Appendix C, the majority of instantaneous SRTs were maintained in the three to five-day range as targetted in the experimental plan. However, during weekends and holidays, instantaneous SRTs routinely exceeded five days. The organic strength of the wastewater entering the system during the weekend was much weaker than that during the week. In order to prevent the MLSS concentration from dropping below 1800 mg/L, the wasting rate was therefore decreased on weekends and holidays. Correspondingly high SRTs occurred during these periods.

The average effluent ammonia concentration was 4.9 mg/L (as N) representing a reduction of 70 percent in the average influent ammonia concentration. Table 5.2 gives additional information regarding nitrogen concentrations for three of the months covering the

TABLE 5.1: Off-Line Data: Composition of Influent Wastewater

Measurement	Mean (mg/L)	Standard Deviation (mg/L)	Sewage Strength Classification* Strong (mg/L)	Medium (mg/L)	Weak (mg/L)
Suspended Solids	113	41	350	220	100
Unfiltered BOD_5	94	32	400	220	110
Filtered BOD_5	36	17	-	-	-
Ammonia (as N)	17.2	4.9	50	25	12

* After Metcalf and Eddy (1979).

TABLE 5.2: Ammonia, Nitrite, and Nitrate Concentrations in the Influent and Effluent for Three Months

	Filtered Influent (mg/L as N)			Filtered Effluent (mg/L as N)		
Date	NH_3	NO_2	NO_3	NH_3	NO_2	NO_3
2/10/81	16.9	0.1	0.0	5.3	0.8	3.9
8/10/81	17.6	0.0	0.0	0.4	2.1	14.9
15/10/81	21.3	0.0	0.0	3.6	0.8	8.1
20/10/81	17.7	0.0	0.3	2.2	0.5	10.8
2/12/81	17.7	0.0	0.4	3.5	1.7	8.6
8/10/81	18.7	0.0	0.3	4.5	1.7	7.7
15/10/81	18.5	0.2	0.1	2.1	4.5	6.5
22/10/81	21.0	0.0	0.0	4.2	5.1	5.7
3/5/82	15.6	0.2	0.3	1.2	0.1	14.3
10/5/82	16.7	0.1	0.2	3.0	0.2	11.6
17/5/82	17.9	0.0	0.2	2.5	0.4	13.3
25/5/82	24.4	0.0	0.1	11.2	0.1	0.2

period of active research. As ammonia concentrations were lower in the effluent than in the influent and as nitrate concentrations increased from influent to effluent, it can be concluded that, despite low SRTs, nitrification was occurring. Consequently, a portion of the suspended solids lost in the effluent from the test clarifier during the experimental program was floating solids caused by the denitrification of sludge within the clarifier.

The BOD_5 concentrations of unfiltered samples from the system clarifier had a mean value of 11.8 mg/L and a standard deviation of 10.3 mg/L while for the filtered samples the mean was 3.8 mg/L and the standard deviation was 1.4 mg/L. Therefore, two-thirds of the total effluent BOD_5 was attributed to the escape of solids from the settler. Further, there was very little variability in the concentration of soluble BOD_5 in the effluent. Major excursions from average total BOD_5 levels were therefore caused by the discharge of suspended solids. These observations, confirmed by the literature, indicated that improving activated sludge performance mainly depended upon obtaining a better understanding of clarification.

To summarize, the off-line data indicated that weak sewage was entering the system. Instantaneous SRTs were controlled in the three to five-day range during the week. During weekends and holidays - periods of low organic loading - higher instantaneous SRT's were required to maintain MLSS concentrations. Two-thirds of the effluent BOD_5 from the system clarifier was suspended and one-third was soluble. There was considerable variation in the suspended fraction; very little in the soluble fraction.

5.2 Clarifier Dynamics

In order to investigate the effect of transient hydraulic loadings on clarifier performance, step changes were made to the feed flow rate into the clarifier. The concentration of suspended solids in the effluent was monitored as changes in the setpoint of the feed flow pump were made. Data was analyzed for 15 of the step tests. For seven of the tests, the flow rate was increased from 100 to 140 L/min, while for the remaining tests flow was decreased from 140 to 100 L/min. As the recycle rate was maintained at 40 L/min during the step changes, the overflow rate changed between 1.02 and 1.69 $m^3/m^2 \cdot hr$.

In addition to introducing changes to feed flow rate, changes were also made to the level of a number of other variables. The variables included the air flow rate, MLSS concentration and the speed of rotation of the sludge scraper arms. Knowledge of the time-varying response of the clarifier to changes in the level of these variables aided in interpretation of steady-state results and enabled a more complete description to be developed of the dynamics of clarification.

The results of the dynamic testing are discussed in the following sub-sections.

5.2.1 Response to a Step Decrease in Flow. To estimate the order of dynamics, a linear least squares analysis was performed on the data (Appendix D) collected following a step decrease in flow. Values for the residual sum of squares were calculated for a series of linear models starting with a first-order linear model through to a fifth-order linear model. The reduction in the residual sum of squares from one model to the next indicated the order which best represented the response to a step down.

Assume that two models are fitted to Nd data points. The first model contains np_1 parameters. The second contains the same parameters as the first plus a number of additional parameters resulting in a total of np_2 parameters. To determine if the additional parameters result in a model which has a significantly better fit, a statistic, Y, is calculated as follows:

$$Y = \left(\frac{RSS_1 - RSS_2}{RSS_2}\right) \left(\frac{Nd - np_2}{np_2 - np_1}\right) \tag{41}$$

where: RSS_1 = residual sum of squares for model 1,

RSS_2 = residual sum of squares for model 2,

Nd = number of data points to which models are fitted,

np_1 = number of parameters contained in model 1, and

np_2 = number of parameters contained in model 2.

Y has an F-distribution. Therefore, if the value of Y is greater than $F(np_2 - np_1, Nd - np_2, \beta)$, we are confident at the β percent level that the addition of the parameters was justified. A more detailed discussion of the procedure as applied to the identification of system dynamics is found in Astrom and Eykhoff (1971).

Misleading results from the least squares analysis can arise if there are serially correlated residuals. As effluent suspended solids measurements collected close together in time are more likely to be correlated than are measurements separated by a suitable length of time, a preliminary analysis was made of the effect of various time intervals on the results from the least squares procedure. Consistent estimates of the order of dynamics were obtained provided that data points were separated by a time interval of not less than 5 min. A time interval of 8 min was selected and used in the subsequent analyses of the "step-down" data.

The plots for the step decreases are contained in Appendix D. No time lags are evident from the change in feed flow rate to the start of the change in effluent concentration. The order of the response as estimated from the least squares analyses is presented in Table 5.3. In addition, the table contains estimates for the time constant (τ), the process gain (Kp), and the standard deviations of effluent turbidity preceding and following the step change.

Table 5.3 indicates that a first-order model is adequate to describe the response of the clarifier to step decreases in feed flow rate. Therefore, the appropriate transfer function, relating time-varying inputs and outputs, is:

$$G(s) = \frac{Ce(s)}{Qa(s)} = \frac{Kp}{(1 + \tau s)} \qquad (42)$$

TABLE 5.3 Summary of Results for Clarifier Response to a Step Decrease in Feed Flow From 140 to 100 L/min

	Effluent Suspended Solids Concentration						
	Mean		Standard Deviation				
Step Number	Before Step (mg/L)	After Step (mg/L)	Before Step (mg/L)	After Step (mg/L)	Kp[1] ($mg \cdot min/L^2$)	τ[2] (min)	Order[3]
2	74.9	54.6	4.3	1.9	0.51	26.	1st
3	44.8	-	2.0	-	-	24.	1st
7	48.9	41.3	1.3	1.0	0.19	24.	1st
8(b)	48.3	38.5	1.6	1.1	0.25	32.	1st
9	43.4	36.8	1.0	1.0	0.17	23.	1st
10	-	34.1	-	1.1	-	30.	1st
21	27.5	22.4	2.6	2.4	0.13	20.	1st
25	-	16.1	-	1.2	-	25.	1st

[1] Kp:- The process gain is the ratio of the change in output to the change in input (Liptak, 1970).

[2] τ:- The time required for the output of a first-order system to reach 63.2 percent of a complete response to a step input (Liptak, 1970).

[3] Order tested at the 99 percent confidence level.

where: $G(s)$ = transfer function,

$Ce(s)$ = Laplace transform of function describing effluent suspended solids concentation,

$Qa(s)$ = Laplace transform of function describing feed flow rate,

Kp = process gain,

τ = process time constant, and

s = Laplace transform variable.

The process time constant (τ) averaged 26 min with a standard deviation of 4 min. In comparison, the theoretical residence time, or HRT, based on net flow rate over the weir was 90 min before the step decrease and 150 min after the step. The process gain for the seven step decreases averaged 0.25 mg/L per L/min change in feed flow rate. The variability of the effluent turbidity, as estimated by the standard deviation, decreased following a step decrease in flow rate.

For step decrease #2, the estimate for the time constant (obtained from the 39, 63, and 78 percent values) was compared to an estimate obtained from the fraction incomplete method. At any time t, the value for the fraction incomplete (FINC) is determined from:

$$FINC = 1 - \frac{Ce(t) - Cei}{Cef - Cei} \quad (43)$$

where $Ce(t)$ = the effluent suspended solids concentration at time t,

Cei = the initial steady-state suspended solids concentration, and

Cef = the final steady-state suspended solids concentration.

The slope of ln (FINC) versus time plot is an estimate of the time constant. For step decrease #2, the results are plotted in Figure 5.1. Based on the plot, the time constant is approximately 23 min. compared to 26 min based on the 39, 63, and 78 percent values. Because there was reasonable agreement between the two estimates, all of the other step changes were analyzed using the 39, 63 and 78 percent values only.

The plots and analyses of data from the responses to the step decreases indicate that feed flow rate is an important variable influencing process performance.

5.2.2 Response to a Step Increase in Flow. The response to a step decrease in flow was consistent from test to test, being characterized by an exponential decay in the effluent suspended solids concentration. As can be seen from the plots contained in Appendix [illegible], effluent turbidity increased following each step increase in feed flow rate. However, in contrast to the responses following step decreases, these plots show noticeable variation in the shape of [illegible]

[illegible] Analysis of [illegible] results [illegible]

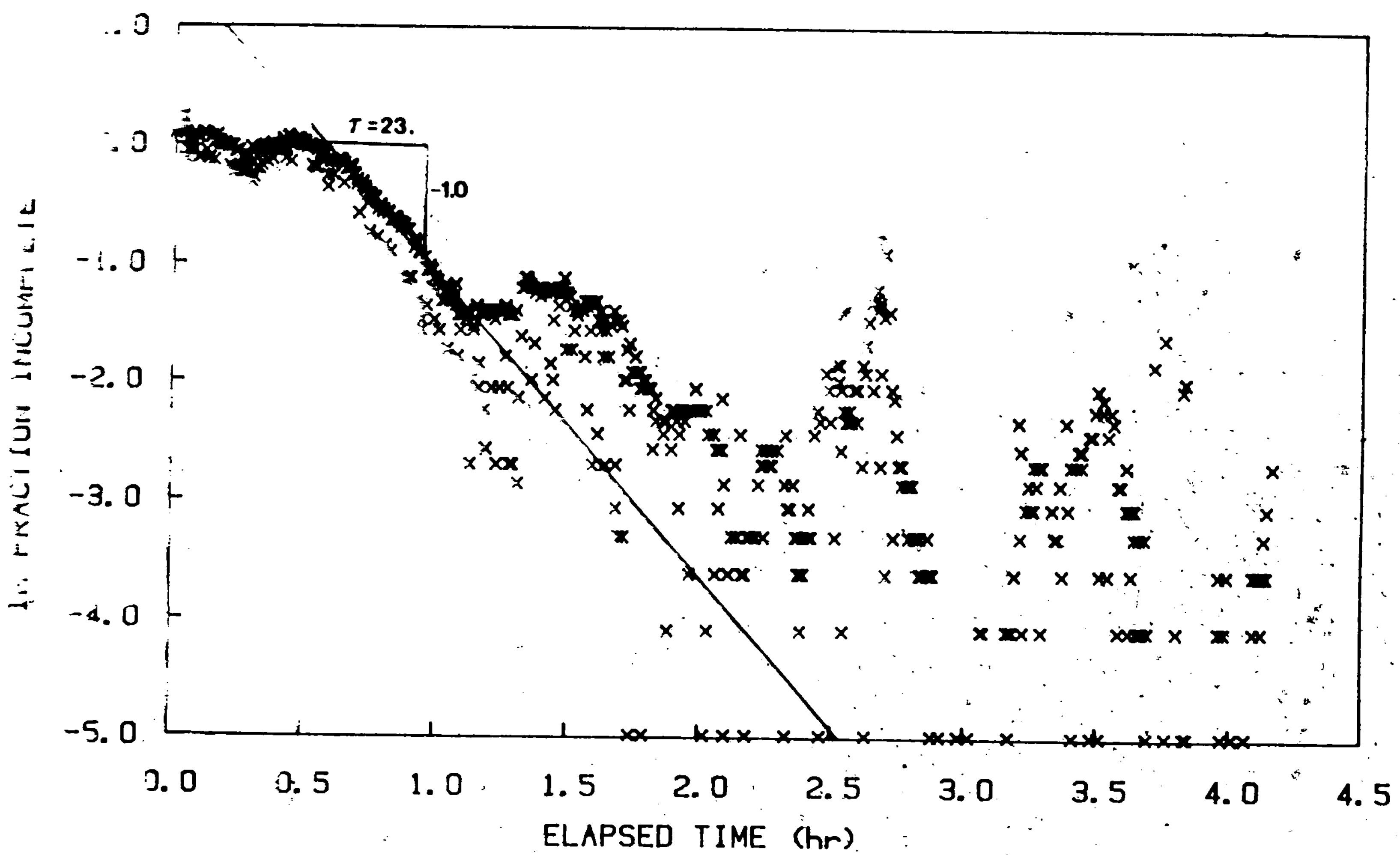


Figure 5.1 Fraction Incomplete Plot for Data from Step Decrease #2.

reflected this variability. In particular, consistent results regarding the order of dynamics could not be obtained using the least squares technique despite repeated attempts using a number of different intervals between the adjacent data points. Therefore, the results of the least squares anlayses are not presented and no estimate of the order was made for the dynamics.

For the step increase responses, Table 5.4 contains a summary of the estimated time constants (τ), the process gains (Kp), the time lags (τ_D, the length of time between the start of the change in flow and the start of the change in response), the mean and standard deviations before and after the step change and values characterizing overshoot.

The average gain for the step increases was 0.22 mg/L per L/min increase in flow rate. The variability of the response, as estimated using the standard deviation, increased as well. The average time constant for the step increases was of the order of 17 min with time lags averaging 1.6 min.

The presence of overshoot in some of the responses implies that a model must be at least second-order if the response of the settler to increases in flow rate is to be predicted. The transfer function for a second-order model with time lag is:

$$G(s) = \frac{Ce(s)}{Qe(s)} = \frac{Kp\, e^{-\tau_D s}}{\tau^2 s^2 + 2\zeta\tau s + 1} \qquad (44)$$

TABLE 5.4 Summary of Results for Clarifier Response to a Step Increase in Feed Flow from 100 to 140 L/min

Test Number	Effluent Suspended Solids Concentration: Mean, Before Step (mg/L)	Mean, After Step (mg/L)	Standard Deviation, Before Step (mg/L)	Standard Deviation, After Step (mg/L)	Kp[1] (mg·min/L^2)	τ[2] (min)	Time Lag (min)	Overshoot[3]
2	54.5	67.0	1.5	1.6	0.31	15	2.3	-
3	33.1	44.5	1.1	2.1	0.29	-	1.2	2.5
4	31.1	37.8	1.1	3.6	0.17	15	1.9	-
7	40.8	47.3	1.2	1.4	0.16	18	1.9	-
10	33.0	40.6	1.0	1.0	0.19	17	1.1	-
12	32.2	37.4	1.2	1.0	0.13	17	1.9	-
21	14.0	26.9	1.0	1.1	0.32	19	0.9	7.1
25	18.8	25.0	1.3	1.8	0.16	18	1.2	0.8

[1] Kp:- The process gain is the ratio of the change in output to the change in input (Liptak, 1970).

[2] τ:- The time required for the output of a first-order system to reach 63.2 percent of a complete response to a step input (Liptak, 1970).

[3] Overshoot is the ratio of the difference between the maximum response value and the final steady-state value to the difference between the initial and final steady-state values.

where: $G(s)$, $Ce(s)$, $\dot{Q}a(s)$, Kp, s are defined as per equation 42,

τ_D = time lag,

ω_n = natural frequency of oscillation, and

ζ = damping ratio.

For those responses displaying overshoot, there was a wide variation in the degree of overshoot.

5.2.3 Response to Changes in Other Variables. Changes were introduced to the air flow rate, MLSS concentration and the rotational speed of the scraper arms. The response in terms of effluent quality was observed from daily plots of the 15-min averages. During the tests the setpoints for all of the other controlled variables remained constant.

a) Air Flow Rate: Figure 5.2 is a plot of the clarifier response to a change in air flow rate from 56 L/sec to 76 L/sec. The plot indicates that, over the range tested, clarifier effluent concentration was unaffected by the change in air flow rate.

b) MLSS Concentration: Four tests were conducted to investigate the influence of time-varying MLSS concentrations. For the first test, the MLSS concentration was increased by shifting the point of addition of return activated sludge (RAS) from the first aeration tank to the third aeration tank (referring to Figure 4.2, from tank "A" to tank "C"). The point of addition of raw sewage remained at tank "A". As tank "C" supplies the test clarifier with mixed liquor, altering the concentration of MLSS in this tank altered the concentration of

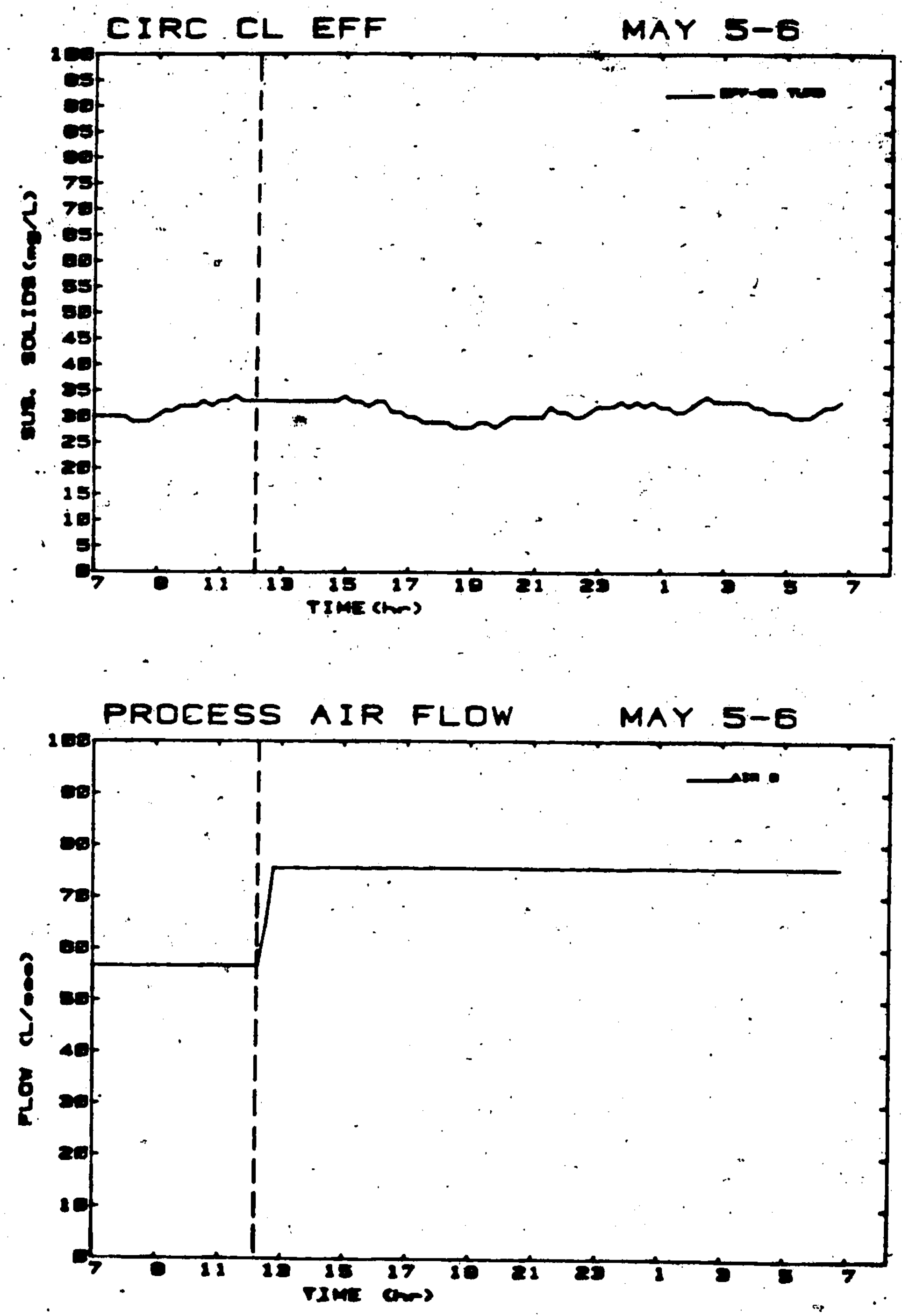


Figure 5.2 The Change in Effluent Suspended Solids Concentration Resulting from a Step Increase in Air Flow Rate (May 5-6, 1982).

suspended solids in the feed to the clarifier. For the second test, the reverse procedure was carried out. The point of addition of recycle flow was shifted back to tank "A" from tank "C".

Preceding the start of the third test in this series, raw sewage was pumped into tank "C", with recycle flow emptying into tank "A". The third test consisted of a two-stage increase in MLSS concentration. The point of addition of raw sewage was shifted from tank "C" to tank "A". Following that operation, RAS addition was shifted from "A" to "C". The reverse procedure was followed for the final test in the series. The clarifier response to these changes in MLSS concentration is contained in Figures 5.3 to 5.6. Note that the response of the settler in terms of effluent suspended solids is the solid line in the upper plot while the forcing function is the solid line in the lower plot. The dashed line is a plot of suspended solids concentration in the settler underflow.

The forcing functions created by manipulating the points of addition and recycle were not steps. Therefore, the methods used in analyzing the response following step changes in flow rate could not be applied to the response following the more complex changes in MLSS concentration. Instead, a first-order difference equation was fitted to the data.

For a first-order model, the appropriate differential equation relating output to input is:

$$\tau \frac{dCe(t)}{dt} + Ce(t) = Kp\ MLSS(t) \tag{45}$$

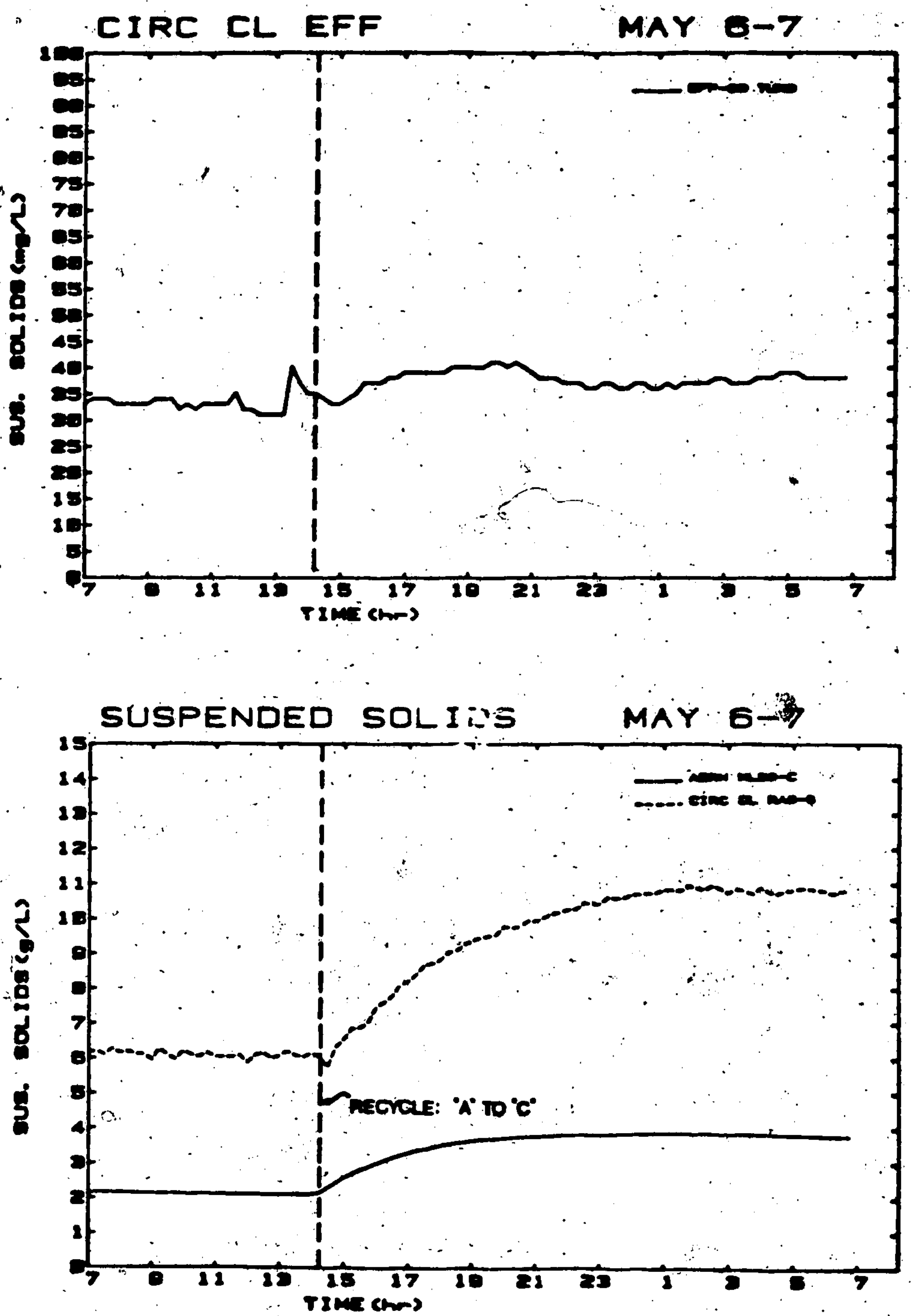
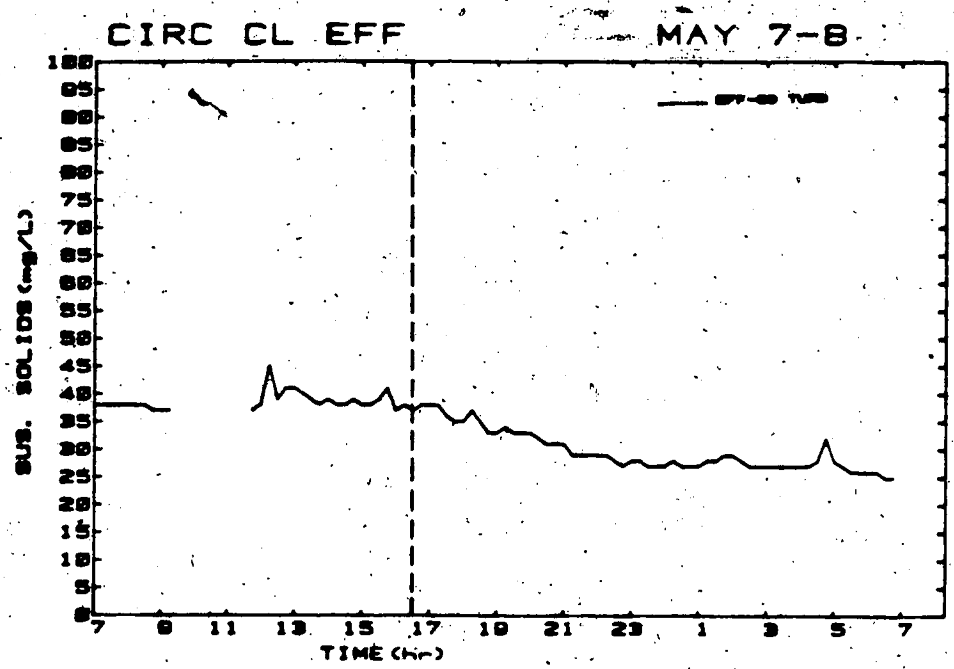


Figure 5.3 The Change in Effluent Suspended Solids Concentration Resulting from an Increase in MLSS Concentration (May 6-7, 1982)



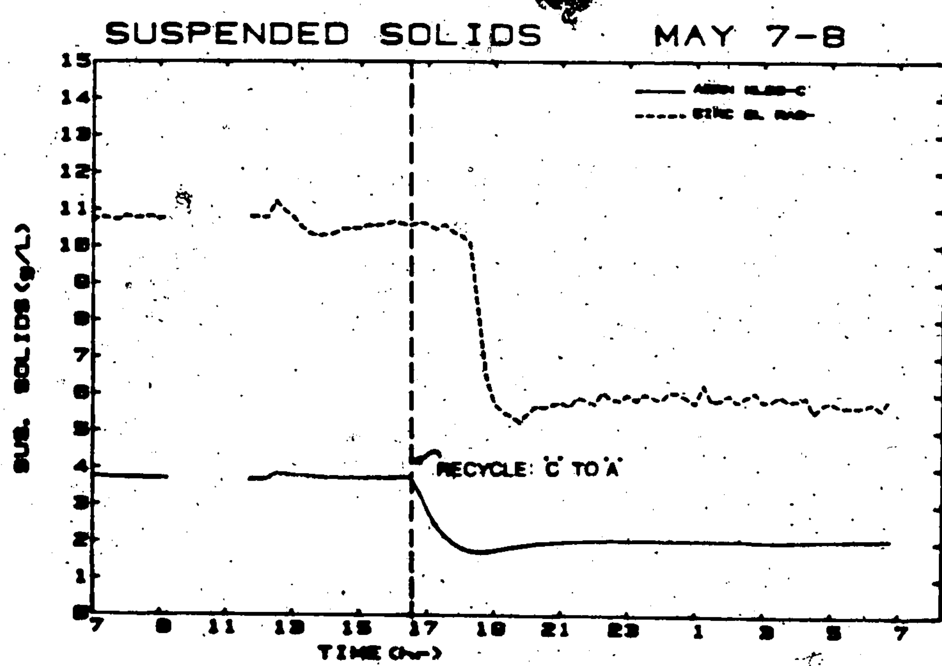


Figure 5.4 The Change in Effluent Suspended Solids Concentration Resulting from a Decrease in MLSS Concentration (May 7-8, 1982).

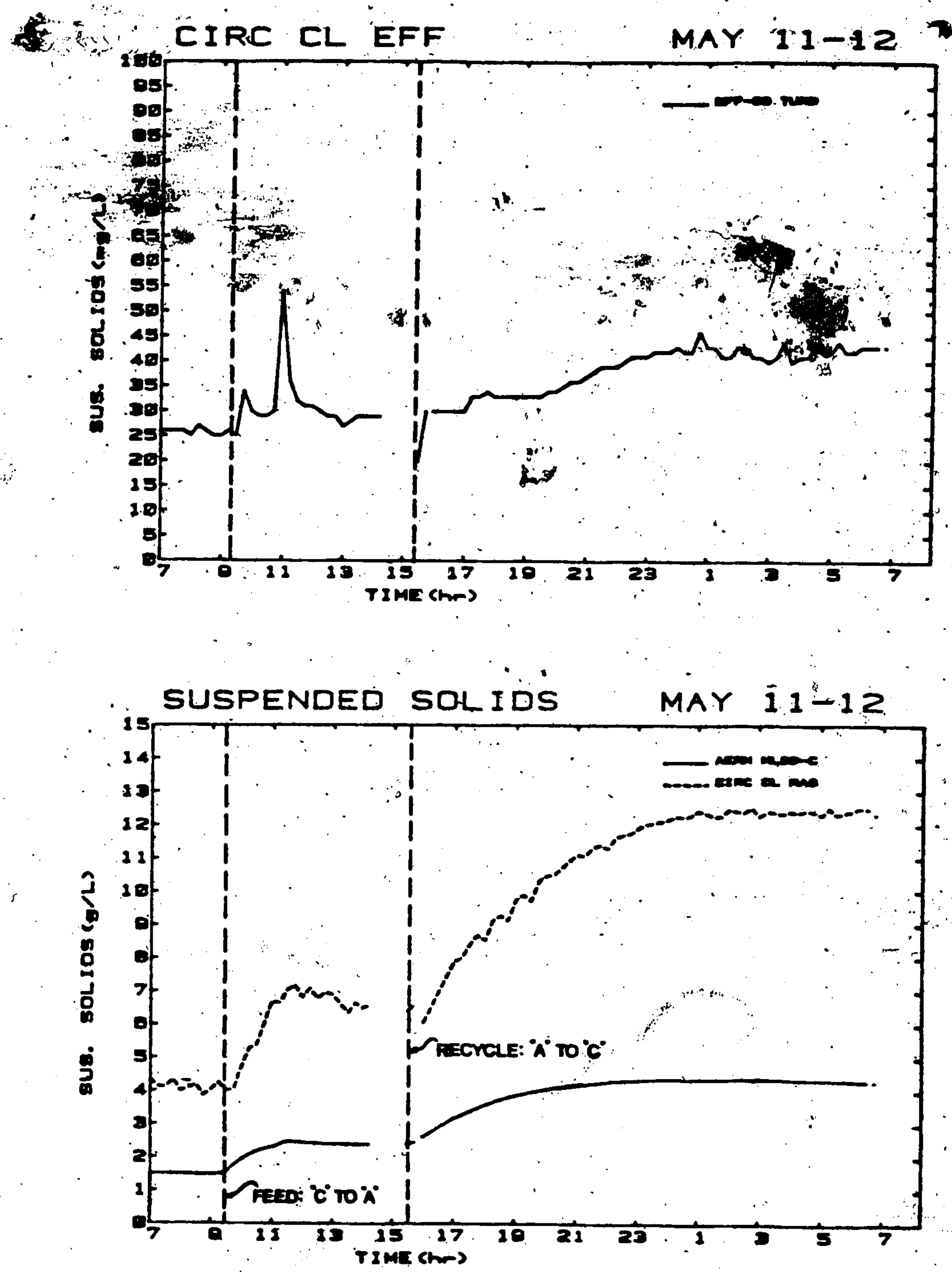


Figure 5.5 The Change in Effluent Suspended Solids Concentration Resulting from Increases in MLSS Concentration (May 11-12, 1982).

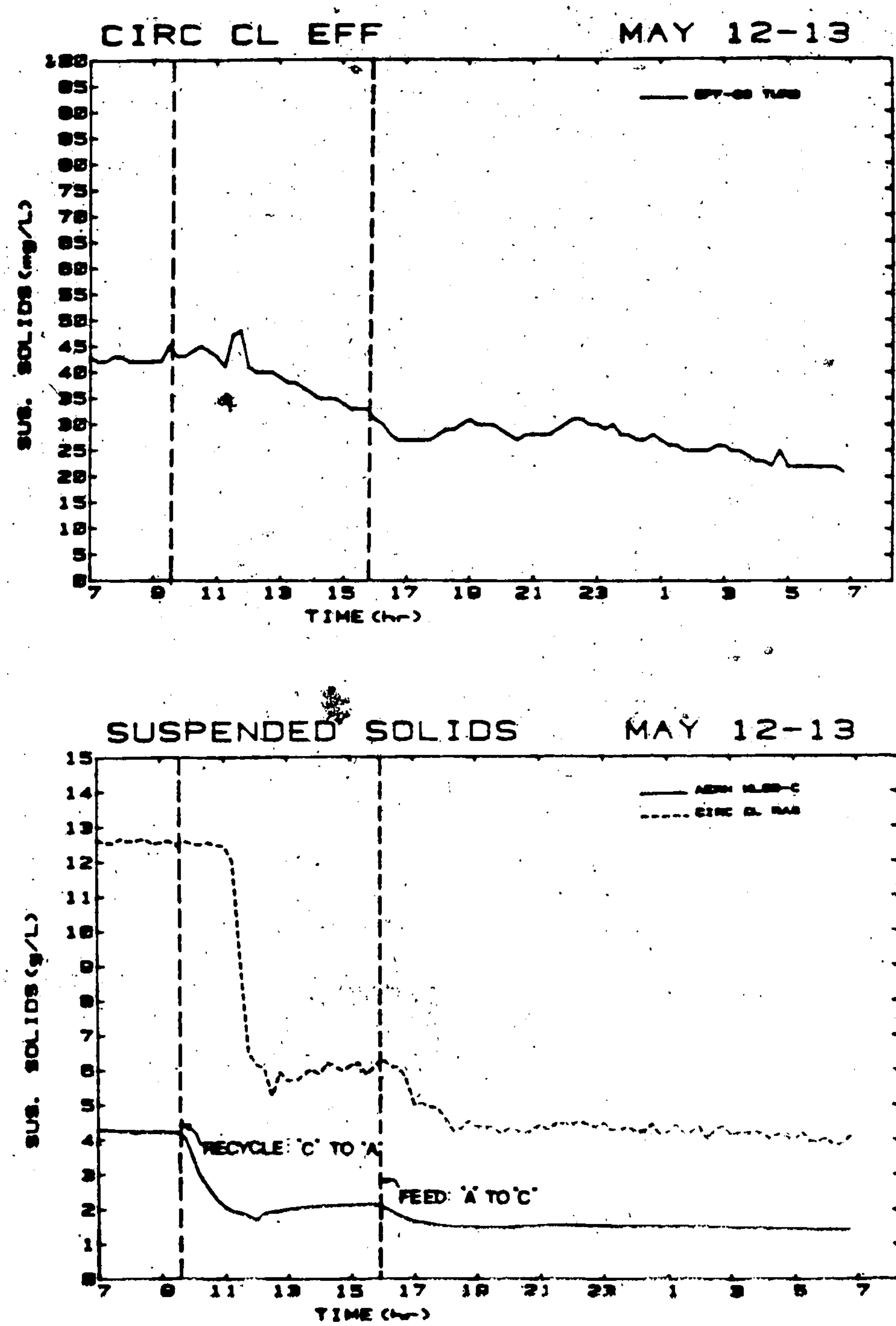


Figure 5.6 The Change in Effluent Suspended Solids Concentration Resulting from Decreases in MLSS Concentration (May 12-13, 1982).

where: $Ce(t)$ = time-varying effluent suspended solids output

$MLSS(t)$ = time-varying MLSS concentration

τ = time constant of system

Kp = process gain.

The above equation can be approximated using a finite difference in place of the derivative:

$$\tau(Ce_{K+1} - Ce_K)/Ts + Ce_K = Kp\ MLSS_K \tag{46}$$

where: Ce_{K+1} = effluent suspended solids concentration at the K+1 time interval

Ce_K = concentration at the Kth time interval

$MLSS_K$ = MLSS concentration at the Kth time interval

Ts = the sample interval.

By solving the following equation for Ce_{K+1} the following results:

$$Ce_{K+1} = (1 - \tau/Ts)Ce_K + (Ts/\tau)\cdot Kp\cdot MLSS_K \tag{47}$$

or,

$$Ce_{K+1} = a_1\,Ce_K + b_1\ MLSS_K \tag{48}$$

where a_1, b_1 = functions of τ, Ts or Kp.

Values of effluent suspended solids concentration (Ce) and MLSS concentration were automatically stored in data files at a specified interval (Ts) by the data collection programs. Regressing Ce_{K+1} on Ce_K and $MLSS_K$ provided estimates of a_1 and b_1. The process time constant (τ) was then related to the regression coefficient a_1 using (Olsson, 1984):

$$a_1 = e^{-Ts/\tau} \tag{49}$$

Table 5.5 summarizes the values for the regression constant a_1 and the estimates of the time constant (τ).

Based on Table 5.5, the most important points can be summarized as follows:

i) A change in effluent concentration followed a change in MLSS concentration. The process gain was in the range of 4 to 7 mg/L of effluent suspended solids per g/L change in MLSS concentration.

ii) Three of the four responses could be adequately modelled using first order dynamics. The [illegible] transfer function is therefore:

$$G(s) = \frac{Ce(s)}{MLSS(s)} = \frac{Kp}{[illegible]}$$

TABLE 5.5 Summary of Results for Clarifier Response to Changes in MLSS Concentration

	MLSS Concentration		Effluent SS Concentration					
Date	Direction	Magnitude (g/L)	Before Step (mg/L)	After Step (mg/L)	Kp (mg/g)	a_1 -	τ (h)	Order*
May 6-7	Up	1.7	33.	39.	3.5	0.952	5.1	1st
May 7-8	Down	1.8	38.	27.5	5.8	0.962	6.5	1st
May 11-12	Up	1.9	26.	42.	5.5	0.908	2.6	2nd
May 12-13	Down	1.7	43.	23.	7.4	0.953	5.2	1st

* Tested at 99 percent confidence level.

where G(s), Ce(s), Kp, τ, s are defined as per equation 42 and

MLSS(s) = Laplace transform of function describing MLSS concentration.

iii) The average process time constant (τ) for the four tests was approximately five hours, indicating that the response to MLSS induced changes was much slower than the response to hydraulically induced changes.

c) <u>Rake Speed</u>: As effluent concentration was monitored, the rotational speed of the sludge scraper arms was increased from 5 to 8 rev/h. As shown in Figure [illegible], no noticeable change in effluent concentration was observed.

5.2.4 <u>Discussion of Dynamic Results</u>. The observation that the transient response of the activated sludge system to changes in hydraulic loading was nonlinear finds support in the published literature. Collins and Crosby (1980), for instance, expressed concern about the effects of transient hydraulic loads on clarifier performance. They indicated that turbulence should be generated very quickly at higher velocities but that it will die off on a time scale equal to, or greater than, the retention time of the settler.

In the literature on the steady-state performance of the secondary clarifier, it was noted that deviations from Camp's model of ideal settling were created by a density current. It was postulated that this "density current or vortex" will exert a major influence over the dynamic behaviour of the clarifier as well [illegible]

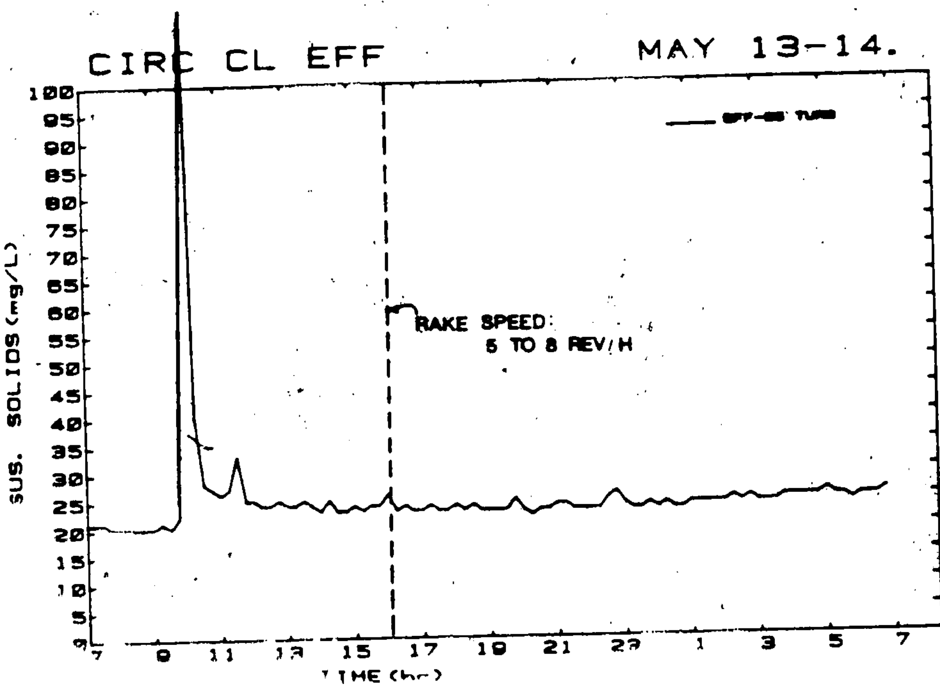


Figure 5.7 The Change in Effluent Suspended Solids Concentration Resulting from an Increase in Rake Speed (May 13-14, 1982).

feed flow rate alters the momentum of the vertical roll. As a consequence, the level of turbulence and mixing in the tank are altered as are the rate of scour of solids from the sludge blanket and possibly the stability of sludge/supernatant interface.

A step decrease in flow decreased the level of mixing in the supernatant above the sludge. The exponential decline in effluent suspended solids observed following a step decrease in flow rate reflects a decay in the level of turbulent energy. The decrease in the level of variability of effluent suspended solids following the step decreases was evidence of the decrease in turbulence in the tank.

The wide variability in the response of the clarifier to a step increase in flow rate from one run to another indicated that several mechanisms, in addition to an increase in turbulence, were responsible. The flow of mixed liquor into a clarifier closely resembles the flow of a sediment-bearing river into a lake. Both situations result in a surface of discontinuity created by differences in velocity and density between two fluids. As described in the literature on physical limnology, waves and, in particular unstable eddies may form at the surface of discontinuity (Smith, 1975). The explosive collapse of distortions in the surface of discontinuity, referred to as Kelvin-Helmholtz instability, has been observed followed by a return to stable conditions (Smith, 1975). Given the similarities between flow in a secondary clarifier to lake flow, it is possible that some of the irregularity in the plots of the clarifier response can be attributed to the generation and collapse of undulations in the sludge blanket prior to the establishment of stable conditions at a higher flow rate.

The presence of floating solids on the tank surface also influenced the response of the clarifier to a step increase in flow. Denitrification occurred during the course of the experimentation causing some flocs to float due to the entrapment of nitrogen bubbles. Those solids, trapped between the scum baffle and the effluent weir, were dislodged by a sudden increase in feed flow rate. The presence of overshoot in a number of the runs was attributed to the presence of floating solids outside the scum baffle. For full-scale settlers, the ratio of surface area between the scum baffle and effluent weir to total surface area is lower than for the test clarifier. The degree of overshoot will, therefore, be exaggerated at pilot-scale in comparison to full-scale.

Variations in MLSS concentration were induced by shifting the points of addition of raw sewage and recycle sludge. Consequently, changes were observed in the level of suspended solids in the effluent from the test clarifier. Such results could arise from altered energy levels as density differences change, from changes in the rate at which nonsettleable solids entered the settler, or from changes in the volume of the clarification zone as the sludge blanket changes. Given that the MLSS induced responses were much slower than the responses induced by time-varying hydraulic loads, the last two the explanations seem more plausible.

In summary, the results of repeated hydraulic step tests indicated that the level and variability of the concentration of suspended solids in the effluent from the clarifier changed following a change in feed flow rate into the clarifier. The response to a step decrease in flow was characterized by a first-order decay in the level of turbulence in the fluid above the sludge blanket. The responses to a step increase in flow were faster and displayed considerable variation from one run to another. A step increase in flow resulted in an increase in the level of turbulence and possibly the generation and collapse of undulations in the sludge blanket (Kelvin-Helmholtz instability). Overshoot, observed in some of the responses to a step increase in flow, was attributed to the presence of floating solids between the scum baffle and effluent weir. They were swept over the weir by the increased momentum of the "density current" as the flow increased. Changes in effluent suspended solids concentration were also observed following changes induced in MLSS concentration. Because the estimated time constants for these responses was large, they were likely created either by a change in the rate at which nonsettleable solids entered the settler or by changes in the volume of the clarification zone above the sludge blanket.

5.2.5 Modelling Recirculation Effects. Evidence of the existence of a vertical roll within final settlers comes from tracer studies as well as from the direct measurement of fluid velocities within the tank. Researchers have observed that tracer washout curves obtained from settlers subjected to a pulse input of tracer frequently display a series of two or more peaks of concentration which attenuate with time (Murphy, 1964; Thyn and Hansson, 1975). The existence of these peaks indicates that fluid is recirculating within the settler.

One view of the clarifcation process holds that it is the concentration of "primary particles" entering the settler which largely determines the efficiency of suspended solids removal (Tuntoolavest et al., 1980). "Primary particles" are stripped from the flocs by the action of turbulence in the aeration basin and are too small to settle. If this view of clarification is correct and it is assumed that action of both flocculation and scour can be neglected, then primary particles entering the settler will be discharged in the effluent, in effect, acting as an inert tracer. A number of the plots of settler response following step changes in feed flow rate display a repeating series of peaks. The plots for step decreases #1, #7 and #10 (Appendix D) and step increase #2 (Appendix D) show them most clearly. Their presence is direct evidence that flow is recirculating within the test settler.

To confirm that the peaks were caused by fluid recirculation and gain additional insight into the mechanisms responsible for suspended solids removal, output from a recirculation model was compared to plots of the settler response. The model consisted of a series of CSTRs with a plug flow line recycling a portion of the flow back to the first CSTR. A form of this model was used previously by Thyn and Hansson (1975) to study fluctuations in a tracer washout curve from a final settler.

A step change in feed flow rate into the settler was therefore assumed to correspond to a step increase in inert tracer into the recirculation model. The response of the model was determined by a FORTRAN program which solved differential equations derived from mass balances using an Euler-type integration.

A schematic of the flow model is given in Figure 5.8. Figure 5.9 shows the model output for two reactor configurations subjected to pulse inputs of inert tracer. Both configurations have the same tank volume although one has recirculation and the other does not. The effect of recirculation is firstly to push the tracer peak through to the effluent at a faster rate. Secondly, a portion of the tracer is recirculated back to the inlet and re-introduced into the reactor. A second and third tracer peak therefore appears in the effluent.

A comparison of the output from the recirculation model and the response following a step decrease in feed flow rate are shown in Figure 5.10. (By plotting moving averages instead of raw data, the settler response has been filtered to remove high frequency noise.) The output of the recirculation model to a step decrease in tracer concentration consists of an exponential decrease in tracer concentration upon which is superimposed equally spaced plateaus, attenuating with time. The shapes of the two curves are quite similar, giving credence to the hypothesis that fluid recirculation influenced the transient response of the settler.

Figure 5.11 shows the model output for a step increase in influent tracer concentration and the filtered response of the settler following a step increase in feed flow rate. There is little similarity in the shape of the plots containing the predicted and measured

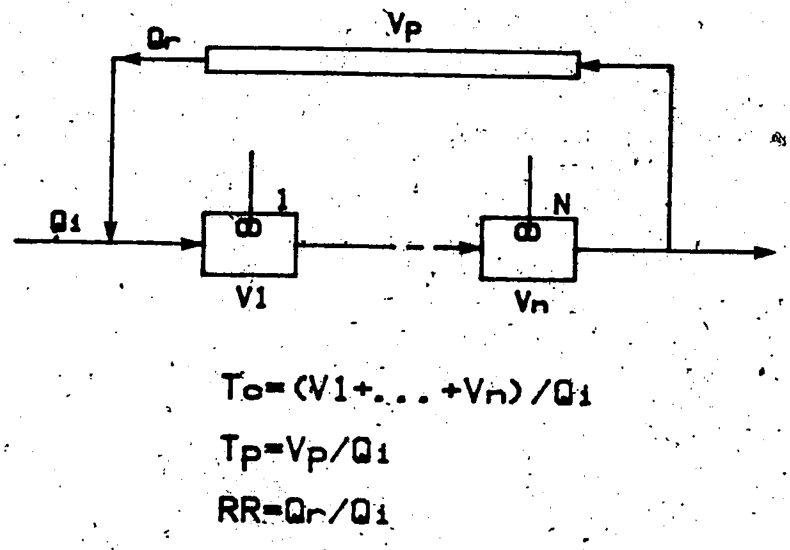


Figure 5.8 Schematic of Recirculation Model.

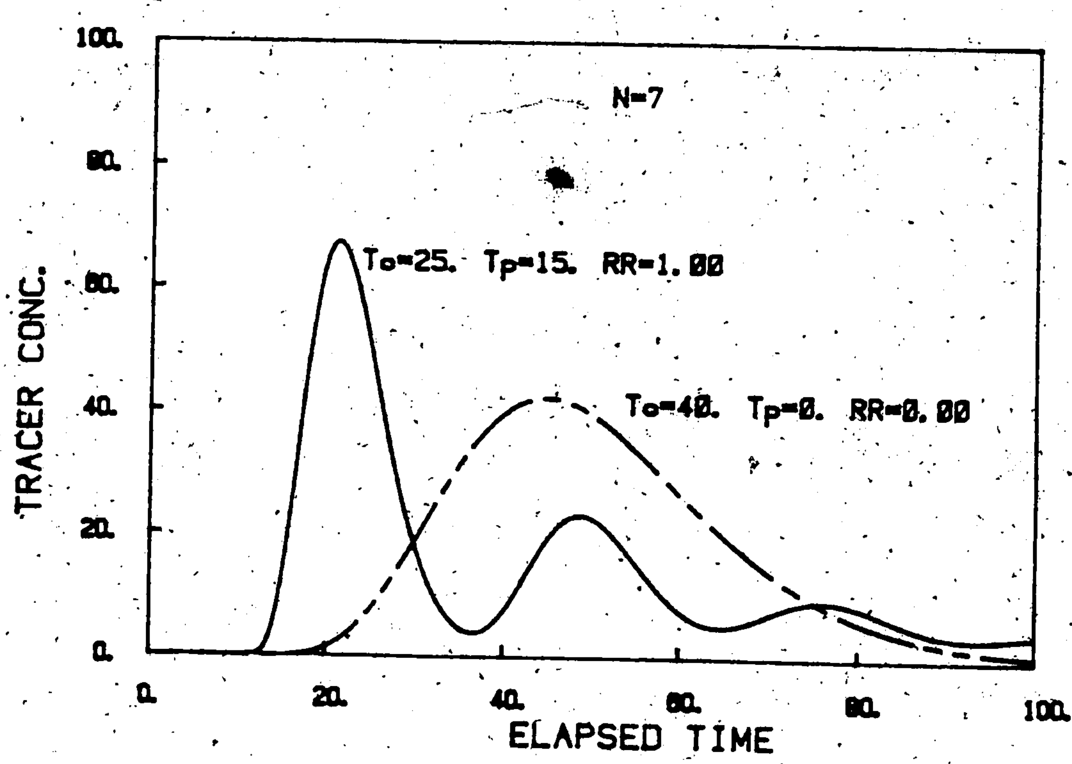


Figure 5.9 The Effect of Flow Recirculation on RTD Curves.

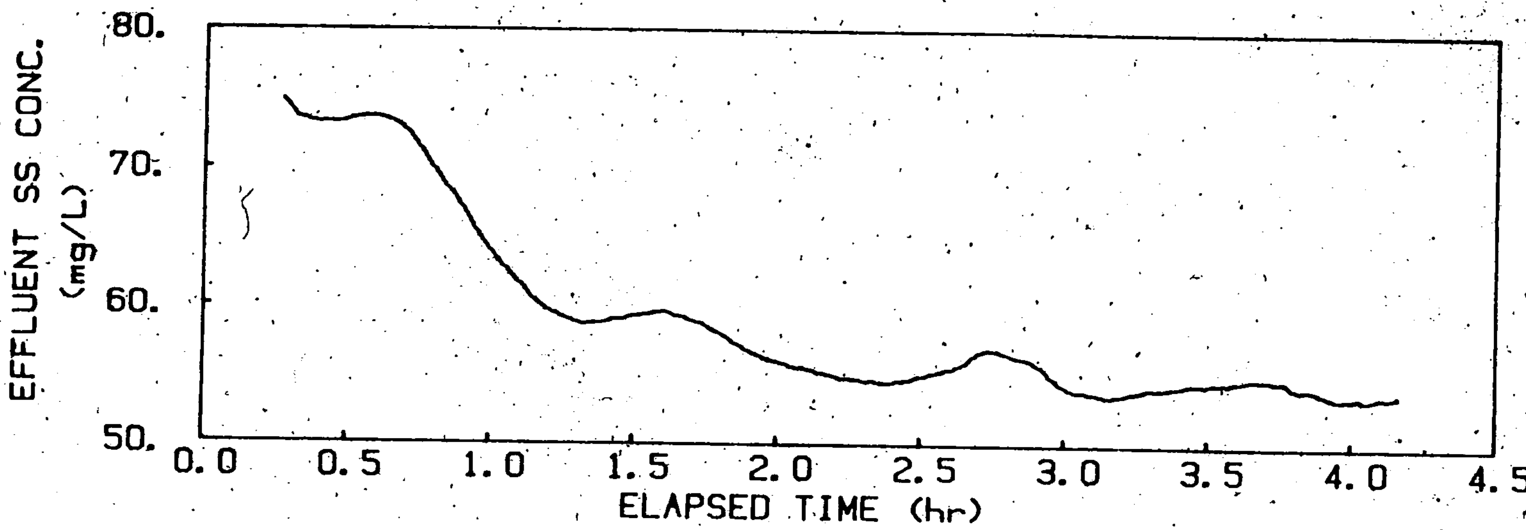


Figure 5.10(a) The Filtered Response of the Settler to a Step Decrease in Flow Rate (Decrease #2).

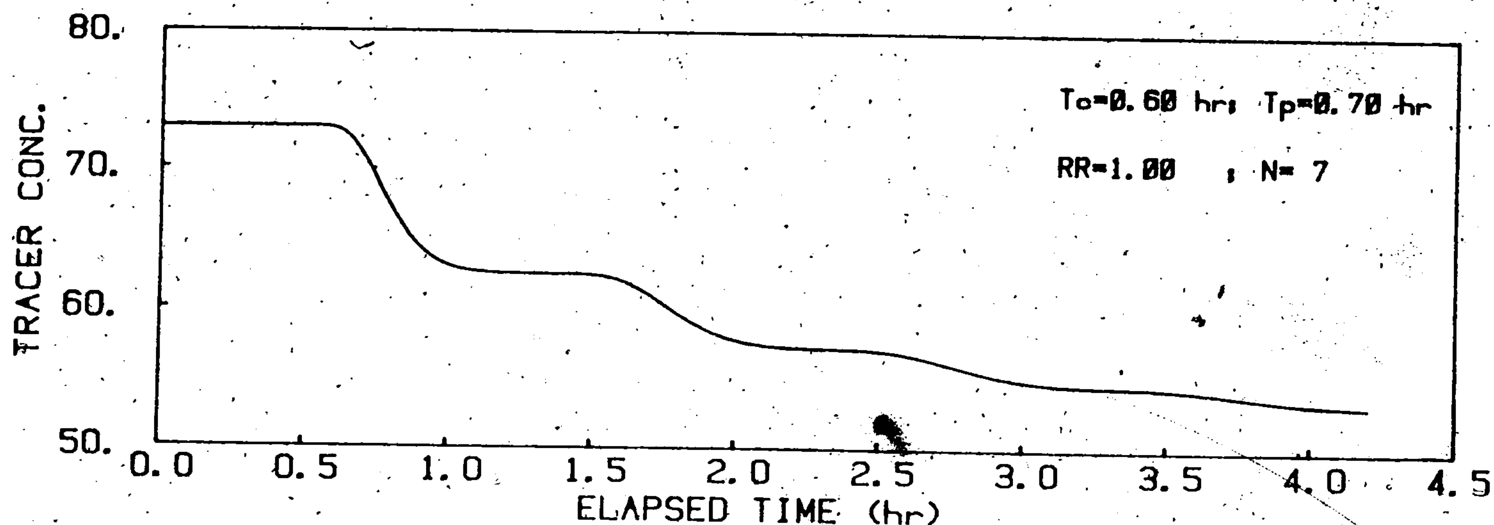


Figure 5.10(b) The Response of the Recirculation Model to a Step Decrease in Tracer Concentration.

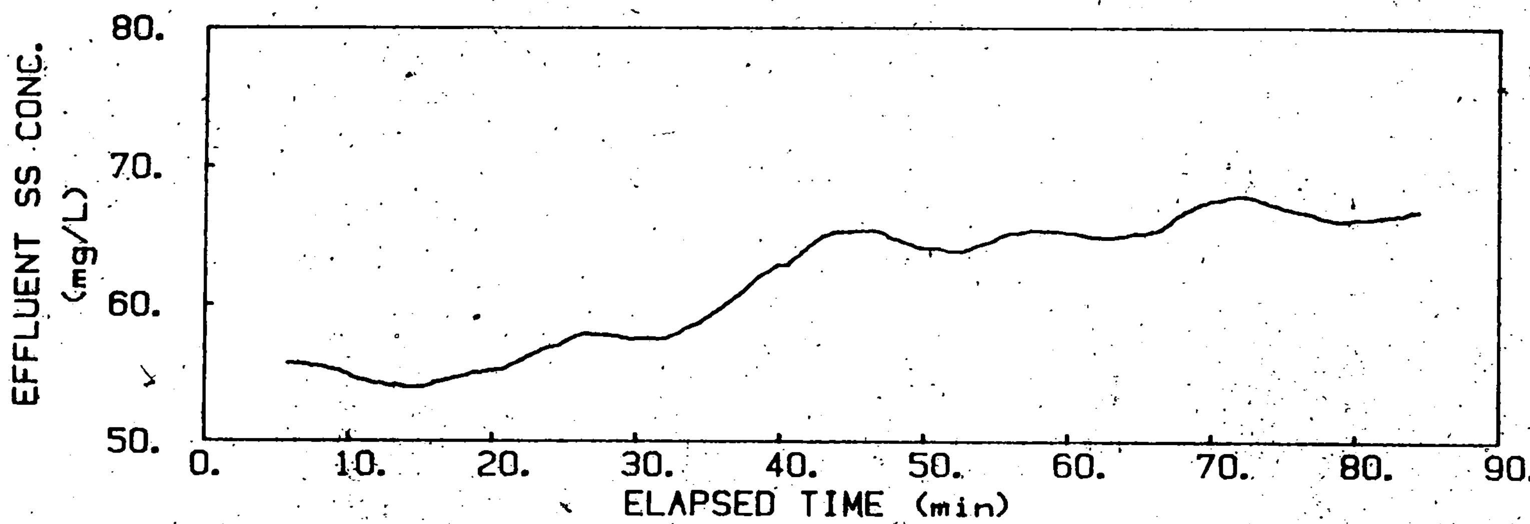


Figure 5.11(a) The Filtered Response of the Settler to a Step Increase in Flow Rate (Increase #2).

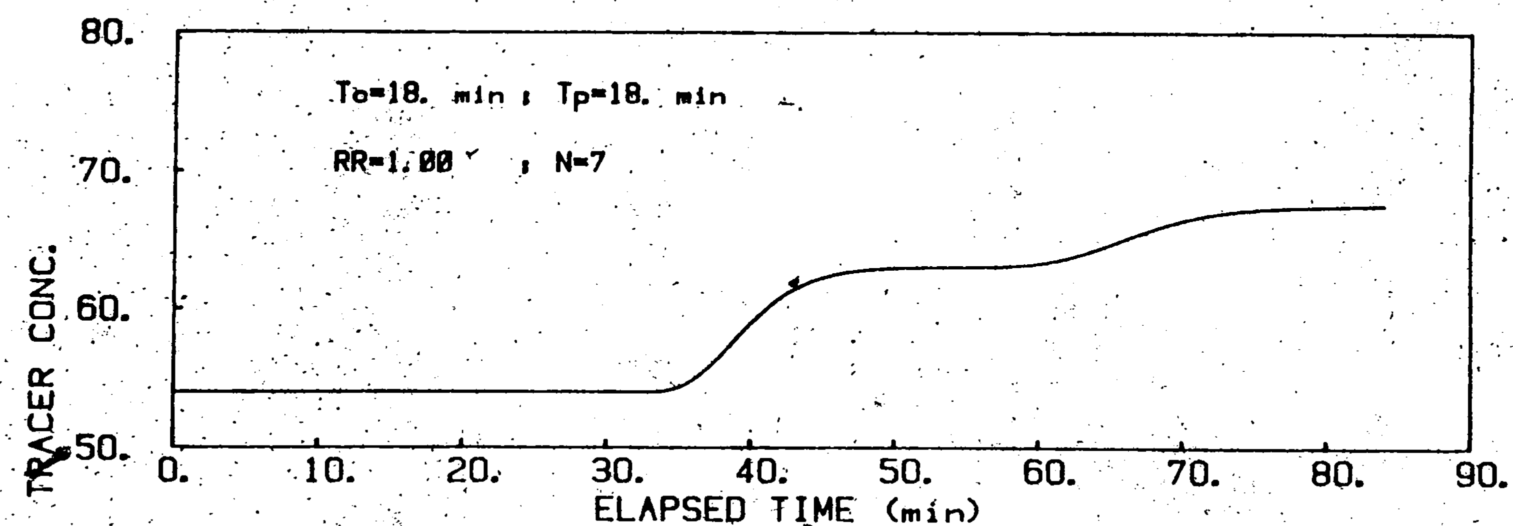


Figure 5.11(b) The Response of the Recirculation Model to a Step Increase in Tracer Concentration.

values. The plot of the model output is an exponential increase in tracer concentration with plateaus which attenuate with time. In contrast, the settler response shows a greater number of peaks with the peaks maintaining their identity much longer than predicted by the model.

The discrepancy between the two plots contained in Figure 5.11 indicates that the settler response to a step increase in flow rate cannot be explained simply in terms of an increase in the concentration of non-settleable solids entering the settler. To obtain a model output with a shape similar to the settler response of Figure 5.11(a), pulses as well as a step increase have to be fed into the recirculation model. This suggests that re-suspension is a factor which helps to determine the shape of the response of the settler following a step increase in flow rate. A more complex model would account for the influence of transient hydraulic conditions on particle re-suspension. For a short period after the step increase, the feed flow rate into the settler exceeds the effluent flow rate over the weirs. More momentum is transferred to the circular roll and the opportunity for particle re-suspension is enhanced.

5.3 Steady-State Data

5.3.1 General. Data was collected from runs for a total of three fractions of the design factorial. The design matrix for the first or principal fraction is presented in Table 3.3. The design matrix for the second fraction was generated by switching the signs in all of the columns of the matrix of the principal fraction. The third fraction was generated by switching the signs of the column for feed flow rate in the design matrix of the principal fraction. As for the principal fraction, blocking for the additional fractions was about sidewater depth.

A summary of the data collected from the runs for the three fractions is contained in Appendix E. In addition, Figure 5.12 presents the means and standard deviations of effluent suspended solids from the test clarifier. These turbidimeter values were plotted according to run sequence for the factorial runs. The mean effluent concentrations ranged from a low of 16 mg/L (Run #23) to a high of 79 mg/L (Run #19-R). The range of engineering variables covered by the factorials was as shown in Table 5.6.

The standard deviations for both factorial and centre-point runs were plotted against the means for the same run. The results, shown in Figure 5.13, indicated that, although the standard deviations and means are not highly correlated ($r_{xy} = 0.66$), there is a trend between the two values. As the mean effluent suspended solids concentration increased, the standard deviation tended to increase.

5.3.2 <u>Regression Analysis</u>. In order to develop a predictive equation for the solids concentration in the effluent from the test clarifier and to separate those variables which significantly influenced effluent clarity from those which did not, regression techniques were employed. The objective was to select a model with the maximum predictive power while avoiding "overfitting".

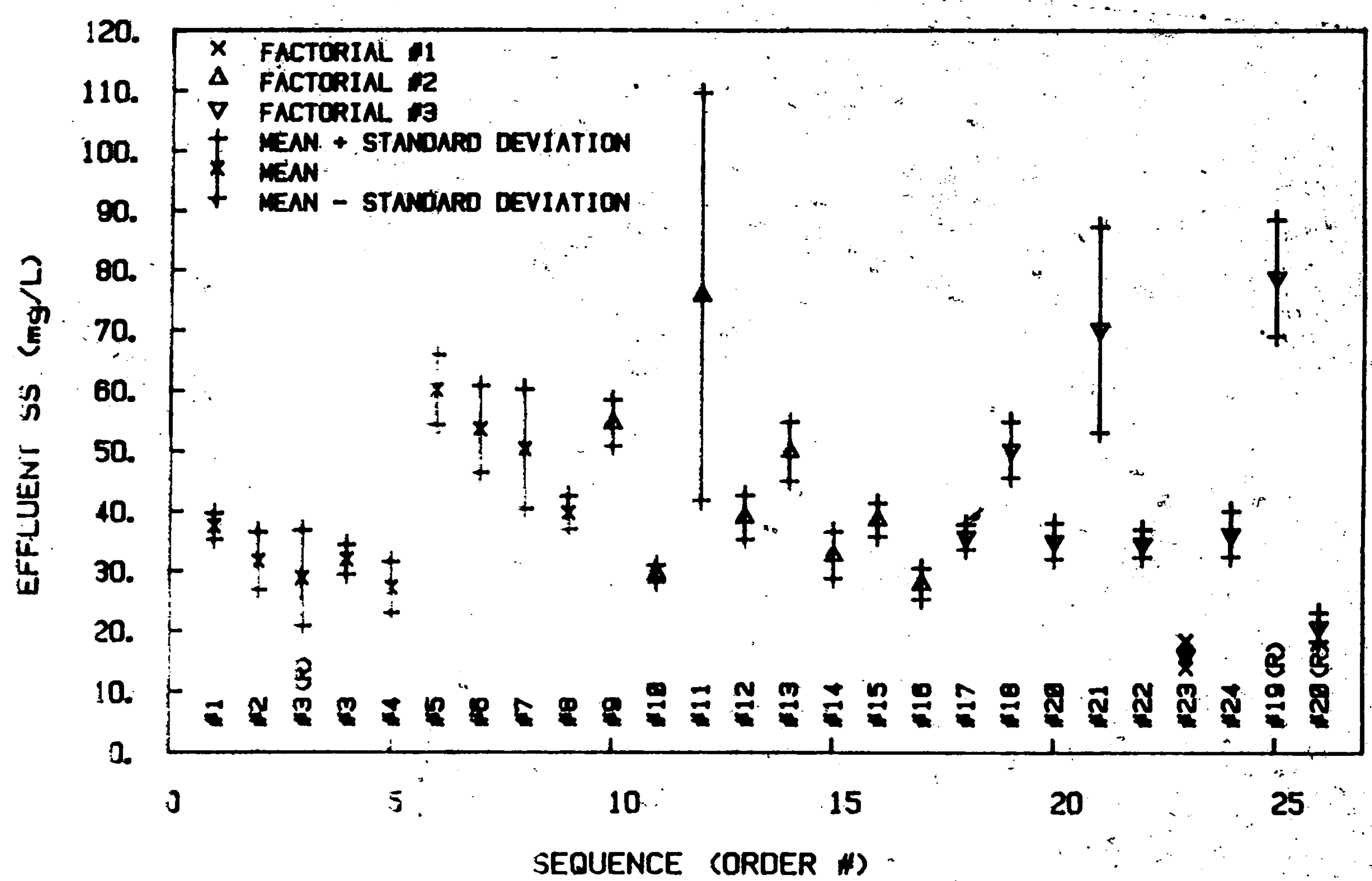


Figure 5.12 Factorial Results Plotted in the Order in Which They Were Collected.

TABLE 5.6 Range of Engineering Design and Operating Parameters

Parameter	Range
Overflow Rate	0.68 - 2.03 $m^3/m^2 \cdot h$
Hydraulic Detention Time	0.95 - 3.73 h
Solids Loading Rate	1.05 - 6.59 $kg/m^2 \cdot h$
Velocity Gradient	137 - 156 sec^{-1}

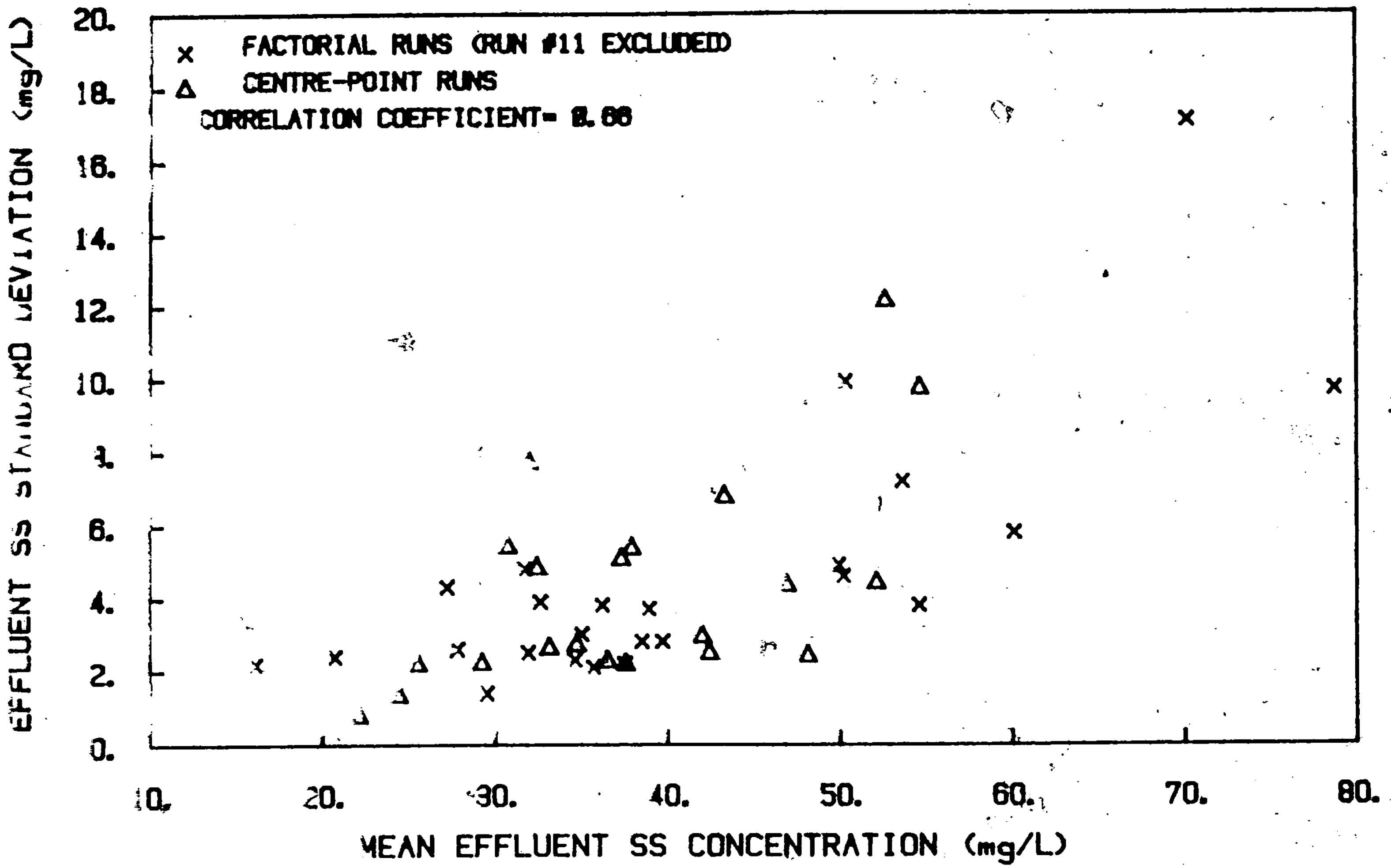


Figure 5.13 Effluent Standard Deviations Versus Effluent Means.

"Overfitting" results in models which are unjustifiably complex (Box and Jenkins, 1970).

A stepwise regression technique was employed to select an appropriate regression equation. A FORTRAN program, titled "BAKLM", was written on the Hewlett-Packard minicomputer. Starting with a predictive equation containing all the terms thought to influence effluent variablity, the program examined each term to determine its contribution to the predictive equation. Terms are eliminated one by one from the initial regression equation starting with the term with the lowest degree of predictive power. Decisions regarding the suitability of a term for inclusion or elimination from the predictive equation were made on the basis of a partial F-test. The partial F-test value was calculated assuming that the variable under investigation was the last to enter the regression equation. If the partial F-test value for the variable was greater than a tabulated F-value at a preselected level, then the elimination of terms from the regression equation was stopped and the regression equation at that stage adopted as suitable.

Details of the "Backward Elimination Procedure" are described by Draper and Smith (1966). In order to "debug" and test the program, a sample problem contained in Draper and Smith was solved using "BAKLM". The program output matched that contained in Draper and Smith. The output "BAKLM" and the output for the test data from Draper and Smith are presented in Appendix F.

The design of the three factorials allowed estimates to be made of the influence of the individual variables and their two-factor interactions with feed flow rate. In addition, two "dummy variables" were added to the starting regression equation. Changes in temperature or composition of the influent sewage over time would have resulted in differences in the average level of effluent clarity from one set of runs to another. The "dummy variables" ensured that differences in the overall level of effluent quality from factorial to factorial were removed. Therefore, the stepwise regression started with the terms contained at the top of Table 5.7 and progressed in accordance with the table. As a check, the multiple coefficient of determination was calculated for each regression in the process. The value served as a measure of how successful the particular equation was at explaining the variability in the effluent data.

The terms listed below the line in Table 5.7 were those which significantly influenced effluent clarity while those above the line did not. The appropriate regression equation therefore contained three variables - MLSS concentration, sidewater depth, feed flow rate - and the product of two variables (feed flow rate and sidewater depth). In statistical terms the product of two variables is known as a "two-factor interaction". Expressing flows as hydraulic loading per unit area of settler, the appropriate regression equation explaining 78 percent of the observed variability in effluent suspended solids from the test clarifier, was:

$$Ce = -180.6 + 4.0 \cdot MLSS + 135.6 \cdot Qa/A + SWD\,(90.2 - 62.5 \cdot Qa/A) \qquad (51)$$

TABLE 5.7 Results of Stepwise Regression of On-Line Data

Terms contained in starting regression equation:

Variables:

- MLSS
- Feedwell Depth
- Rake Speed
- Feed Flow
- Underflow
- Sidewater Depth
- Air Flow
- Intercept Term (b)
- "Dummy #1"
- "Dummy #2"

Interactions:

- MLSS/Feed Flow
- Feedwell Depth/Feed Flow
- Rake Speed/Feed Flow
- Underflow/Feed Flow
- Sidewater Depth/Feed Flow
- Air Flow/Feed Flow

Number of Terms	R^2 (%)	Partial F-Test	Variable Eliminated
16	83.1	0.04	Air Flow
15	83.0	0.07	"Dummy #2"
14	82.9	0.20	Rake Speed
13	82.6	0.24	Underflow/Feed Flow
12	82.3	0.27	"Dummy #1"
11	82.0	0.38	MLSS/Feed Flow
10	81.5	0.48	Rake Speed/Feed Flow
9	81.0	0.51	Air Flow/Feed Flow
8	80.4	0.58	Feedwell Depth
7	79.8	0.59	Feedwell Depth/Feed Flow
6	79.1	1.16	Underflow
5	77.9	4.58	
Stop Elimination!: F-value (95%) = 4.32			Sidewater Depth
4	73.1	10.21	MLSS
3	62.2	11.14	Sidewater Depth/Feed Flow
2	11.2	3.04	Feed Flow

where: Ce = effluent suspended solids concentration, mg/L

MLSS = concentration of suspended solids in the mixed liquor, g/L

Qa/A = clarifier feed flow rate per unit of surface area, m/h

SWD = sidewater depth, m

A = surface area of settler less areas accounted for by the feedwell and weirs, m^2

Graphical representations of the above equation were developed. Regression equation predictions were generated by holding constant the value for one of the three variables while changing in increments the values of the other two. As indicated in Figure 5.14, with sidewater depth constant, an increase in MLSS concentration or feed flow rate (due to a change in either plant inflow or recycle rate) resulted in a deterioration in effluent quality.

With MLSS concentration held constant, predictions of effluent quality were determined as feed flow and sidewater depth changed. As indicated in Figure 5.15, the lines were not parallel to one another reflecting the presence of the interaction between feed flow rate and sidewater depth in the predictive equation. Therefore, the relative deterioration in effluent quality for a given increase in feed flow rate was greater for a low sidewater depth than for a high one. For variables involved in an interaction, the response to changes in one of the variables depends on the level of the other variable.

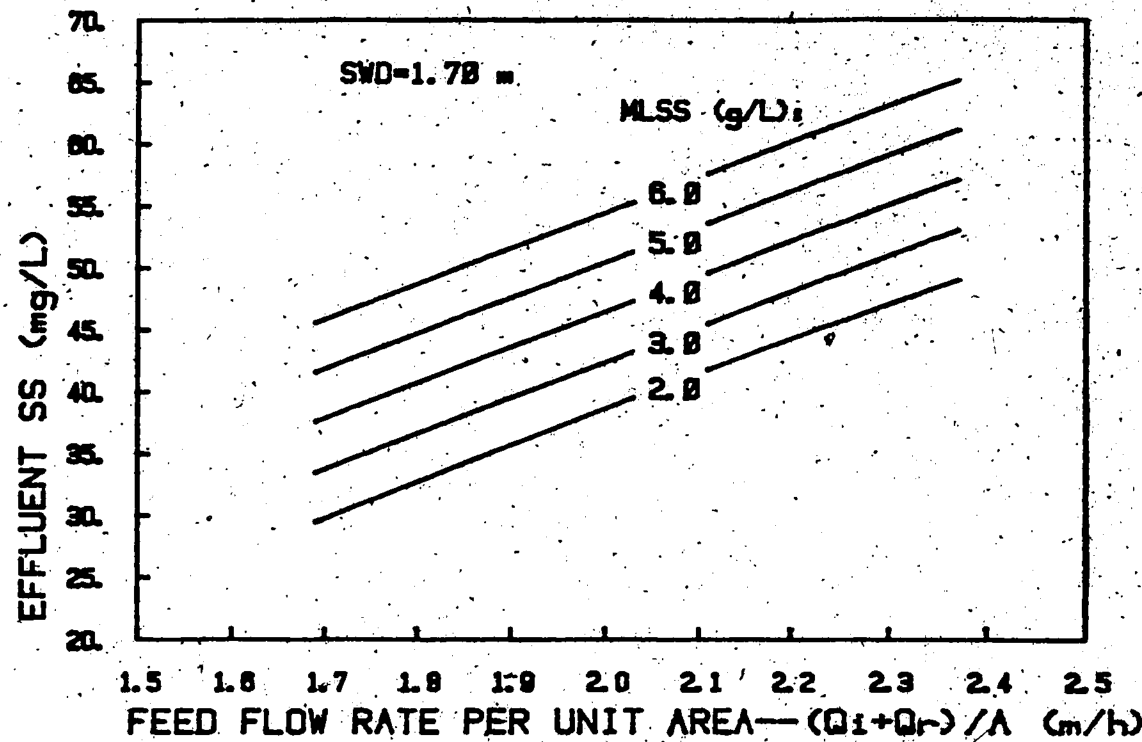


Figure 5.14 Regression Predictions With Sidewater Depth Constant.

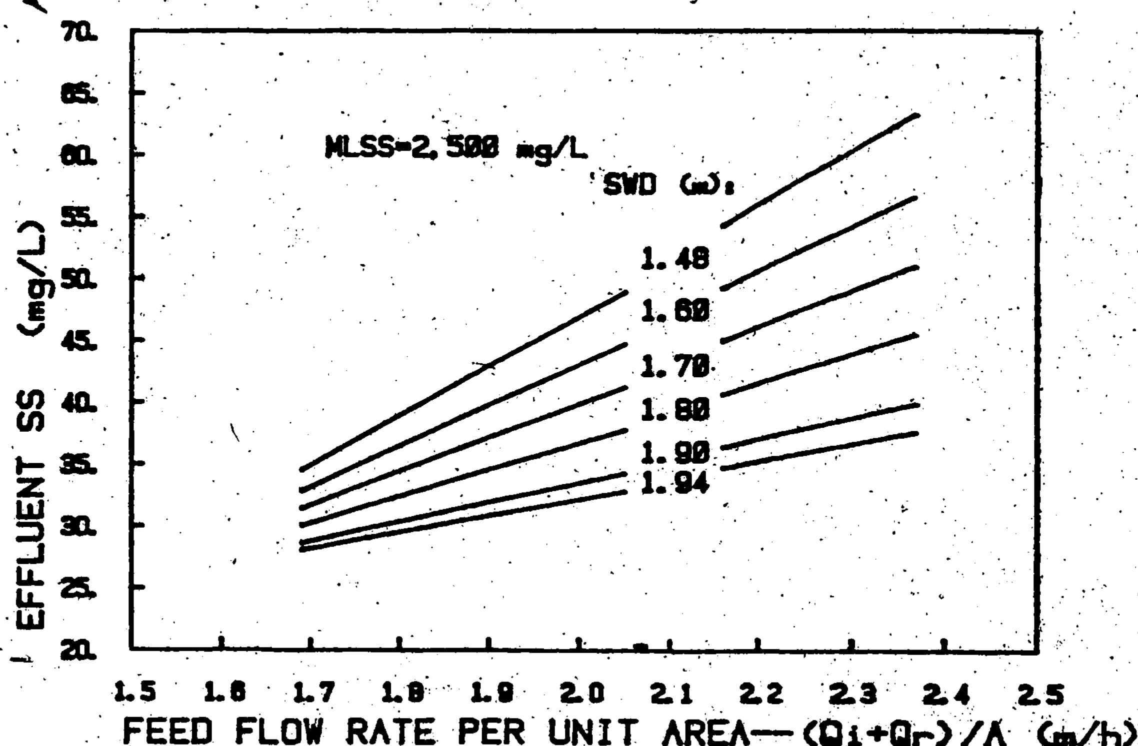


Figure 5.15 Regression Predictions With MLSS Concentration Constant.

5.3.3 <u>Diagnostic Model Checks</u>. The adequacy of the regression equation was checked by carrying out additional statistical tests and by analyzing the residuals.

In addition to partial F-values, program "BAKLM" also displayed the 95 percent confidence limits for the model coefficients. For the regression equation adopted, Table 5.8 presents these results. As zero is not contained in the confidence limits for these model coefficients listed in Table 5.8, the corresponding variables are seen to be significant. A "statistically significant" regression was obtained as the observed F-value for the model (also listed in Table 5.8) exceeded the reference F-value. In other words, the proportion of variation observed in the data which was accounted for by the regression equation was greater than could be expected by chance in similar data sets (Box <u>et al</u>., 1978). The observed F-value for lack-of-fit was less than the corresponding reference F-value. The model therefore displayed no lack-of-fit at the 95 percent confidence level.

A residual is the difference between the value observed at one set of conditions and the value predicted by the regression equation at the same set of conditions. Regression analysis assumes that errors in the observed response are independent and normally distributed with a zero mean and a constant variance. If these assumptions are correct and the model is adequate, the residuals will be normally distributed with a zero mean. The residuals were therefore examined in order to detect any gross inadequacies in the model and to determine if any of the unexplained variation in observed

TABLE 5.8 Confidence Limits for Regression Coefficients and Expanded Analysis of Variance Table

a) 95% Confidence Limits on Regression Coefficients

Term	Coefficient Values	95% Confidence Limits
MLSS Concentration	4.03	1.62 - 6.45
Feed Flow Rate	135.60	60.26 - 210.95
Sidewater Depth	90.16	2.49 - 177.83
Feed Flow Rate/ Sidewater Depth Interaction	-62.54	-106.20 - -18.89

b) Expanded ANOVA Table

Source	SS	DF	MS	F
Model	5064.46	4	1226.11	18.54(1)
Residual	1433.92	21	68.28	
Lack-of-fit	1067.02	13	82.08	1.79(2)
Pure error	366. 0		45.86	
Mean	45343.84	1		
Total (corrected)	6498.38	25		
Total (uncorrected)	51842.22	26		

Tabulated F-Values

(1) $F[4,21,95\%] = 2.84$

(2) $F[13,8,95\%] = 3.26$

effluent clarity could be attributed to other variables which were measured (but not controlled) during experimentation.

The methods used to analyze the residuals were graphical (Draper and Smith, 1966). The residuals were plotted in the following ways:

a) in time order;

b) against the predicted values obtained from the adopted regression equation;

c) as frequency histograms - for all the runs and for each of the three fractions of the factorial; and

d) against the following measured values: liquid temperature, zone settling velocity, oxygen uptake rate of the mixed liquor and the concentration of suspended solids in the influent to the pilot plant.

The residual plots are contained in Figures 5.16 to 5.22. They display no gross discrepancies from the assumption that the residuals are normally distributed with a zero mean. Therefore, there is no reason to reject the adequacy of the regression model developed using the backward elimination technique. Further, none of the unexplained variation in effluent suspended solids could be attributed to variations in liquid temperature, zone settling velocity, oxygen uptake rate or influent suspended solids concentration. (The previous statement should not be interpretted as meaning that there is no relationship between liquid temperature, zone settling velocity and other measured (but uncontrolled) variables and the quality of effluent secondary settlers. Rather it means that no influence could

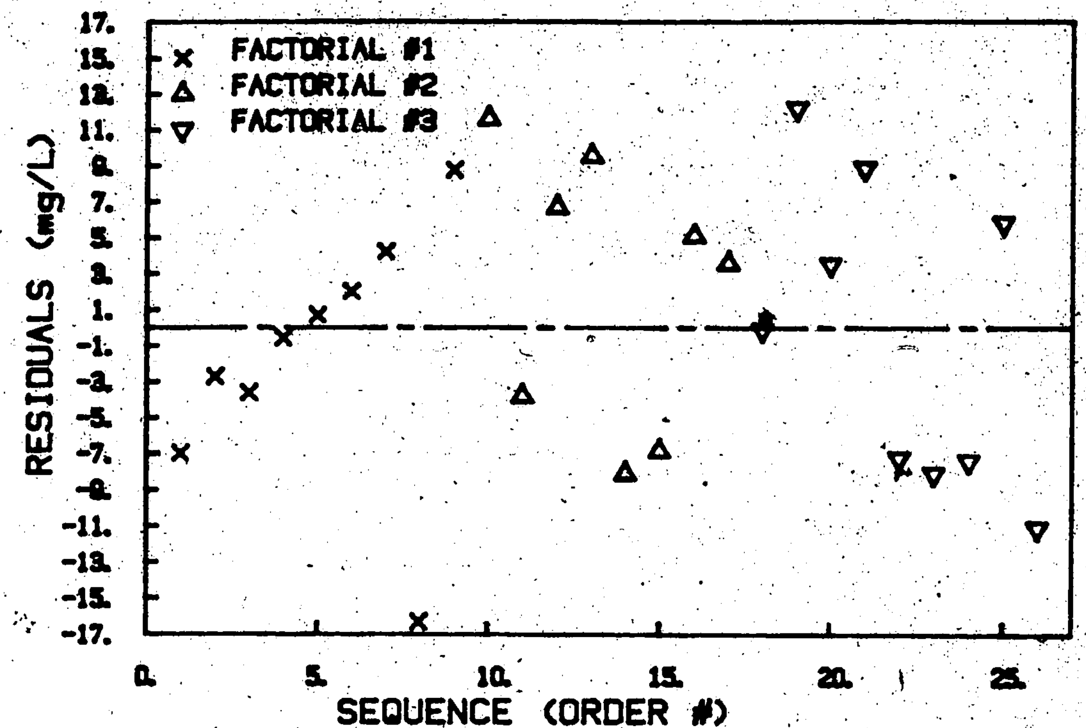


Figure 5.16 Examination of Residuals: Residuals Plotted in the Order in Which They Were Collected.

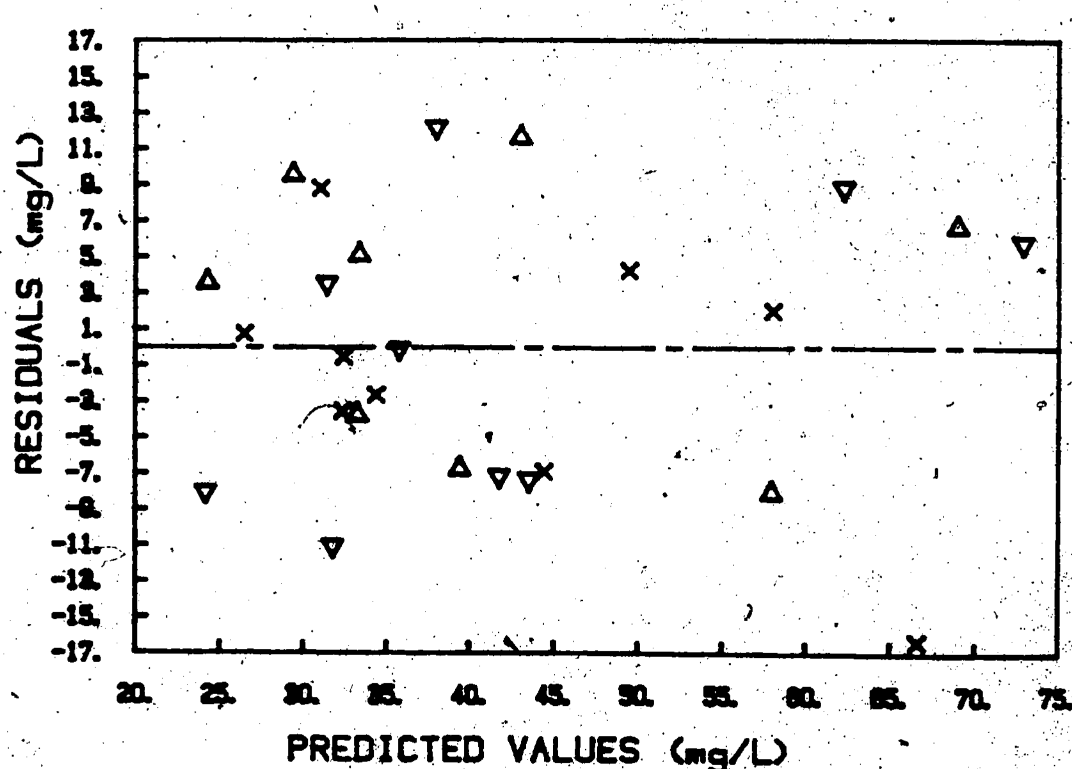


Figure 5.17 Examination of Residuals: Residuals Versus Regression Predictions.

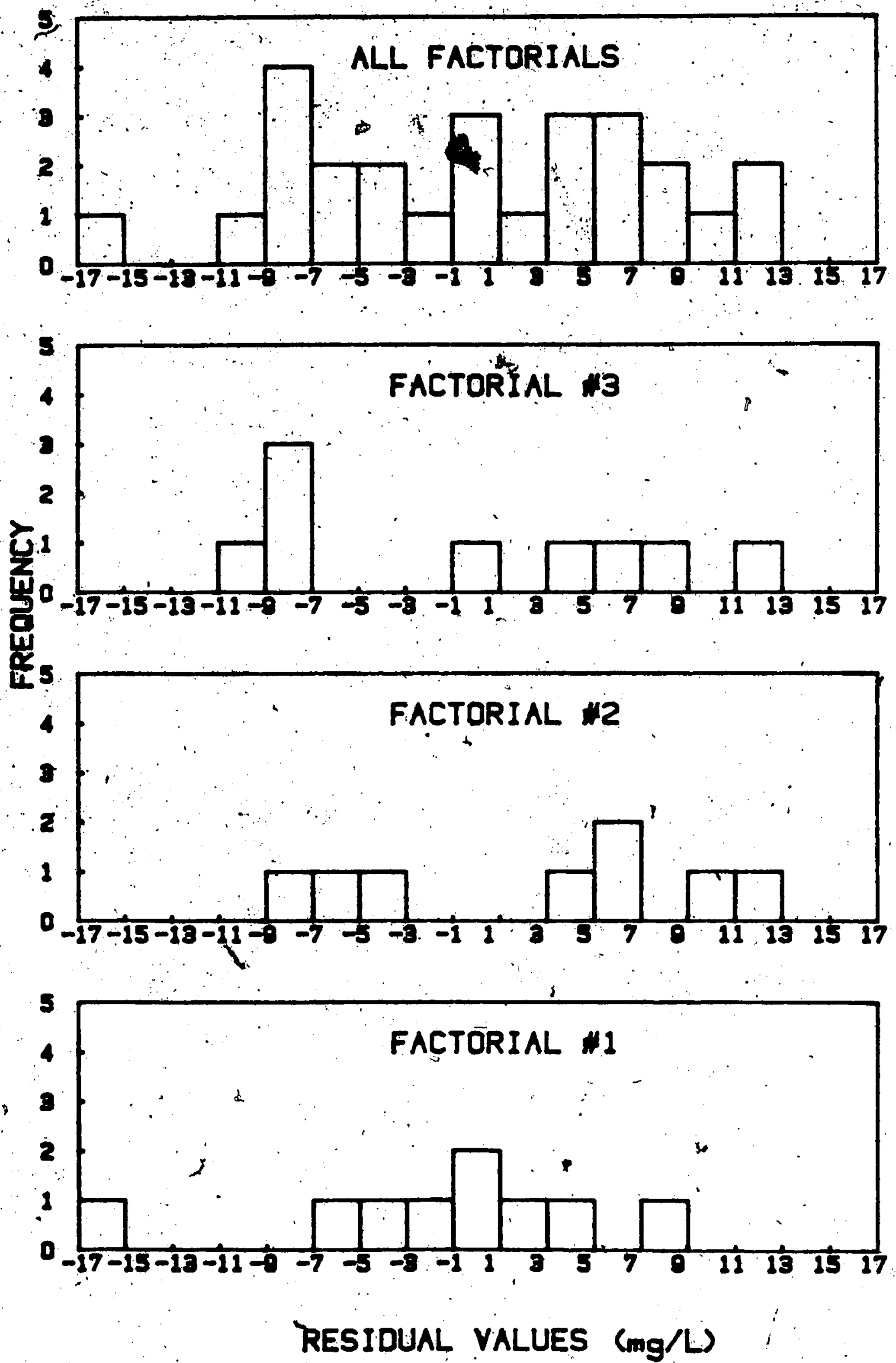


Figure 5.18 Examination of Residuals: Histograms of Residuals

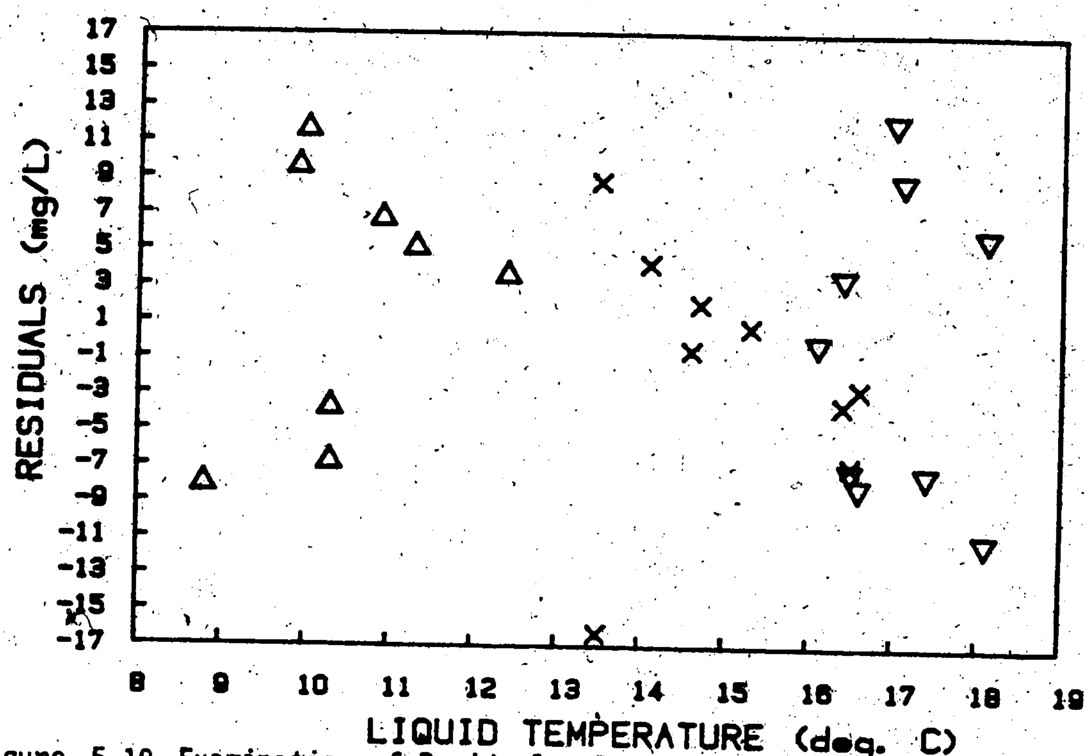


Figure 5.19 Examination of Residuals: Residuals Versus Process Liquid Temperature.

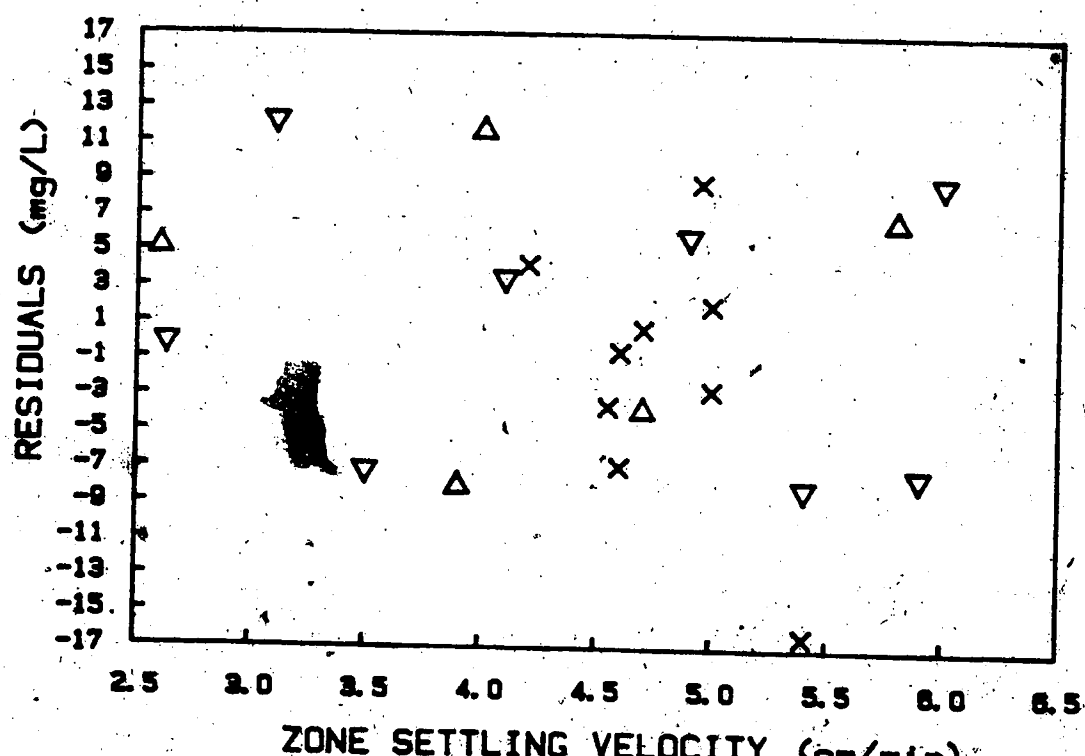


Figure 5.20 Examination of Residuals: Residuals Versus Zone Settling Velocity.

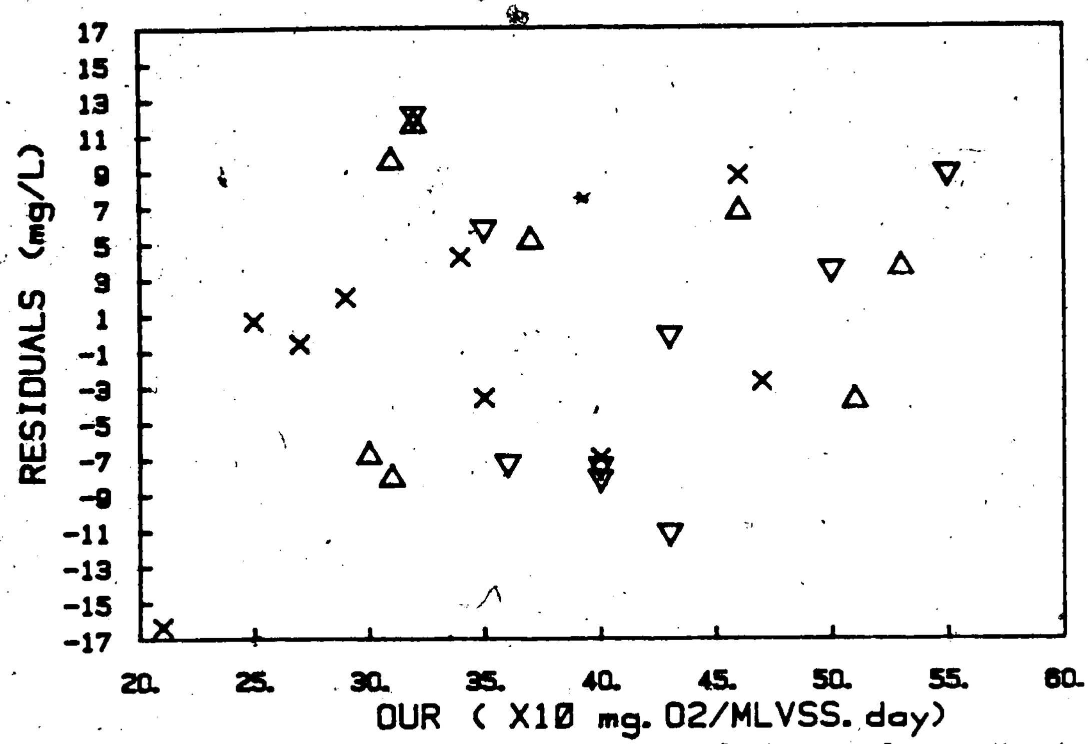


Figure 5.21 Examination of Residuals: Residuals Versus Oxygen Uptake Rate.

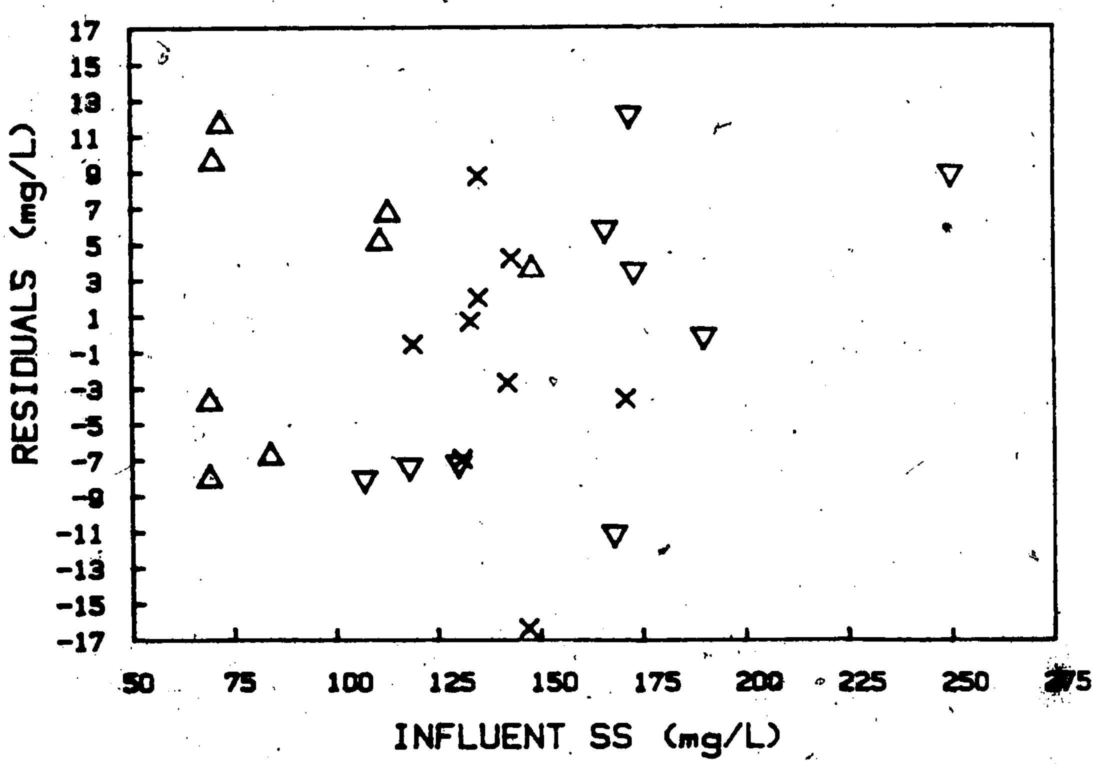


Figure 5.22 Examination of Residuals: Residuals Versus Influent Suspended Solids Concentration.

be discerned given the changes in those variables which occurred by chance during the course of the experimentation. As succinctly stated by Box et al., (1978): "To safely infer causality the experimenter cannot rely on natural happenings to choose the design for him; he must choose the design for himself....")

5.3.4 Comparison with Other Research. To establish the credibility of the findings of this study, these results were compared with those obtained by Pflanz (1969), Fitch (1957), Tuntoolavest et al. (1980), and Parker (1982).

Pflanz observed that the concentration of suspended solids in the effluent from the three full-scale clarifiers increased as the MLSS concentration increased. The rate of this increase was measured by Pflanz and Table 5.9(a) presents his observations for a rectangular settler with a depth of 1.2 m.

As indicated from the stepwise regression analysis, MLSS concentration was found to be an important variable with effluent suspended solids concentration from the test clarifier increasing by approximately 4 mg/L for each g/L increase in MLSS concentration. Table 5.9(b) presents measured values averaged and grouped according to flow conditions.

The efficiency of the test clarifier was compared with that observed by Pflanz at the plant at Celle, which had a circular clarifier 33 m in diameter and 2.3 m in depth. For a 30 mg/L effluent suspended solids concentration, permissible loading rates for both the Celle clarifier and the test clarifier were plotted. As shown in

TABLE 5.9 Comparison of Influence of MLSS Concentration on Effluent Suspended Solids Concentration

(a) Estimates for the Pflanz Data for the Final Tanks at Bennigsen (After Pflanz, 1969)

MLSS Range (g/L)	Overflow Rate (m/h)	Increase in Effluent SS per g/L Increase in MLSS (mg/L)
2.0 - 6.0	0.89	5 - 6
2.0 - 4.0	1.00	7
4.0 - 6.0	1.00	10 - 11

(b) Results from "Test" Settler at "+" Sidewater Depth

Flow FF (L/min)	Rates UF (L/min)	Overflow Rate ($m^3/m^2 \cdot h$)	Average Increase in Effluent SS per g/L Increase in MLSS (mg/L)
100	60	0.68	5.5
140	60	1.36	2.5
140	20	2.03	3.5

Figure 5.23, the permissible rates were higher for the full-scale clarifier than for the pilot-scale test clarifier. Over the range of loadings common to both studies, the relative change in effluent quality was similar for the two clarifiers. For instance, for the Celle clarifier, an increase in overflow rate from 0.85 to 1.15 m/h required that MLSS concentration decrease by 0.9 g/L in order to maintain a 30 mg/L effluent. For the test clarifier, the same change in overflow rate would require a decrease of 1.1 g/L in MLSS concentration. Therefore, given the differences in areas and depths between the clarifiers studied by Pflanz and the test clarifier employed in this series of experiments, there was reasonable agreement regarding the influence of solids and hydraulic loading.

Conclusions differed, however, regarding the importance of recycle rate. Pflanz found that variations in return sludge flow did not influence effluent quality. For the test clarifier, the equipment was configured so that the rate at which flow was pumped into the settler could be controlled independently of the rate at which underflow was pumped out. For the conventional activated sludge system, a feedback loop exists between the settler recycle and feed flow rates. By increasing the recycle rate the underflow and feed flow rates are automatically increased. The results from the factorial analysis indicated that an increase in underflow rate did not significantly influence effluent quality whereas an increase in feed flow rate deteriorated effluent quality. Because an increase in feed flow rate ensues automatically from an increase in recycle rate for conventional

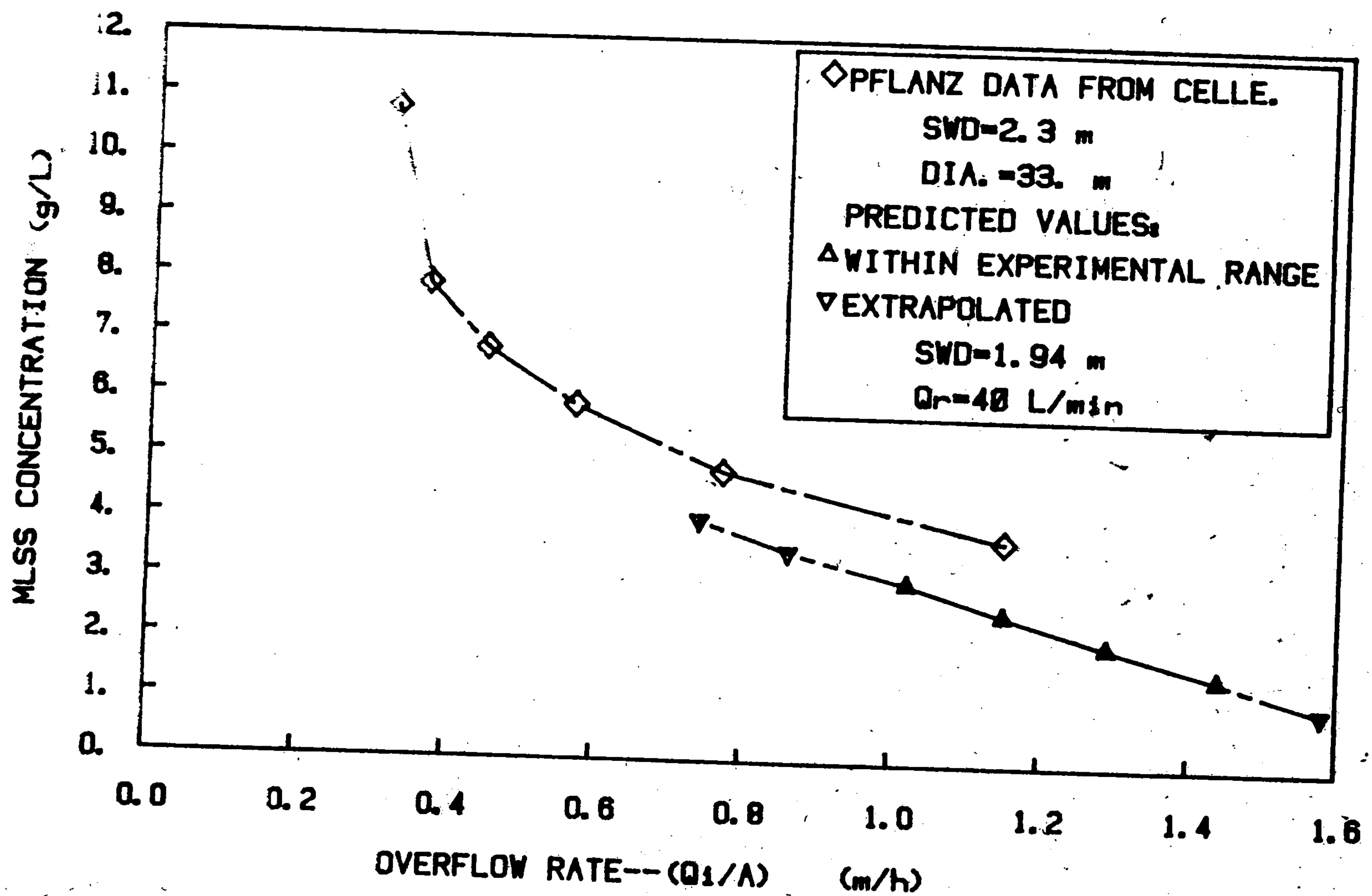


Figure 5.23 Comparison of Loading Rates to Achieve a 30 mg/L Effluent--Pflanz Data and Regression Predictions.

systems, these results suggest the net effect of such an increase is a deterioration in effluent quality.

Garrett <u>et</u> <u>al</u>. (1983) also studied the effect of recycle rate on effluent quality. At the City of Houston's Southwest Wastewater Treatment Plant, they reduced the recycle rate from 100 percent to 50 percent of the inflow rate. As a result, the effluent suspended solids concentration from the final settler decreased from 10.2 mg/L to 6.9 mg/L. These results, in contrast to those of Pflanz, support the conclusion that recycle rate has the potential to influence effluent quality.

Fitch (1957) studied the removal efficiencies using settling columns and a suspension of calcium carbonate. His results were plotted (Figure 5.24) in conjunction with a similar plot (Figure 5.24), developed from the test clarifier predictions expressed as percentage removal. Both sets of plots indicated that the relationship between removal efficiency and overflow rate was linear, or nearly so, and, that, as lines for both sets were not parallel to one another, an interaction was involved. To confirm the existence of the interaction, Fitch's data was subjected to the backward elimination technique. The results were as indicated in Table 5.10.

As can be seen, column height, overflow rate and the interaction between them could not be eliminated from the regression equation. The three terms significantly influenced effluent clarity. Analysis of the Fitch data therefore provided strong support for the conclusions that both overflow rate and depth were important

Figure 5.24 (a) was removed due to copyright restrictions. The figure, obtained from a paper by Fitch (1957), showed the relationship between removal efficiency and overflow rate for different settling column heights.

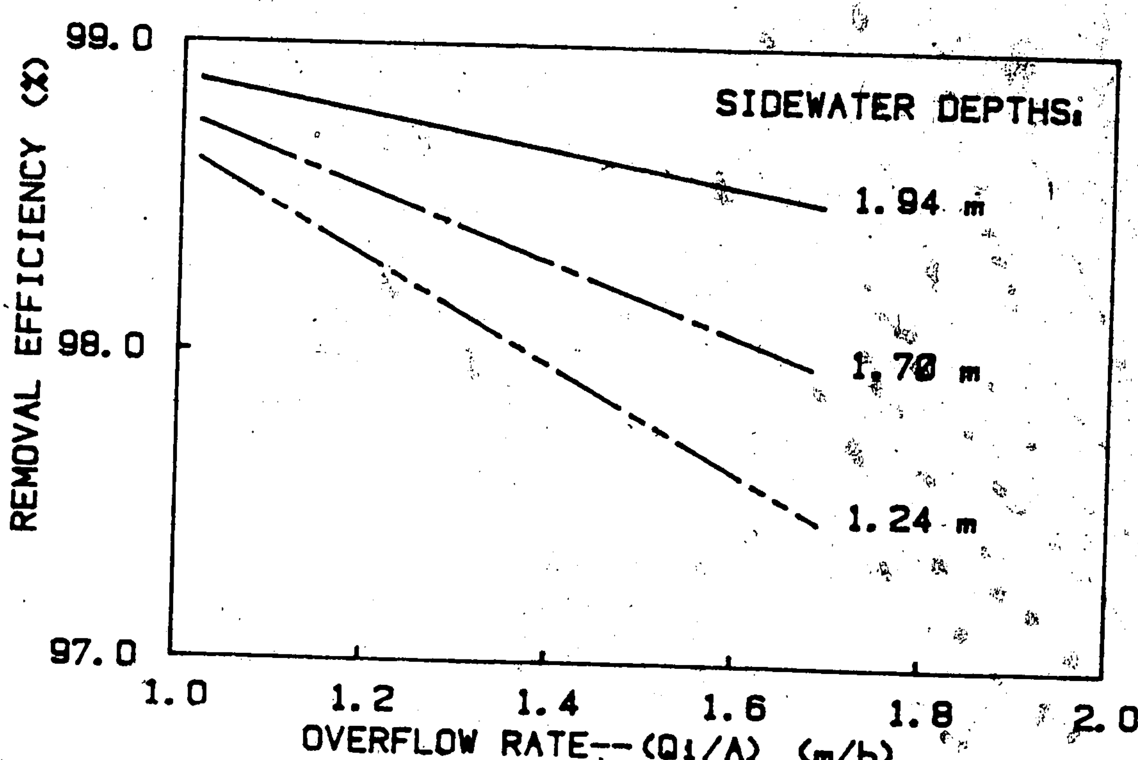


Figure 5.24 (b) Removal Efficiency Versus Overflow Rate for Regression Predictions.

TABLE 5.10 Results of Stepwise Regression for Fitch Data

Terms contained in starting regression equation:

Variables:
- Column Height
- Overflow Rate
- Intercept Term (b)

Interactions:
- Overflow Rate/Column Height

Number of Terms	R^2 (%)	Partial F-Test	Variable Eliminated
4	96.8	5.03	
Stop Elimination!: F-value (95%) = 4.28			
			Column Height
3	96.1	321.17	
			Overflow Rate/Column Height
2	43.9	19.54	
			Overflow Rate

clarification variables and that there is indeed an interaction between them.

A finding of the research conducted by Tuntoolavest et al. (1980) was that the MLSS concentration exerted a major effect on the concentration of suspended solids in the clarifier effluent. As well, they determined that 87 percent of the variability in effluent clarity was accounted for by the air flow rate/MLSS concentration interaction, thereby suggesting that velocity gradients at high air flow rates were shearing the biological flocs. Although the results from the test clarifier confirmed the importance of MLSS concentration, they did not provide evidence for the importance of air flow rate. Air flow rate was one of the first variables dropped from the regression equation when using the backward elimination procedure.

The differences between the two studies can be attributed to differences in the range of variables, the use of tapered aeration and to fundamental problems with the measurement of turbulence. For the research conducted by Tuntoolavest et al., changes in the air flow rates created average shear of G-values which ranged from 162 to 200 sec^{-1}. Due to the limited capacity of the blowers on the activated sludge pilot plant, the G-values for this study were both lower and varied over a narrower range, from 137 to 156 sec^{-1}.

The G-values for this study were calculated using measurements of the total air flow rate delivered to the three aeration tanks. Tapered aeration was used to match the air supply to the oxygen demand as exerted by the micro-organisms. As a result, the valves on the air lines to compartments "B" and "C" were throttled back in comparison to compartment "A". Therefore, the G-value in compartment "C", establishing the hydraulic environment of the mixed liquor prior to its discharge to the test settler, was actually lower than the G-value based on total air flow rate to all three aeration compartments. With only one air flow meter located on the discharge from the blowers, G-values were based on total air flow rate.

A fundamental problem in examining the influence of air flow rate on floc shear has to do with the fundamental nature of turbulence and how it relates to the average shear velocity. Turbulence consists of highly unsteady eddies or rotational flow elements. The largest eddies have the lowest frequency fluctuations in velocity and have a size which is of the same order of magnitude as the flow domain as determined by the walls and floor of the tank and the liquid surface. Energy from large eddies is transferred to smaller and smaller eddies, a process known as "energy cascade" (Rodi, 1980). The smallest eddies, with the highest frequencies, dissipate the mechanical energy into heat through viscous forces. The mechanical energy imparted to a fluid is distributed to eddies with a range of frequencies.

The average shear velocity is a measure of the energy input per unit volume. In essence, it represents an infinite combination of turbulence scale and intensity measures (Kalinske, 1971). In determining floc shear, interest centres on how much energy is contained in those eddies which are of a size to disrupt the flocs. Eddies which are of the same order of magnitude as the flocs - from approximately 100 μ to 1000 μ - can shear the flocs into smaller flocs or strip away primary particles. In contrast, large-scale eddies simply transport the floc from one place to another in the aeration basin. Determining the effect of aeration equipment and air flow rates in floc shear requires a knowledge of the distribution of turbulent energy amongst the various eddy frequencies. The average shear velocity in this context is simply not an adequate characterization of the nature of turbulence.

Direct evidence for the influence of sidewater depth on effluent quality at full-scale plants was presented in a recent paper by Parker (1982). He listed conditions which he believed would optimize suspended solids removal in the activated sludge system. With respect to the aeration tank, desirable design and operating procedures are those which lead to lower mean velocity gradients and hence produce fewer primary particles. For clarifiers, Parker recommended the use of deep tanks, low overflow rates, inboard weirs, and large diameter feedwells.

In order to demonstrate the importance of sidewater depth in clarifier design, Parker obtained historical operating records from a number of plants designed by his firm. The results, reproduced on Figure 5.25, indicated that the average concentration of suspended solids in the effluent decreased as the depth of the tank increased. As well, process stability, represented by the spread of the 10-percentile and 90-percentile values, improved with depth.

Parker's observations regarding sidewater depth are completely consistent with those obtained from the test clarifier. The regression equation developed from the factorial results predicted improved effluent clarity with increased depth. An unpaired Student's

Page 181 was removed due to copyright restrictions. The page contained Figure 5.25, obtained from a paper by Parker (1982), showing the effect of depth on effluent suspended solids concentration.

t-test was carried out to compare standard deviations for runs conducted with the test clarifier at a high depth to the standard deviations for runs at a low depth. The resulting value of the t-statistic (2.75) was significant at the 95 percent confidence level indicating that process stability was better at the higher depth. However, it should be recalled from Figure 5.13 that there was a general relationship between the mean level of suspended solids for a run and the standard deviation of the same run. Therefore, process stability is not simply a function of sidewater depth alone. The test clarifier research indicated that those conditions which reduced the mean level of effluent suspended solids also tended to improve process stability.

5.3.5 <u>Discussion of Steady-State Results</u>. A conceptual model commonly employed in sanitary engineering visualizes clarification as a process consisting of a single particle settling in a column of water. The particle attains a constant velocity as the force due to the weight of the particle in water is balanced by the drag force. If the flow of water around the particle is laminar and the particle spherical, Stokes's Law will predict its velocity. If Stokes's Law applies, settler overflow rate and water temperature are the variables which determine removal efficiency.

From the results of this set of experiments and the results and observations of other researchers, a more complex and less easily quantifiable picture of the clarification process emerges. The clarifier contains two zones delineated by the sludge blanket interface. Above the interface there is considerable turbulence. Particles are removed from suspension by sedimentation, flocculation and entrapment in the sludge blanket (Gregory, 1979). Below the interface, flow conditions are quiescent and sludge is thickened. The settling and flocculation properties of the suspension are determined by the biological nature of the flocs. These properties are influenced by conditions in the aeration tanks and vary with time.

Turbulence in the settler is created by the existence of a density current. As described in the literature, the incoming flow plunges down from the feedwell until it is deflected laterally by the sludge blanket. The flow then moves as a thin sheet or "intensive flow zone" across the top of the blanket. It is turned upwards toward the weirs by the walls of a settler. A counter-current is induced in the upper levels of the tank with fluid moving back toward the influent. Velocities in the "intensive flow zone" are high. Anderson (1945), for instance, measured velocities of the order of 4.8 m/min. In contrast, no measurable velocity was found in the sludge blanket.

A number of forces combine to create the circular roll in secondary clarifiers. Flow is accelerated by the difference in density between the incoming flow which has a high suspended solids concentration and the clarified liquid which has a low concentration. Temperature differences between the two liquids increase the density effect. As demonstrated by Murphy (1964), even in the absence of density differences, the vertical roll in secondary settlers is created by the momentum of the incoming flow.

The presence of turbulence in the settler means that, superimposed on the mean velocity of the fluid in the "density current", there are velocity fluctuations or "gusts" of varying size and intensity in all directions (Smith, 1975). Providing that the scale and intensity of the turbulence is within certain limits, the flocculation of particles is promoted. However, vertical velocity fluctuations will maintain particles in suspension and prevent removal by settling. As well, erosion from the sludge blanket of previously settled particles is enhanced as the intensity of the turbulence increases.

The suspension entering the secondary settler is biological, flocculent and its settling properties vary over time. The rate at which the suspension settles and compacts is governed by the proportion of filamentous to zoogleal bacteria in the flocs. A floc which is largely composed of filaments settles and compacts very slowly. As a filamentous floc has a large surface area to volume ratio, it tends to sweep the fluid clean of smaller particles as it settles. Exopolymers, produced by the bacteria, promote flocculation through the action of polymer bridging. In addition to flocs, the suspension contains colloidal particles which are too small to settle. These colloids are mainly cell walls and other cellular debris (Rickert and Hunter, 1972). If not incorporated into a floc, these primary particles pass through the settler and contribute to the concentration of suspended solids in the effluent.

The conceptual model incorporating flocculation and turbulence is useful in explaining the results obtained from the steady-state experimentation. For instance, MLSS concentration was observed to significantly influence the concentration of suspended solids in the effluent. There are three possible reasons for this. Firstly, if the proportion of nonsettleable cellular debris in the mixed liquor remains constant for a given set of conditions in the aeration tanks, increasing the MLSS concentration will increase the rate at which these particles enter the settler. Secondly, the density difference between the incoming flow and the clarifier liquid increases. Thirdly, the thickness of the sludge blanket in the settler increases as the solids flux loading increases. The volume of clarifier liquid above the blanket decreases accordingly. Opportunities for the flocculation of dispersed particles are diminished. Therefore, at higher MLSS concentrations, the settler must handle more nonsettleable particles in a smaller clarification volume at higher levels of turbulence. The net effect is a deterioration in effluent quality.

Similar arguments can be used to explain the importance of feed flow and sidewater depth in influencing removal rates. An increase in feed flow rate was observed to result in a poorer quality effluent. With the change in flow into the settler, the momentum of the "density current" in the tank was increased. Additional energy was therefore available for mixing the contents of the clarified zone and for keeping particles in suspension. Opportunities for erosion of particles from the sludge blanket were enhanced. Finally, the rate at which nonsettleable particles entered the settler was increased with an increase in feed flow rate.

As the depth of the settler increased, with all other conditions remaining constant, the concentration of effluent solids decreased. The increase in depth resulted in a larger volume of clarifier liquid above the sludge blanket interface. The perimeter of the "density current" was expanded therefore the length of time a particle remained in the settler was increased enhancing opportunities for removal due to flocculation. The importance of the sidewater depth in influencing clarification provides strong evidence that flocculation is a major mechanism promoting solids removal. As well, with additional distance between the top of the sludge blanket and the effluent weirs, there was less opportunity for disturbances generated at the interface to reach the weirs.

CHAPTER 6

SIGNIFICANCE TO DESIGN AND OPERATION

Settler design and operating procedures are effective if they lead to a net benefit in total system performance. System performance can be evaluated with regards to the following four criteria:

a) prevention of system failure;

b) minimization of the discharge of BOD;

c) minimization of capital and operating costs; and

d) minimization of sludge processing and disposal costs (Keinath et al., 1979).

In other words, the best design or the best operating strategy is the one which produces the cleanest effluent at the lowest cost.

For secondary settlers, prevention of system failure requires that design and operation prevent thickening failure. If the surface area of the settler is insufficient or the recycle rate too low, the applied solids loading will exceed the capacity of the settler. The height of the sludge blanket in the settler will accordingly increase, eventually resulting in high suspended solids loss as the blanket is swept into the effluent weir. Therefore, procedures to maximize clarifier performance must be subject to the provision that the settler not be allowed to fail in thickening.

6.1 Significance to Design

Providing that the potential for thickening failure has been prevented, consideration can be given to optimizing the clarification function of the settler. Based on the work of Camp (1953), surface overflow rate - the rate of plant inflow per unit of surface area - has been used by the engineering profession as the fundamental parameter determining clarification efficiency. Although Camp's analysis was based on the model of a discrete particle settling in a column of water, he recognized that for activated sludge, settling efficiency was increased due to flocculation. Camp contended that flocculation due to turbulence at the inlet and drag at the walls and floor was most effective in shallow sedimentation tanks with high velocities.

As a result of Camp's work, design procedures have concentrated on the selection of appropriate tank areas, either ignoring or minimizing tank depth. The following two quotes serve to illustrate this point:

1) "Although it may appear that increasing the depth will increase the removal, it is untrue. ... For a given volume, the best settling tanks should be designed to a more shallow depth than is common by today's design standards." (Aqua-Aerobic Systems, 1975)

2) "Camp's work points out that clarifier design may be arrived at independent of depth." (Mazurczyk et al., 1980)

The results from the factorial runs confirm that the plant inflow rate is an important design parameter. Accordingly, where multiple clarifiers are used, the designer must ensure that preferential feeding of the clarifiers is avoided. Where multiple clarifiers are supplied from a common channel or pipe, Perkins and Wood (1979) recommend the use of flow-splitting over free-fall weirs.

The factorial results do not support the contention that the depth of clarifier has an insignificant effect on effluent quality nor the contention that the best settling tank design is the one with the shallowest depth. On the contrary, the study results clearly indicated that removal efficiency for a given flow and MLSS concentration improved as tank depth increased. As discussed earlier, increasing the depth of a settler enhances flocculation, decreases the turbulent energy per unit of clarified volume and decreases the likelihood that disturbances created at the sludge blanket interface will reach the launders.

The depth of the settler and the rate at which the feed entered the settler were found to be interactive. Therefore, the deterioration in effluent quality resulting from increased flow into the settler was greater for a low sidewater depth than for a high one. This suggests that, not only is sidewater depth an important design parameter, but that it should be increased as the frequency and magnitude of peak flows into the plant increases. Deeper tanks also provide storage for sludge displaced from the aeration basin during peak hydraulic events.

A design procedure which accounts for settler depth is based on the results of column studies. As described by Conway and Edwards (1961) and Ford and Eckenfelder (1970), a column 150 mm in diameter and 3 m in height is filled with mixed lquor. Samples are collected from ports located 0.6 m apart. The percent solids removal is plotted versus overflow rate for various depths. From the plots, the required settler area and depth are selected with the values increased by a factor of safety.

The design procedure based on column studies is fundamentally more sound than the procedure based on overflow rate alone. The column studies account for the actual clarification properties of the floc and for the improvement in solids removal which accompanies an increase in depth. However, the columns do not reproduce the hydraulic environment and subsequent solids distribution which exists in a full-scale settler. At the start of a column test, particles are distributed uniformly over the cross-section of the column. Subsequently, the suspension settles under quiescent conditions. In a full-scale settler, due to the action of the "density current", neither the flow nor the solids distribution is uniform over the cross-section. Arbitrary factors of safety must be used to resolve the differences in performance between the two types of units. More research is required to document and improve the ability of the columns to predict full-scale performance.

As well, column studies do not account for the influence of recycle rate on solids removal. The results from the test settler indicated that the total rate of flow entering the settler influenced removal efficiency. For a given tank volume and influent flow rate, increasing the recycle rate increased the momentum of the "density current" and hence the amount of turbulent mixing. Therefore hydraulic loading criteria should be based on the rate per unit area of the total flow into the settler, instead of overflow rate calculated in the traditional manner.

MLSS concentration also influenced effluent quality and therefore must be considered in order to achieve cost efficient designs. Effluent suspended solids concentration increased as MLSS concentration increased. The rate of change for the test settler was found to be 4 mg/L per g/L increase in MLSS (for changes in MLSS which ranged from 1.2 to 6.2 g/L). MLSS is therefore a key parameter with regards to the optimization of overall capital costs for the activated sludge system. For low MLSS values, the required volume for the aeration basin is large while for the settler it is small. For high MLSS values, the situation is reversed. As pointed out by Tuntoolavest et al. (1980), evidence for a direct relationship between MLSS and effluent suspended solids concentration suggests that optimal activated sludge designs call for low MLSS concentrations.

Effluent suspended solids concentration did not change significantly with air flow rate. The study therefore failed to confirm the results of previous research as to the effect of air flow rate on floc shear. Given the current state of knowledge in this area, the designer of the activated sludge system would be prudent to design the oxygen transfer system to be as efficient as economically possible. Fine bubble aeration, dissolved oxygen control loops, and step aeration are elements of an efficient transfer system. Besides reducing the possibility of floc shear, these measures result in energy savings.

Floating solids contributed to the solids concentration in the effluent from the test clarifier. It is important therefore to equip all secondary clarifiers with scum baffles and skimmers. Scum baffles are inset from the weirs resulting in an annular area for floating solids to escape. In order to avoid this, consideration should be given to the addition of a horizontal baffle underneath the effluent weir and the scum baffle as recommended by Stukenberg et al. (1983). This plate would deflect floating solids away from the annulus between the scum baffle and weir. Also, it would deflect the upflow zone of the "density current" away from the zone. In this function, it is to be preferred over the use of inboard launders. Considerable expense is involved in cantilevering weirs into the tank and once located inboard they are difficult and dangerous to clean.

6.2 Significance to Operation

A good operational strategy for secondary settlers is to minimize the recycle rate subject to the previously stated provision that the settler not be allowed to fail in thickening. Decreasing the recycle rate reduces the feed flow rate into the settler, thereby reducing the intensity of turbulence within the tank. Recycle rate should therefore be maintained constant, at the lowest value which is consistent with the proper functioning of the settler with regards to thickening. With recycle rate constant, the sludge blanket in the settler will rise and fall due to diurnal fluctuations in flow. A "safe" minimum recycle rate is achieved if, for peak diurnal flows, there is sufficient distance from the top of the sludge blanket to the effluent weirs to prevent any solids scoured from the blanket from reaching the weirs.

The recycle rate determines the range of sludge blanket depths associated with the diurnal fluctuations. In terms of effluent quality, maintenance of low sludge blanket levels is probably the safest strategy. With low blanket levels, the ratio of turbulent energy to supernatant volume is lowered and there is less chance for disturbances generated at the sludge blanket interface to reach the effluent weirs.

The concentration of solids in the underflow from a clarifier with a low blanket level will tend to be dilute as there is more opportunity for supernatant to be pulled though the blanket. Provided that the treatment plant has a separate thickener, this is not serious. With a thickener to concentrate the clarifier underflow prior to conditioning, clarification and thickening can be optimized separately. However, if the plant does not have a separate thickener, the choice of blanket depths will be a tradeoff between the two functions.

The strategy of varying the recycle rate in proportion to blanket height or the rate of plant inflow suffers from the disadvantage of further increasing flow into the settler during periods of peak flow. A high rate of feed flow creates high momentum, increasing the velocity of the roll in the tank and hence increasing effluent turbidity. However, carrying a high sludge blanket does offer the advantage of enhancing solids removal by entrapment. Sorensen (1979) examined this strategy and found strong evidence to indicate that effluent quality was improved (see Figure 2.6). The strategy will be inappropriate if the sludge in the settler denitrifies or the plant is subject to high peak flows. Strategies which control recycle rate based on other variables such as the respiration rate of the mixed liquor (Flanagan, 1975) or the solids concentration of the influent to the aeration tank (Pearse, 1942) are likely to disturb the settler and offset any potential gains.

The results of the dynamic testing of the pilot-scale clarifier indicated that the deterioration in effluent quality following a sudden increase in flow rate was much faster than the improvement in quality following a sudden decrease of the same magnitude. Large surges in hydraulic flow in the plant will therefore result in a poorer quality effluent with the deterioration persisting long after the surge itself has passed. To prevent surges, on-off control of large influent or recycle pumps should be avoided. For small plants subject to large and sudden influent flow changes, flow equilization should be investigated. Stephenson (1983) has suggested that special attention be given to the design of outlet weirs in the primary settler and aeration tank to aid in dampening out hydraulic surges.

In addition to equipping clarifiers with scum removal equipment, an anoxic zone can be provided at the inlet of the aeration tank to control floating solids caused by denitrification. With a retention time of 30 to 40 min, the anoxic zone results in the reduction of nitrites and nitrates as they are used as electron acceptors by the bacteria (Wheatland, 1981; Tomlinson and Chambers, 1979). Briggs and Jones (1978) reported that 50 percent denitrification is consistently achieved with the use of the zone. As a result of adding an anoxic zone, oxygen requirements are reduced slightly and the mixed liquor is less susceptible to denitrification in the secondary clarifier.

Improved clarification can be obtained using the step feed modification which allows for the addition of influent feed from the primary settler along the length of the aeration basin. A constant recycle rate is maintained with sludge returned to the head end of the aeration basin. During periods of hydraulic overloading, the point of addition of influent feed is shifted towards the outlet end of the aeration basin. Consequently, biosolids are stored in the aeration tank near the inlet end while the flow entering the settler contains a reduced solids concentration. The net effect is to reduce solids loading to the settler leaving the recycle rate unchanged. For relatively high average overflow rates, Torpey and Chasick (1955) used step-feed in conjunction with low MLSS concentrations to obtain average effluent solids concentrations of 6, 17 and 19 mg/L at three New York treatment plants.

CHAPTER 7

SUMMARY AND CONCLUSIONS

The major objectives of this study were:

a) To evaluate the response of the clarifier to time-varying changes in hydraulic flows and operating conditions.

b) To determine, using statistical techniques, the influence of the important design and operating variables on solids removal during steady-state operation.

c) To compare the experimental results with those obtained by other researchers.

Research was conducted on a 2.4 m diameter settler to which mixed liquor was pumped from an activated sludge plant treating municipal sewage. Performance of the settler and activated sludge plant was monitored by sensors interfaced to a computer. On the basis of the results of the study and the supporting literature, it can be concluded that:

1) Suspended solids in the effluent from the final settler of an activated sludge plant treating municipal sewage account for the majority of BOD_5 discharged from the system. Consequently, the greatest improvement in overall system efficiency will come from improving the solids removal efficiency of the system.

2) A first-order model adequately described the change in effluent suspended solids concentration following a step decrease in feed flow which changed the overflow rate from 1.7 to 1.0 $m^3/m^2 \cdot hr$.

The change in solids concentration reflected the exponential decay of turbulence within the tank.

3) Effluent suspended solid concentrations increased following step increases in feed flow rate equal in magnitude to the step decreases. As some of the runs displayed overshoot, there was considerable variability in the shape of the response from run to run. Consequently, a second-order model was required to predict the settler response for increasing flows. The presence of overshoot was attributed to the discharge of floating solids previously trapped in the annular area between the effluent weir and the scum baffle.

4) The responses to step increases in flow rate were considerably faster than the responses to step decreases of equal magnitude. The time constants for the increases averaged 17 min compared to a value of 26 min for the decrease.

5) Changes in the concentration of effluent suspended solids followed time-varying changes in MLSS concentration with the majority of responses adequately modelled using first-order dynamics. For a 1 g/L change in MLSS concentration, induced changes in effluent solids ranged from 4 to 7 mg/L for MLSS concentrations which ranged from 1.2 to 6.2 g/L. Responses following changes in MLSS concentration were much slower than those following changes in feed flow rate, having time constants of approximately five hours.

6) Based upon regression analyses, changes in MLSS concentration, feed flow rate into the settler and sidewater depth accounted for

78 percent of the variability in effluent suspended solids concentration from the test clarifier. Effluent quality deteriorated as either MLSS or feed flow rate increased. Increasing sidewater depth improved effluent quality.

7) The effects of sidewater depth and feed flow rate on effluent quality were interactive; that is, the magnitude of the effect on effluent quality of changes in one of the variables depended on the level of the other.

8) Data collected by Fitch (1957) using settling columns and a suspension of calcium carbonate were subjected to regression analysis. The results from this analysis were consistent with those from the test clarifier in that both overflow rate and column depth were found to be important variables and that there was an interaction between them.

9) Analysis of residuals - the differences between the observations and model predictions - indicated that none of the unexplained variation in effluent suspended solids concentration could be attributed to variations in liquid temperature in the settler, the zone settling velocity or oxygen uptake rate of the mixed liquor or the influent suspended solids concentration to the plant.

10) Over the ranges investigated, the changes in air flow rate in the aeration tanks, rake speed, recycle rate out of the clarifier and feedwell depth had no significant influence on effluent quality.

11) Changes which resulted in an increase in the mean level of effluent suspended solids concentration also resulted in an increased variability in the concentration.

12) The results were consistent with the view that flocculation and entrapment as well as sedimentation are responsible for solids removal in the settler. The degree of flocculation and entrapment is influenced by the biological nature of the flocs and the existence of a vertical "density current" or roller which creates turbulence within the settler.

A number of recommendations concerning the design and operation of secondary clarifiers emerge from this study:

1) The selection of sidewater depth is an important design consideration. Depth should be related to the ratio of average to peak hydraulic flows, as well as to the need for preventing thickening failure.

2) Hydraulic loading criteria used in clarifier design should be based on the feed flow rate - the sum of plant inflow and recycle rates.

3) MLSS concentration is a key parameter with regards to the optimization of overall capital costs for the activated sludge systems. Designs therefore should investigate several target MLSS concentrations.

4) Recycle rate should be minimized subject to the need to prevent thickening failure.

5) Because large surges in hydraulic flow result in the deterioration of effluent quality long after the surge is past, on-off

control of large influent or recycle pumps should be strictly avoided. Flow equalization should be investigated for small plants subject to large variations in hydraulic flows.

CHAPTER 8

RECOMMENDATIONS FOR FUTURE WORK

1) A fundamental modelling and simulation study should be conducted on both secondary clarification and thickening with the models validated by the results from this and other pilot and field studies.

2) As the sidewater depth of a settler influences removal efficiency, there is a need to establish optimum depth to diameter ratios in terms of both removal efficiency and capital cost.

3) Floating solids due to denitrification contributed to the concentration of suspended solids in the test settler. The effect of pre-denitrification on effluent quality should be quantified.

4) Clarification dynamics should be investigated more extensively. The research should employ a variety of step sizes in hydraulic flows, examine the effect of the same size step at different sidewater depths and monitor the sludge blanket height during the changes.

5) Better data is required concerning the effect of air flow on the floc shear. Eddy size as well as the intensity of turbulence should be considered.

6) Methods should be developed for reducing the frequency and intensity of hydraulic surges in wastewater treatment plants. In

particular, special attention should be paid to the control of influent wastewater pumps.

7) Research is required to document and improve the ability of settling columns to predict the performance of full-scale settlers.

8) Alternative feedwell configurations should be developed and evaluated.

9) To further study the influence of process variables and system dynamics, data should be collected and analyzed from a number of full-scale activated sludge plants using on-line sensors.

REFERENCES

______________________. The Nalgene Settleometer Kit. N-165, Nalge Company, Rochester, New York.

ADAMS, R.M., ASANO, T. Removal mechanisms and design suggestions for activated sludge. Journal of the Water Pollution Control Federation, 50, 8: 1931-1942, 1978.

AGNEW, R.W. A mathematical model of a final clarifier for the activated sludge process. EPA Water Pollution Research Service Dept. No. 17090 FJW, February, 1972.

ANDERSON, N.E. Design of final settling tanks for activated sludge. Sewage Works Journal, 17, 1: 50-63, 1945.

ANDREWS, J.F. Dynamics and control of biological treatment processes. In: Research Needs for Automation of Wastewater Treatment. Proceedings of a Workshop held at Clemson, S.C. Editors: H.O. Buhr, J.F. Andrews and T.M. Keinath, Clemson University, 1975.

ANNIS, L. Secondary sedimentation study. MSc Thesis, University of Alberta, Edmonton, Alberta, 1979.

AQUA-AEROBIC SYSTEMS. Clarifiers. Bulletin 301, Aqua-Aerobic Systems, Rockford, Illinois, 1975.

ASTROM, K.J., EYKHOFF, P. System identification - a survey. Automatica, 7: 123-162, 1971.

BAYLEY, R.W., ADAMS, R.J. Discussion of: Flow patterns in a rectangular sewage sedimentation tank, by R.F. Willis and C. Davis. In: Advances in Water Pollution Research, Vol. 2. Editor: W.W. Eckenfelder, Pergamon Press, 1964. pp. 378-380.

BEBIN, J., JACQUART, J.C. Pilot studies on the utilization of flotation and accelerated clarification units for the direct physico-chemical treatment of domestic wastewaters. Progress in Water Technology, 10, 1/2: 205-216, 1978.

BENEFIELD, L.D., RANDALL, C.W. Biological Process Design for Wastewater Treatment. Prentice Hall, Englewood Cliffs, N.J., 1980.

BISGONI, J.J., DICK, R.I. Discussion of: Activated sludge - unified system design and operation. Journal of the Environmental Engineering Division, ASCE, EE5: 1048-1050, 1978.

BISGONI, J.J., LAWRENCE, A.W. Relationships between biological retention time and settling characteristics of activated sludge. Water Research, 5: 753-763, 1971.

BOX, G.E.P., HUNTER, J.S. The 2^{k-p} fractional factorial designs. Part I. Technometrics, 3, 3: 311-351, 1961.

BOX, G.E.P., HUNTER, W.G., HUNTER, J.S. Statistics for Experiments. An Introduction to Design, Data Analysis and Model Building. John Wiley and Sons, New York, 1978.

BOX, G.E.P., JENKINS, G.M. Time Series Analysis Forecasting and Control, Holden-Day, San Francisco, 1970.

BOYLE, W.H. Improved secondary clarifier performance: peripheral vs center effluent arrangement. Water and Sewage Works, 123, 10: 90-94, 1976.

BRIGGS, R., G.L. JONES. Aspects of automation of the activated sludge process. Water Pollution Control, 77, 2:439-446, 1978.

BROWN, M.J., LESTER, J.N. Comparison of bacterial extracellular polymer extraction methods. Applied and Environmental Microbiology, 40, 2: 179-185, 1980.

BRUCE, F.E. Discussion of: Flow patterns in a rectangular sewage sedimentation tank, by R.F. Willis and C. Davis. In: Advances in Water Pollution Research, Vol. 2. Editor: W.W. Eckenfelder, Pergamon Press, 1964. pp. 380-385.

BUHR, H.O. Discussion of: Dynamics and control of biological treatment processes. In: Research Needs for Automation of Wastewater Treatment Systems. Editors: H.O. Buhr, J.F. Andrews and T.M. Keinath. Clemson University, 1975.

BUSBY, J.B., ANDREWS, J.F. Dynamic modelling and control strategies for the activated sludge process. Journal of the Water Pollution Control Federation, 47, 5: 1055-1080, 1975.

BUSH, A.W., IRVINE, R.L. Solids separation in activated sludge system design and operation. Journal of the Water Pollution Control Federation, 52, 4:804-809, 1980.

CAMP, T.R. Sedimentation basin design. Sewage and Industrial Wastes, 25, 1: 1-4, 1953.

CAMPBELL, H.W., CRESCUOLO, P.J. Evaluation of microwave drying for rapid solids analyses. Environmental Technology Letters, 3: 289-296, 1982.

CASHION, B.S., KEINATH, T.M., SCHUK, W.W. Control strategies for the activated sludge process. Journal of the Water Pollution Control Federation, 51, 4: 851-833, 1979.

CHAPMAN, D.T. Memo to file, Project 80-12, Environment Canada, Environmental Protection Service, August 18, 1980.

CHAPMAN, D.T. Mixing efficiency as estimated by nonlinear least squares. Canadian Journal of Civil Engineering, 10, 4: 703-712, 1983.

CLEMENTS, M.S. Design of radial flow sedimentation tanks on velocity principles. Surveyor (U.K.), November 21: 2831, 1969.

CLOUGH, C.F.G. Discussion of: The design and operation of activated sludge final settling tanks. Water Pollution Control, 73, 1: 14-32, 1974.

COE, H.S., CLEVENGER, G.H. Methods for determining the capacities of slime settling tanks. Trans AIME, 55: 356, 1915.

COLLINS, M.A., CROSBY, R.M. Impact of flow variation on secondary clarifier performance. Proceedings of the Research Symposia, 53rd Annual WPCF Conference, Las Vegas, Nevada, September 28 to October 3, 1980.

COMMITTEE ON WATER POLLUTION MANAGEMENT of the Environmental Engineering Division, ASCE. Engineering design variables for the activated sludge process. Journal of the Environmental Engineering Division, ASCE, EE3: 473-503, 1980.

CONWAY, R.A., EDWARDS, V.H. How to design sedimentation systems from laboratory data. Chemical Engineering, September 18: 167-170, 1961.

COUGHANOWR, D.R., KOPPEL, L.B. Process Systems Analysis and Control. McGraw Hill Book Co., New York, 1965.

CROSBY, R.M., BENDER, J.H. Hydraulic considerations that affect secondary clarifier performance. Technology Transfer, EPA, March 1980.

CZARNECKI, G.J., LIPPINCOTT, J.K. Screw pumps. In: Pump Handbook. Editors: I.J. Karassik, W.C. Krutzsch, W.H. Fraser, and J.P. Messina, McGraw-Hill, 1976. pp. 3-47 - 3-70.

DAGUE, R.R., BAUMANN, E.R. Hydraulics of circular settling tanks determined by models. Presented at the 1961 Annual Meeting of the Iowa Water Pollution Control Association, Lake Okoboji, Iowa, June 8, 1961.

DICK, R.I. Role of activated sludge settling tanks. Journal of the Sanitary Engineering Division, ASCE, SA2: 423-436, 1970a.

DICK, R.I. Thickening. In: Water Quality Improvement by Physical and Chemical Processes. Editors: E.F. Gloyna and W.W. Eckenfelder, University of Texas, 1970b. pp. 358-369.

DICK, R.I. Discussion of: Generic feed forward control of activated sludge. Journal of the Environmental Engineering Division, ASCE, EE3, June, 1974. pp. 770-771.

DICK, R.I. Folklore in the design of final settling tanks. Journal of the Water Pollution Control Federation, 48, 3: 633-644, 1976.

DICK, R.I., VESILIND, P.A. The sludge volume index - what is it? Journal of the Water Pollution Control Federation, 41, 7: 1285-1291, 1969.

DICK, R.I., YOUNG, K.W. Analysis of thickening performance of final settling tanks. Proceedings of the 27th Industrial Waste Conference, Purdue University, 1972. pp. 33-54.

DIETZ, J.D., KEINATH, T.M. Clarification mechanisms for a flocculent slurry. Presented at the 55th Annual Conference of the Water Pollution Control Federation, St. Louis, Missouri, Oct. 3-8, 1982.

DRAPER, N.R., SMITH, H. Applied Regression Analysis. Wiley and Sons, New York, 1966.

ECKHOFF, D.W., JENKINS, D. Transient loading effects in the activated sludge process. Journal of the Water Pollution Control Federation, 38, 3: 367-368, 1966.

EDGAR, T.F. Process Analysis and Description. In: CACHE Monograph Series in Real-Time Computing, D.A. Mellichamp, editor, CACHE, Cambridge Massachusetts, 1979.

ENGLER, F. Survey of currently available portable sludge level detectors. Ontario Minstry of the Environment Technical Information Bulletin, No. 1006, May 25, 1978.

EUR-CONTROL. Instruction Manual, ME6 1 Automatic Measuring Instrument, M218.43g, Eur-Control U.S.A. Inc., 2579 Park Central Boulevard, Decatur, Georgia.

FAIR, G.M., GEYER, J.C., OKUN, D.A. Water and Wastewater Engineering. Vol. 2, Water Purification and Wastewater Treatment and Disposal. John Wiley and Sons Inc., 1968.

FIEDLER, R.A., FITCH, E.B. Appraising basin performance from dye test results. Sewage and Industrial Wastes, 31, 9: 1016-1021, 1959.

FITCH, E.B. The significance of detention in sedimentation. Sewage and Industrial Wastes, 29, 10: 1123-1363, 1957.

FITCH, E.B. The significance of detention in sedimentation. Discussion II. Detention time concepts. Sewage and Industrial Wastes, 30, 11: 1361-1363, 1958.

FITCH, E.B. Sedimentation of flocculant suspensions: State of the art. AIChE Journal, 25, 6: 913-930, 1979.

FITCH, E.B., KOS, P. Toward a more meaningful index of sludge quality. Journal of the Water Pollution Control Federation, 48, 8: 1979-1987, 1976.

FLANAGAN, M.J. Automation of the activated sludge process. In: Research Needs for Automation of Waste Treatment Systems. Editors: H.O. Buhr, J.F. Andrews and T.M. Keinath, Clemson University, 1975.

FORD, D.L., ECKENFEDLER, W.W. Water Pollution Control, Experimental Procedures for Process Design. The Pemberton Press, Austin and New York, 1970.

GARRETT, M.T. Jr., MA, J., YANG, W., HYARE, G., NORMAN, T., AHMAD, Z. Improving the performance of Houston's Southwest Wastewater Treatment Plant. Presented at the Workshop on Design and Operation of Large Watewater Treatment Plants, Vienna, Austria, 19-23 Sept., 1983.

GHOBRIAL, F.H. Importance of the clarification phase in biological process control. Water Research, 12: 1009-1016, 1978.

GREAT LAKES-UPPER MISSISSIPPI RIVER Board of State Sanitary Engineers. Recommended standards for sewage works. 1971.

GREGORY, R. Floc blanket clarification. Water Research Centre, Technical Report TR 111, 1979.

HALL, E.J. Discussion of: Hydraulic and removal efficiencies in sedimentation basins by J.R. Villemonte et al. In: Advances in Water Pollution Research, 2, Water Pollution Control Federation, 1966. pp. 399-404.

HAMLIN, M.J. Discussion of: Hydraulic and removal efficiencies in sedimentation basins by J.R. Villemonte et al. In: Advances in Water Pollution Research, 2, Water Pollution Control Federation, 1966. pp. 404-408.

HAZEN, A. On sedimentation. Transaction ASCE, 53: 45, 1904.

HEINKE, G.W., QAZI, M.A., TAY, A. Design and performance criteria for settling tanks for the removal of physical/chemical flocs. Vol. III. Project Report No. 56, Research Program for the Abatement of Municipal Pollution Under Provisions of the Canada-Ontario Agreement on Great Lakes Water Quality, 1977.

HENRY, J.G., SALENIEKS, E.E. Variation in sludge settleability with temperature. Water Pollution Research Journal of Canada, 15, 1: 73-82, 1980.

HIBBERD, R.L., JONES, W.F. The design and operation of activated sludge final settling tanks. Water Pollution Control, 73, 1: 14-32, 1974.

HILL, J.W. Discussion of: The design and operation of activated sludge final settling tanks. Water Pollution Control, 73, 1: 14-32, 1974.

IEC INTERNATIONAL ENVIRONMENTAL CONSULTANTS LTD. Improved primary and secondary clarifier operating efficiency. CPAR Project Report 491-1, Environment Canada, Forestry Service, October 31, 1975.

JOINT COMMITTEE of the Water Pollution Control Federation and the American Society of Civil Engineers. Wastewater Treatment Plant Design. Lancaster Press, Lancaster, Pa., 1977.

KALBSKOPF, K.H. Discussion of: Hydraulic and removal efficiencies in sedimentation basins by J.R. Villemonte et al. In: Advances in Water Pollution Research, 2, Water Pollution Control Federation, 1966. pp. 408-416.

KALBSKOPF, K.H. European practices in sedimentation. In: Water Quality Improvement by Physical and Chemical Processes. Editors: E.F. Gloyna and W.W. Eckenfelder, University of Texas Press, No. 3, 1970. pp. 92-103.

KALINSKE, A.A. Turbulence in aeration basins. Industrial Water Engineering, March: 35-38, 1971.

KARASSIK, I.J., KRUTZSCH, W.C., FRASER, W.H., MESSINA, J.P. Pump Handbook. McGraw-Hill Book Co., New York, 1976.

KATZ, W.J., GEINOPOLOS, A. Discussion of: Flow patterns in a rectangular sewage sedimentation tank by R.F. Willis and C. Davis. In: Advances in Water Pollution Research, Vol. 2. Editor: W.W. Eckenfelder, Pergamon Press, 1964. pp. 366-378.

KEINATH, T.M., RYCKMAN, M.D., DANA, C.H., HOFER, D.A. Design and operational criteria for thickening of biological sludges, Part I: Synopsis. Water Resources Research Institute, Clemson University, September 1976a.

KEINATH, T.M., RYCKMAN, M.D., DANA, C.H., HOFER, D.A. Design and operational criteria for thickening of biological sludges, Part II: General. Water Resources Research Institute, Clemson University, September 1976b.

KEINATH, T.M., RYCKMAN, M.D., DANA, C.H., HOFER, D.A. Activated sludge - unified system design and operation. Journal of the Environmental Engineering Division, ASCE, EE5: 829-849, 1979.

KELEGHAN, W. The hi-capacity thickener. Filtration and Separation, November/December: 534-538, 1980.

KRENKEL, P.A. Waste dispersion characteristics of streams using turbulent diffusion phenomenon. Journal of the Water Pollution Control Federation, 34, 12: 1203-1212, 1962.

KRONIS, H. TONELLI, F. Full-scale evaluation of high rate screening devices. Research Program for the Abatement of Municipal Pollution Within the Provisions of the Canada-Ontario Agreement on Great Lakes Water Quality, Project No. 72-1-22, Minister of Supply and Services, 1980.

KYNCH, G.J. A theory of sedimentation. Transactions of the Faraday Society, 48: 166, 1952.

LAQUIDARA, V.D., KEINATH, T.M. Mechanism of clarification failure. Journal of the Water Pollution Control Federation, 55, 1: 54-57, 1983.

LECH, R.F., GRADY, C.P.L. JR., LIM, H.C., KOPPEL, L.B. Automatic control of the activated sludge process - II. Efficacy of control strategies. Water Research, 12: 91-99, 1978.

LEVENSPIEL, O. Chemical Reaction Envineering, An Introduction to the Design of Chemical Reactors, John Wiley and Sons, New York, 1962.

LIPTAK, B.G., editor. Instrument Engineer's Handbook. Volume II. Process Control, Chilton Book Company, Philadelphia, 1970.

LOGAN, R.P., BUDD, W.E. Effect of BOD loadings on activated sludge plant operation. In: Biological Treatment of Sewage and Industrial Wastes, Vol. I: Aerobic Oxidation. Editors: J. McCabe and W.W. Eckenfelder, Reinhold Publishing Corp., New York, 1956.

LAUGHTON, P.J. Action slip to J. Stephenson (Environment Canada) regarding Ontario Ministry of the Environment Design Parameters, October 8, 1980.

LUYBEN, W.L. Process Modelling, Simulation, and Control for Chemical Engineers. McGraw-Hill Book Co., New York, 1973.

MAZURCZYK, A.L., SMITH, J.E., APPLEGATE, C.S., METLER, M. A methodology for the assessment of clarifier operation as demonstrated at Pruszkow, Poland. Proceedings of Research Symposia, 53rd Annual WPCF Conference, Las Vegas, Nevada, 1980.

McKINNEY, R.E. Sedimentation tank design and operations. University of Kansas, Lawrence, Kansas. [No year available.]

McKINNEY, R.E. Design and operational model for complete mixing activated sludge system. Biotechnology and Bioengineering, XVI: 703-722, 1974.

MENDIS, J.B., BENEDEK, A. Tube settlers in secondary clarification of domestic wastewater. Journal of the Water Pollution Control Federation, 22, 7: 1893-1897, 1980.

METCALF AND EDDY INC. Wastewater Engineering: Treatment Disposal Reuse, 2nd Edition. McGraw-Hill Book Co., New York, 1979.

MUNCH, W.L., FITZPATRICK, J.A. Performance of circular final clarifiers at an activated sludge plant. Journal of the Water Pollution Control Federation, 50, 2: 265-276, 1978.

MURPHY, K.L. Tracer studies in circular sedimentation basins. Proceedings of the 18th Industrial Waste Conference, Purdue University, 1964. p. 374-390.

NEO NISHI HARA. Neo Portable Sludge Density Meter NU-PS. Control and Metering, 711 Kipling Avenue, Toronto, Ontario.

OLSSON, G. State of the art in sewage treatment plant control. Presented at the Engineering Foundation Conference on Chemical Process Control, Asilomar Conference Grounds, Pacific Grove, California, January 18 to 23, 1976.

OLSSON, G. Personal communication, Wastewater Technology Centre, January, 1984.

O'MELIA, C.R. Coagulation in water and wastewater treatment. In: Water Quality Improvement by Physical and Chemical Processes. Editors: E.F. Gloyna and W.W. Eckenfelder, Jr., University of Texas Press, 1970. pp. 219-316.

PALM, J.C., JENKINS, D., PARKER, D.S. The relationship between organic loading, dissolved oxygen concentration and sludge settleability in the completely-mixed activated sludge process. Presented at the 51st Annual Conference, Water Pollution Control Federation, Anaheim, California, October 4, 1978.

PARKER, D.S. Assessment of secondary clarification design concepts. Presented at the 55th Annual Conference of the Water Pollution Control Federation, St. Louis, Missouri, Oct. 7, 1982.

PARKER, D.S., KAUFMAN, W.J., JENKINS, D. Floc breakup in turbulent flocculation processes. Journal of the Sanitary Engineering Division, ASCE, SA1: 78-89, 1972.

PARTECH. Model ASLD 2000 automated sludge level detector. Cancoppas Ltd., 1031 North Service Road East, Oakville, Ontario.

PEARSE, L. The operation and control of activated sludge sewage treatment works. Sewage Works Journal, 14, 3: 3-69, 1942.

PERKINS, R., WOOD, T.R. Some thoughts on the design of secondary-settlement tanks in the light of operational experience in the West Midlands. Water Pollution Control, 78, 1: 72-81, 1979.

PFLANZ, P. Performance of (activated sludge) secondary sedimentation basins. In: Advances in Water Pollution Research, Proceedings of the 4th International Conference. Editor: S.H. Jenkins, Pergamon Press, New York, 1969.

PIPES, W.O. Discussion of: Toward a more meaningful index of sludge quality. Journal of the Water Pollution Control Federation, Part 1, 49, 3: 503-507, 1977.

PIPES, W.O. Activated sludge bulking and settling tank performance. Journal of the Water Pollution Control Federation, 51, 10: 2534-2537, 1979.

POLTA, R.C., STULC, D.C. Automatic sludge blanket control in an operating gravity thickener. EPA-600/2-79-159, Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, Ohio, November 1979.

PORTA, F.R., WILLIAMS, S.E., POSTHUMA, A.R. Plant scale clarifier studies at Detroit. Presented at the 53rd Annual Conference of the Water Pollution Control Federation, Las Vegas Sept. 28 - Oct. 3, 1980.

REED, S.C., MURPHY, R.S. Low temperature activated sludge settling. Journal of the Sanitary Engineering Division, ASCE, SA4: 747-767, 1969.

RESCH, H. Investigations of the performance of upflow final settling tanks in activated sludge plants. Progress in Water Technology, 12, 3: 201-211, 1980.

RICHARDSON, J.R., ZAKI, W.W. Sedimentation and fluidization, Part 1. Transactions Institution of Chemical Engineers, 32: 35, 1954.

RICKERT, D.A., HUNTER, J.V. Colloidal matter in wastewaters and secondary effluents. Journal of the Water Pollution Control Federation, 44, 1: 134-139, 1972.

RODI, W. Turbulence Models and Their Application in Hydraulics - A State of the Art Review, International Association for Hydraulic Research, Delft, Netherlands, 1980.

SAWYER, C.N. Final clarifiers and clarifier mechanisms. In: Biological Treatment of Sewage and Industrial Wastes, Vol. 1, Aerobic Oxidation. Editors: J. McCabe and W.W. Eckenfelder, Jr., Reinhold Publishing Co., New York, 1956. pp. 328-342.

SCHAFFNER, M.W., PIPES, W.O. Underflow rate and control of an activated sludge process. Journal of the Water Pollution Control Federation, 50, 1: 20-30, 1978.

SESTAK, J. Secondary sedimentation basins with sludge removal by suction. Vodni Hospodarstvi, 30, 3: 69-74, 1980.

SEZGIN, M., JENKINS, D., PARKER, D.S. A unified theory of filamentous activated sludge bulking. Journal of the Water Pollution Control Federation, 50, 2: 362-381, 1978.

SHERMAN, R.W. Judging the sludge blanket. Deeds and Data, Water Pollution Control Federation, May: 9-10, 1980.

SHIH, C. Investigation of Dynamic Performance of Watewater Treatment Plants. PHD Thesis, University of Wisconsin-Madison, 1976.

SHIN, B.S., DICK, R.I. Effect of permeability and compressibility of flocculent suspensions on thickening. Progress in Water Technology, 7: 177, 1975.

SILVESTON, P.L. Design of settling basins with allowance for residence time distributions. The Canadian Journal of Chemical Engineering, 47: 521-524, 1969.

SILVESTON, P.L., STEPKO, W.A., SAKATA, N., CORDOBA-MOLINA, J.F., RIETEMA, K., HUDGINS, R.R. On the use of resident time distributions for the design of clarifiers. The Canadian Journal of Chemical Engineering, 57: 83-97, 1979.

SMITH, I.R. Turbulence in lakes and rivers. Scientific Publication No. 29, The Freshwater Biological Association, The Ferry House, Ambleside, Cumbria, 1975.

SPEIRS, G. Stationary hydraulic screen. Memo to file, Government of Canada Memorandum, File No. 80-12, August 25, 1981.

SPEIRS, G., CHAPMAN, D., LUXON, E., MATTHEWS, J. "The Calibration and Evaluation of Dissolved Oxygen Sensors in an Activated Sludge Plant", accepted for publication in the Water Pollution Research Journal of Canada, 1983.

SORENSEN, P.E. Pilot-scale evaluation of control schemes for the activated sludge process. Contributions from the Water Quality Institute, Danish Academy of Technical Science, No. 1, 1979.

SORENSEN, P.E. Plant optimization for effluent quality and energy control purposes. Workshop: Automation of the Activated Sludge Process, Wastewater Technology Centre, Canada Centre for Inland Waters, Burlington, Ontario, June 27, 1980.

STANDARD METHODS FOR THE EXAMINATION OF WATER AND WASTEWATER. 14th Edition, American Public Health Association, Washington, D.C., 1975.

STEPHENSON, J.P. Personal communication, Wastewater Technology Centre, November 1980.

STEPHENSON, J.P. Personal communication, Wastewater Technology Centre, Burlington, Ontario, 1983.

STEPHENSON, J.P., LUXON, E.G., MONAGHAN, B.A., GILLESPIE, R.G. Evaluation of instruments for continuous activated sludge monitoring. Presented at the International Workshop on Practical Experience of Control and Automation in Wastewater Treatment and Water Resources Management, Munich and Rome, June 20 to 26, 1981.

STUKENBERG, J.R., RODMAN, L.C., TOUSLEE, J.E. Activated sludge clarifier design improvements. Journal of the Water Pollution Control Federation, 55, 4: 341-348, 1983.

TALMAGE, W.P., FITCH, E.B. Determining thickener unit areas. I&EC, 47: 38, 1955.

TASK COMMITTEE on Final Clarification of Water Pollution Management Committee of the Environmental Engineering Division. Final clarifiers for activated sludge plants. Journal of the Environmental Engineering Division, ASCE, EE5: 803-816, 1979.

TEBBUTT, T.H.Y. The performance of circular sedimentation tanks. Water Pollution Control, 68. [illegible] 467-474, 1969.

TEBBUTT, T.H.Y., CHRISTOULAS, D.G. Performance relationships for primary sedimentation. Water Research, 9: [illegible] 356, 1975.

TENNEY, M.W., VERHOFF, F.H. Chemical and autoflocculation of microorganisms in biological wastewater treatment. Biotechnology and Bioengineering XV: 1045 1073, 1973.

THACKSTON, E.L., HAYS, J.R., KRENKEL, P.A. Least squares estimation of mixing coefficients. Journal of the Sanitary Engineering Division, ASCE, SA3: 47 [illegible] 1967.

[illegible] HANSSON, L. The residence time distribution in systems with recirculation [illegible] flow. The [illegible]

TOMLINSON, E.J. Bulking - a survey of activated sludge plants. Technical Report TR35, Water Research Centre, Stevenage Laboratory, November 1976.

TOMLINSON, E.J., CHAMBERS, B. The Use of Anoxic Mixing Zones to Control the Settleability of Activated Sludge. Technical Report TR116, Water Research Centre, Stevenage Laboratory, 1979.

TORPEY, W.N. Practical results of step aeration. Sewage Works, 20, 5: 781-788, 1948.

TORPEY, W.N., CHASICK, A.H. Principles of activated sludge operation. Sewage and Industrial Wastes, 27, 11: 1271-1233, 1955.

TRACY, K.D., KEINATH, T.M. Dynamic model for thickening of activated sludge. AIChE Symposium Series, 70, 136: 291-307, 1974.

TSUGITA, R.A., DECOITE, D.C.W., RUSSELL, L. Process Control Manual for Aerobic Biological Wastewater Treatment Facilities: Section II - Process Control. National Technical Information Service, U.S. Department of Commerce, Springfield, Va., March 1977.

TUNTOOLAVEST, M., MILLER, E., GRADY, C.P. LESLIE JR. Characterization of wastewater treatment plant final clarification performance. Purdue University Water Resources Research Centre, Technical Report. No. 129, June 1980.

U.S. ENVIRONMENTAL PROTECTION AGENCY. Process Design Manual for Suspended Solids Removal. EPA 625/1-75-0032, January 1975.

VESILIND, P.A. The influence of stirring in the thickening of biological sludge. PhD Thesis, University of North Carolina, Chapel Hill, 1968.

VESILIND, P.A. Treatment and Disposal of Wastewater Sludges, Ann Arbor Science, 1975.

VILLEMONTE, J.R., ROHLICH, G.A. Hydraulic characteristics of circular sedimentation. Proceedings of the 17th Industrial Waste Conference, Purdue University, 1962. pp. 682-702.

VILLEMONTE, J.R., ROHLICH, G.A., WALLACE, A.T. Hydraulic and removal efficiencies in sedimentation basins. In: Advances in Water Pollution Research. 2. Water Pollution Control Federation, 1966. pp. 381-395.

WALTON, E. Discussion of: The design and operation of activated sludge final settling tanks. Water Pollution Control, 73, 1: 14-32, 1974.

WATER POLLUTION CONTROL FEDERATION. WPCF Manual of Practice No. 6, Units of Expression for Wastewater Treatment. WPCF, Washington, D.C., 1976.

WHEATLAND, A.B. Discussion of: Hogsmill valley works: twenty years on" by E.G. Eves. Water Pollution Control, 460, 1981.

WHITE, M.J.D. Settling of activated sludge. Water Research Centre, Technical Report TR 11, 1975a.

WHITE, M.J.D. Instruction manual for WRC settling apparatus for activated sludge. Water Research Centre, Technical Memorandum TM 103, March 1975b.

WHITE, M.J.D. Letter to D. Chapman, Ref. SL 514/00/00, November 26, 1980.

WOLFE, P. Non-linear least squares (GAUSHAUS). University of Wisconsin Computing Centre, December 1965.

YOSHIOKA, N. Continuous thickening of homogeneous flocculated slurries. Kagaku Kogaku, 21: 66, 1957.

APPENDIX A

PROCEDURE FOR ESTIMATING CONTRIBUTIONS OF EFFLUENT SUSPENDED SOLIDS TO EFFLUENT BOD_5

APPENDIX A

PROCEDURE FOR ESTIMATING CONTRIBUTION OF EFFULENT SUSPENDED SOLIDS TO EFFLUENT BOD_5

Calculations for Table 2.2

Question: For the grand means of Table 2.2, how much of the effluent BOD_5 is due to the suspended solids concentration?

Influent Characteristics - From Committee on Water Pollution Management, ASCE, 1980, Table 2, for conventional system:

Number of Plants		Mean BOD_5 (mg/L)	Mean SS (mg/L)
3		186.3	202.0
8		165.6	145.8
1		140.0	128.0
	GRAND MEAN	168.6	158.4

Process Characteristics - From Metcalf and Eddy (1979), Table 10.4, for conventional system:

Hydraulic retention time (θ): 6.0 h

Solids retention time (θ_c): 5 to 15 days

Procedure:

a) From Benefield and Randall (1980):

Insoluble $BOD_5 = .5 (1.42 \times k_d \times X_a C_e)$

where C_e = effluent SS (mg/L)

k_d = decay coefficient = 0.36 day^{-1}

$X_a = \frac{M_a}{M_T}$ = ratio of active to total mass

b) From McKinney (1974):

$$M_a = \frac{y \cdot K_m \cdot s\ \theta_c}{1 + K_d \cdot \theta_c}$$

where k_m = metabolism factor = $15(h^{-1})$

M_e = endogenous mass = $0.2 \times k_d \times M_a \times \theta_c$ (mg/L)

M_{ir} = inert organic mass

$= 0.35 \times 0.75 \times INFSS \times \theta_c/\theta$

M_{iir} = inert inorganic mass

$= 0.25 \times INF \cdot SS \times \theta_c/\theta$

S = effluent BOD_5 (mg/L)

S_0 = influent BOD_5 (mg/L)

Y = maximum yield coefficient = 0.63

Results:

Using the preceding information and procedure:

θ_c (days)	(1)*	(2) Insoluble BOD_5 (mg/L)	(3)**
5	0.78	28.6	88%
15	0.40	14.6	45%

(1)* : Fraction of effluent SS which is biodegradable
(3)** : Fraction of total BOD_5 which is insoluble

APPENDIX B

MONTHLY PROJECT DIARY

APPENDIX B

MONTHLY PROJECT DIARY

July, 1980:

- Literature review initiated and relevant papers collected.
- Initiation of re-design of pilot plant equipment.

August, 1980:

- Dye tests conducted on aeration tanks of PP#1 to determine mixing characteristics.
- Literature review.
- Identification of required instrumentation and pilot plant modifications.

September, 1980:

- Analysis of tracer study results for aeration tanks.
- Cost estimates for new instrumentation.

October, 1980:

- Modifications initiated to extended aeration plant.
- Meeting with Dr. Les Grady at Purdue University to discuss project objectives.
- Meeting with Dr. Dan Smith of University of Alberta to review proposed experimental program.

November, 1980:

- Ongoing modifications to extended aeration plant.
- Completion of renovations to test clarifier and placement next to aeration tanks.
- Overhaul of pumps and flow meters.

December, 1980:

- Insulation of test clarifier.
- Installation of three of four process pumps.
- Pipework completed for step-feed and step-recycle.
- Identification of air flow monitors.

MONTHLY PROJECT DIARY (Cont'd)

January, 1981:

- First draft of literature review was completed.
- Laboratory trailer located next to test facility.
- A shed was constructed to cover the clarifier feed-pump.

February, 1981:

- Turbidimeters installed on effluent lines.
- Process instrumentation wired to digital display panel and terminal box prior to interfacing to HP-1000 computer.
- Pipework completed.
- Review of research proposal by Thesis Committee.

March, 1981:

- Test clarifier and activated sludge system filled with water to check for leaks in tanks and piping.
- Piping modifications carried out to allow for mag meters to be connected in series.

April, 1981:

- Magnetic flow meters were calibrated.
- Keen "Aircomb" diffusers were replaced.
- Plant was seeded on April 21 with waste sludge from Burlington Skyway Treatment Plant.
- Statistical design reviewed with Dr. John MacGregor of McMaster University and Joe Stephenson of Wastewater Technology Centre.

May, 1981:

- Pump controllers were "tuned" and influent sewage controlled at 140 L/min.
- Installation of blower control equipment initiated.
- Clogging of pump control valves due to "stringy" material in feed sewage.

June, 1981:

- A stationary vertical screen was installed to screen raw sewage and prevent clogging of control valves.
- Tracer study on test clarifier was initiated.

MONTHLY PROJECT DIARY (Cont'd)

July, 1981:

- Vortex air flow meter positioned in air line.
- Calibration of on-line sensors initiated.

August, 1981:

- Heat tracing for piping was installed.
- Completion of tracer study for test clarifier.
- Step changes introduced to inflow and recycle rates to test clarifier.

September, 1981:

- Housing constructed for vertical stationary screen.
- A lack of reproducibility in pHOX DO measurements resulted in further calibration and investigation.
- Circuit boards for the Parajust blower controller were replaced and computer control of air flow rate was initiated.

October, 1981:

- Completion of pipe insulation and heat tracing.
- Packing in system recycle pump was replaced.
- Project review with Dr. Dan Smith.

November, 1981:

- Nine of the 16 runs for the first factorial were completed.
- Variations in the suspended solids concentrations in the test clarifier underflow were observed to be related to the action of the rake.
- Parajust blower controller failed; air flow rate was controlled using manual adjustment.

December, 1981:

- The remaining runs for the second factorial were completed.
- Preliminary step test results reviewed by Dr. Gustaf Olsson of the Lund Institute; plans developed for additional dynamic runs.

MONTHLY PROJECT DIARY (Cont'd)

January, 1982:

- Plant maintenance carried out and mag meters re-calibrated.
- Program "BAKLM" written and debugged.

February, 1982:

- Analysis of results from first factorial.
- Review of results in conjunction with Dr. John MacGregor of McMaster University.
- Second set of runs designed.
- Effluent turbidimeters and solids meters were re-calibrated.

March, 1982:

- A series of dynamic tests were carried out.
- Runs initiated for second factorial.

April, 1982:

- Runs for second factorial were completed.

June, 1982:

- Runs for a third factorial were completed.
- Review of results with Dr. John MacGregor
- A further series of dynamic tests completed.

July, 1982:

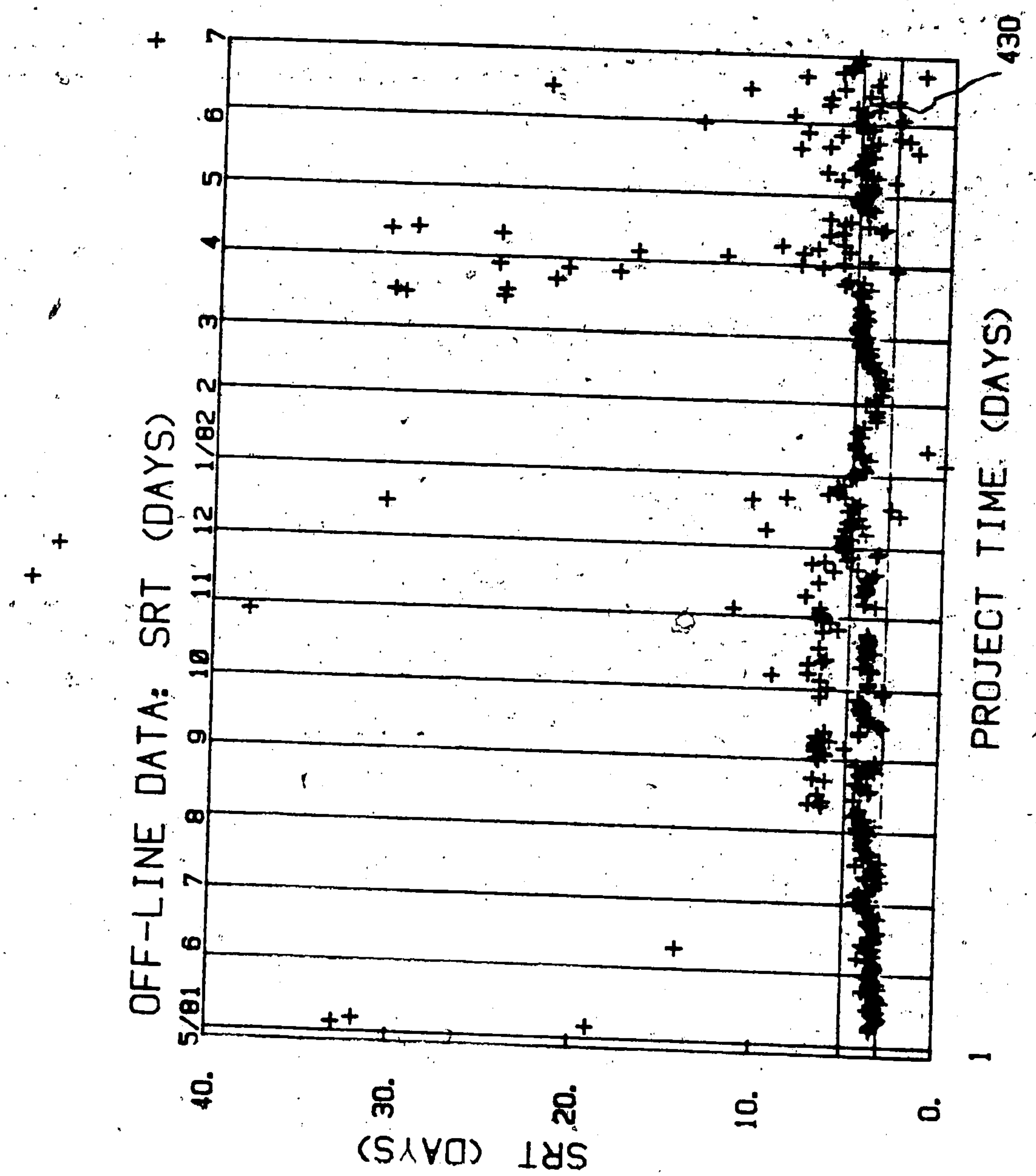
- Completion of experimental work on July 1, 1982.

July, 1982 to February, 1983:

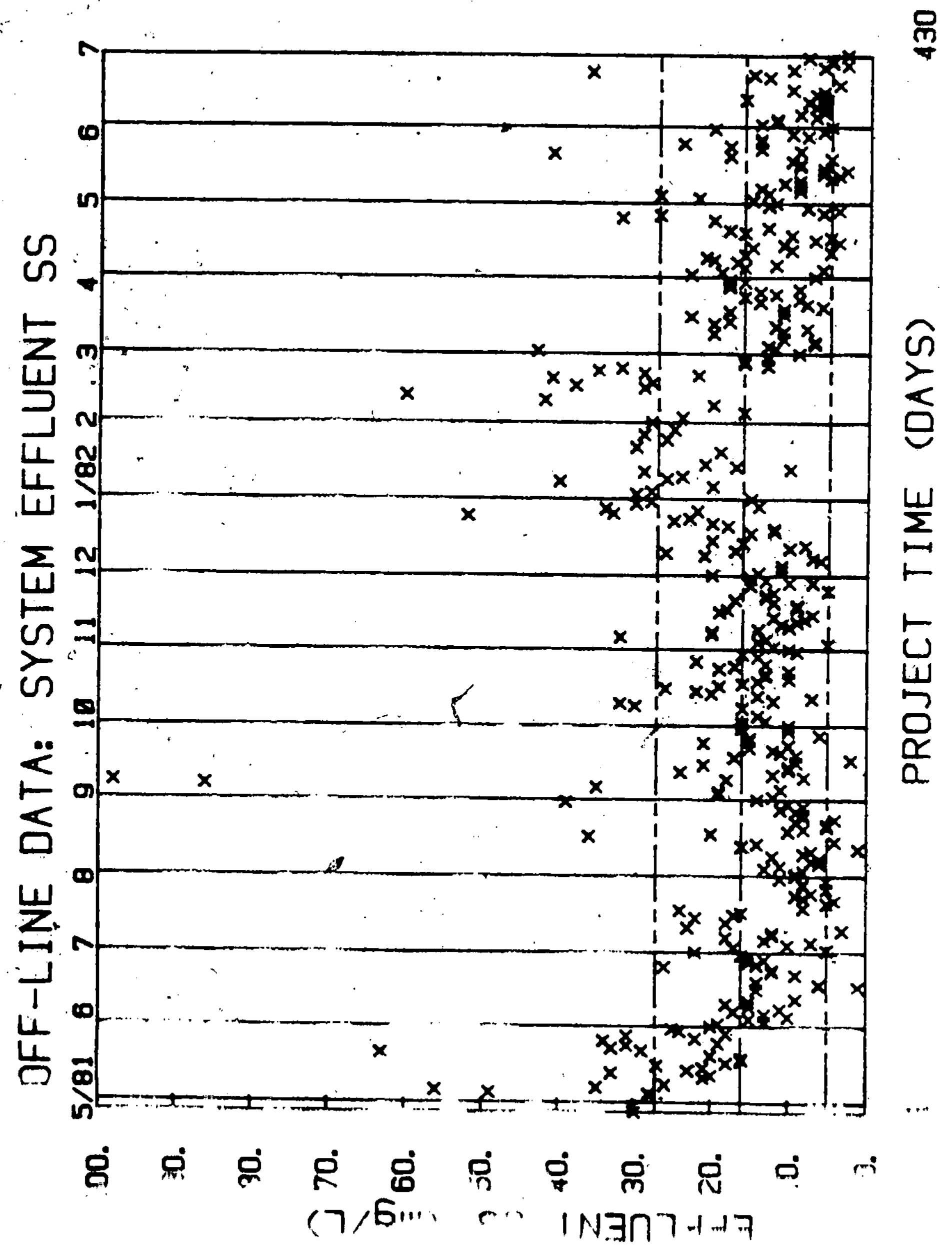
- Analysis of results.

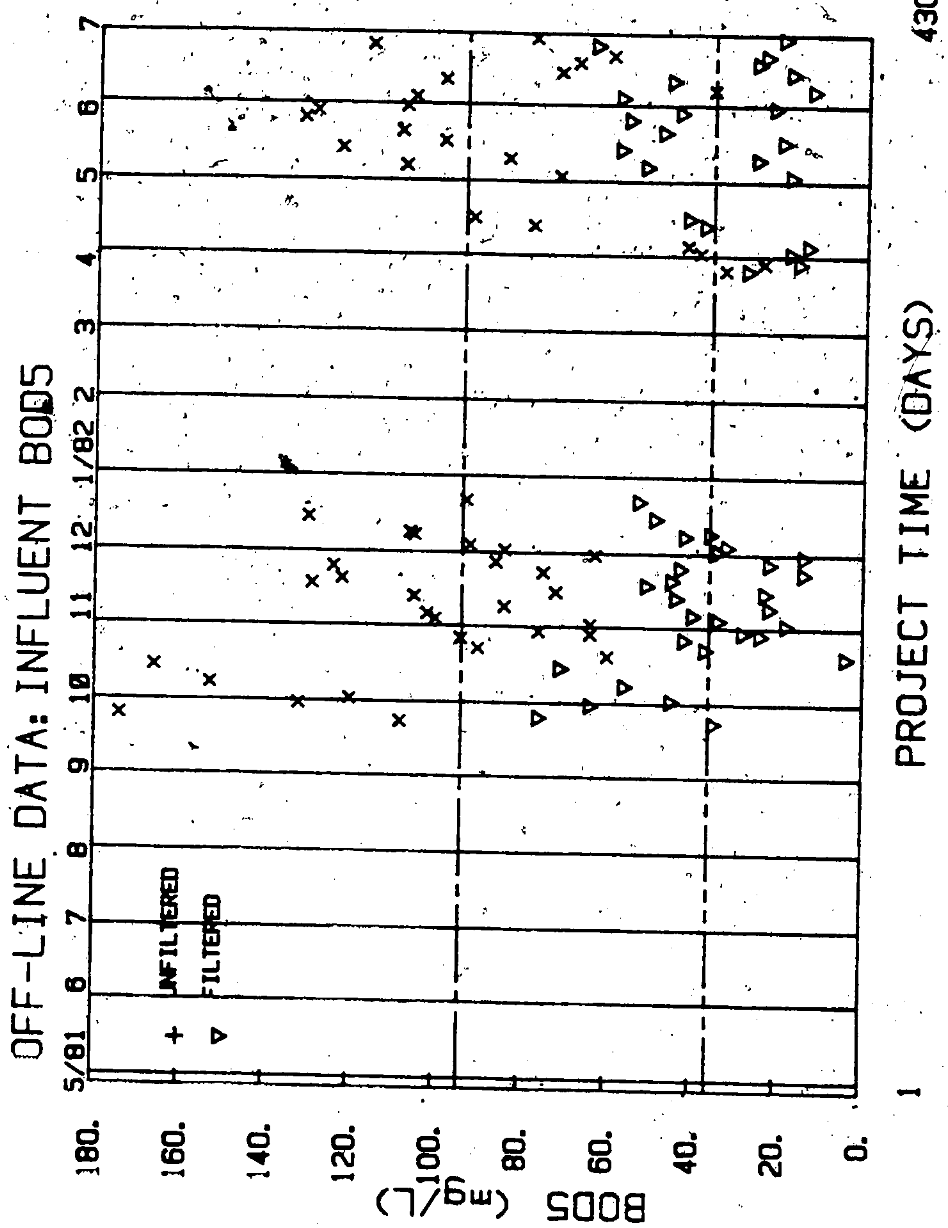
APPENDIX C

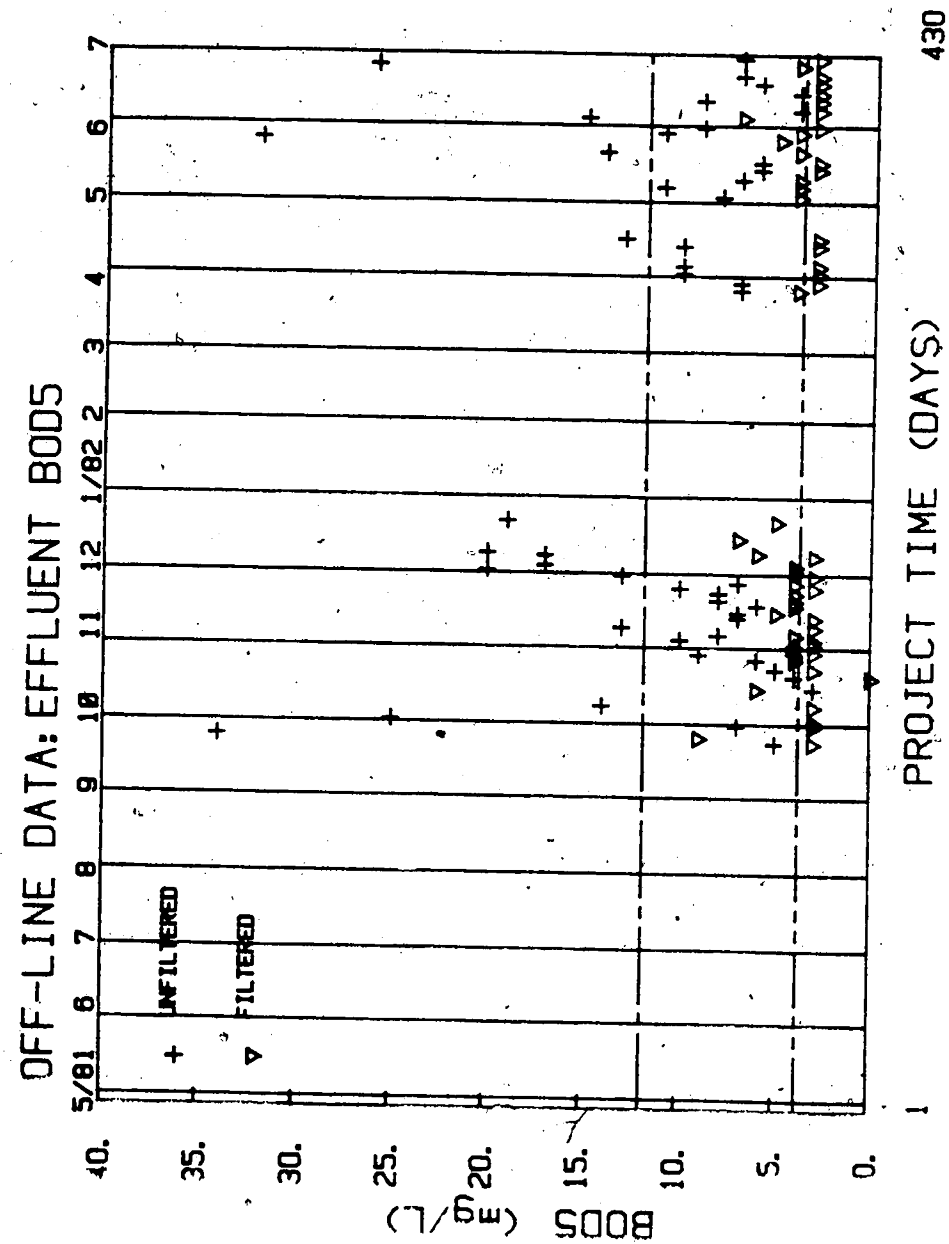
OFF-LINE DATA PLOTS

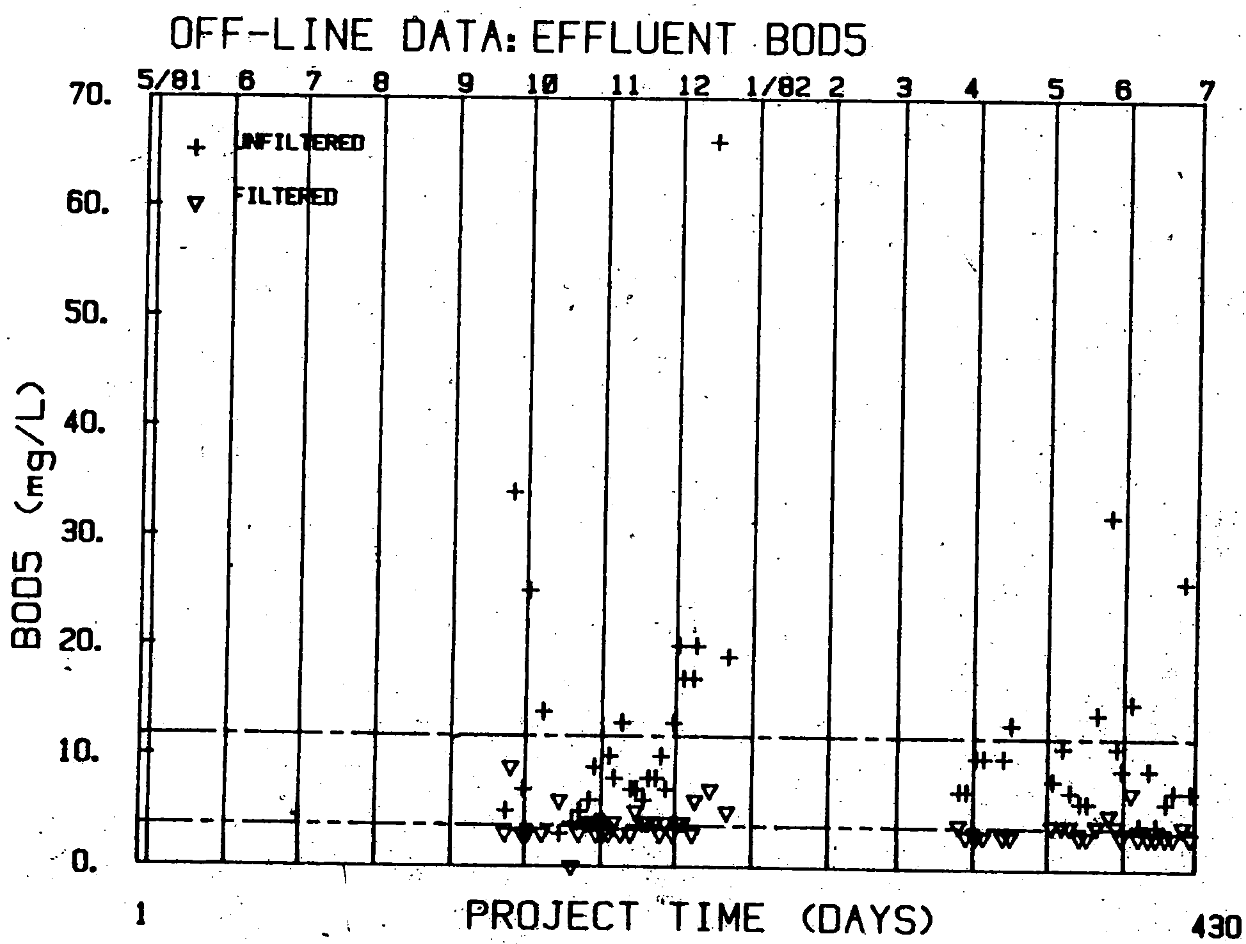


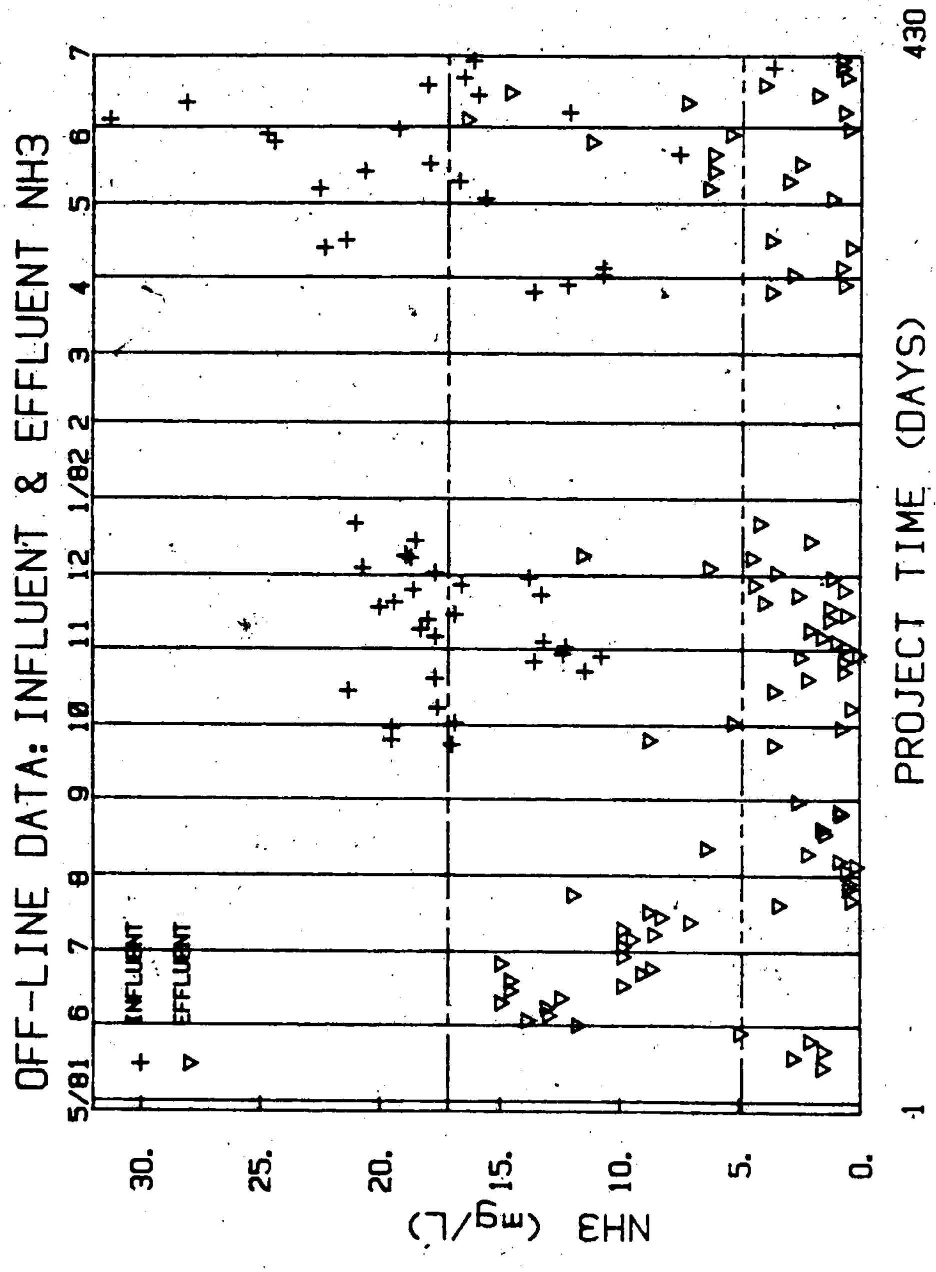
OFF-LINE DATA: SRT (DAYS)
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
40.
30.
20.
10.
0.
SRT (DAYS)
PROJECT TIME (DAYS)
430

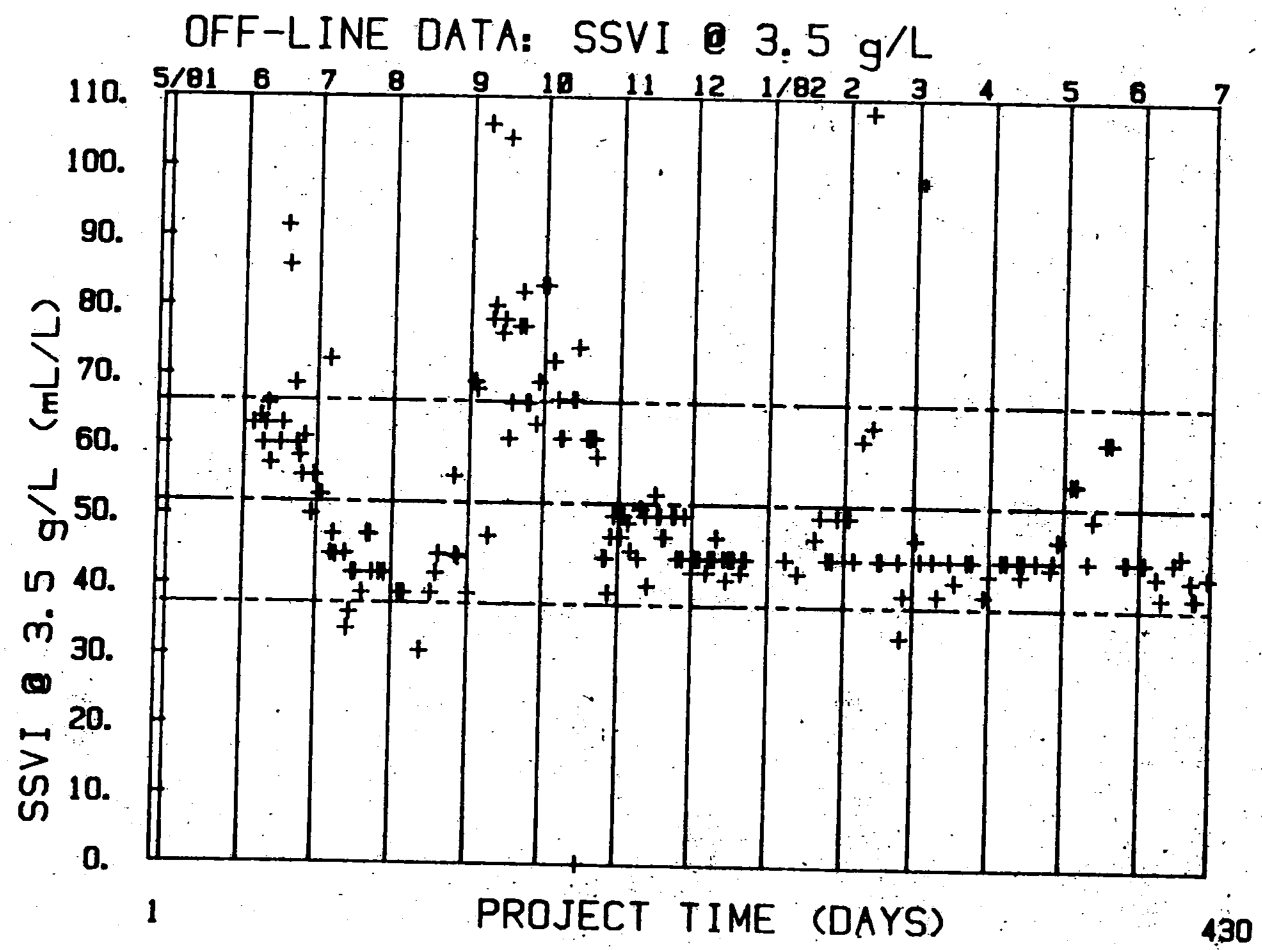

OFF-LINE DATA: SYSTEM EFFLUENT SS
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
70.
60.
40.
30.
20.
PROJECT TIME (DAYS)
430

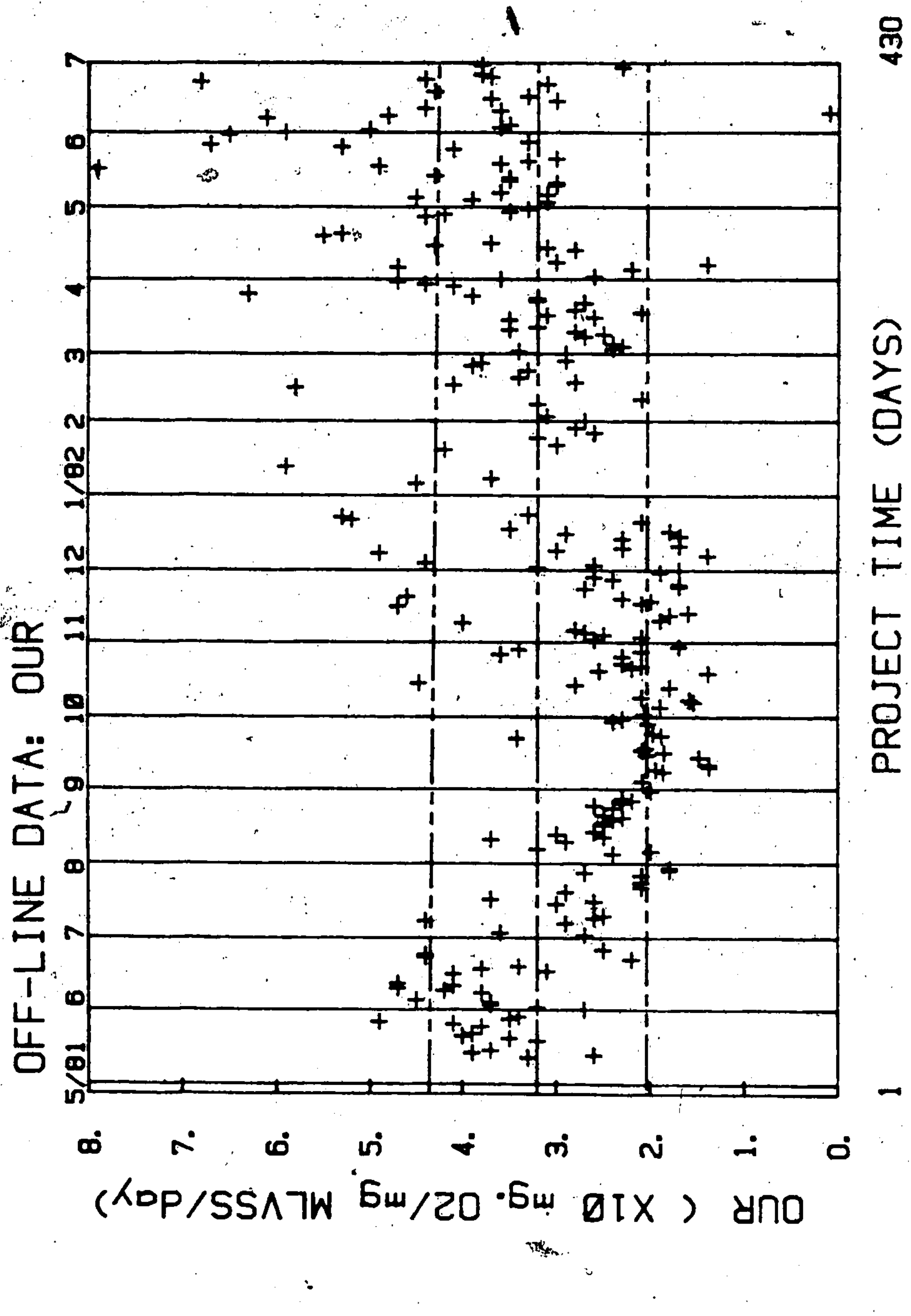

OFF-LINE DATA: INFLUENT BOD5
+ UNFILTERED
▷ FILTERED
5/81 6 7 8 9 10 11 12 1/82 2 3 4 5 6 7
1
430
PROJECT TIME (DAYS)
180. 160. 140. 120. 100. 80. 60. 40. 20. 0.
BOD5 (mg/L)


OFF-LINE DATA: EFFLUENT BOD5
+ UNFILTERED
▽ FILTERED
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
1
430
PROJECT TIME (DAYS)
40.
35.
30.
25.
20.
15.
10.
5.
0.
BOD5 (mg/L)

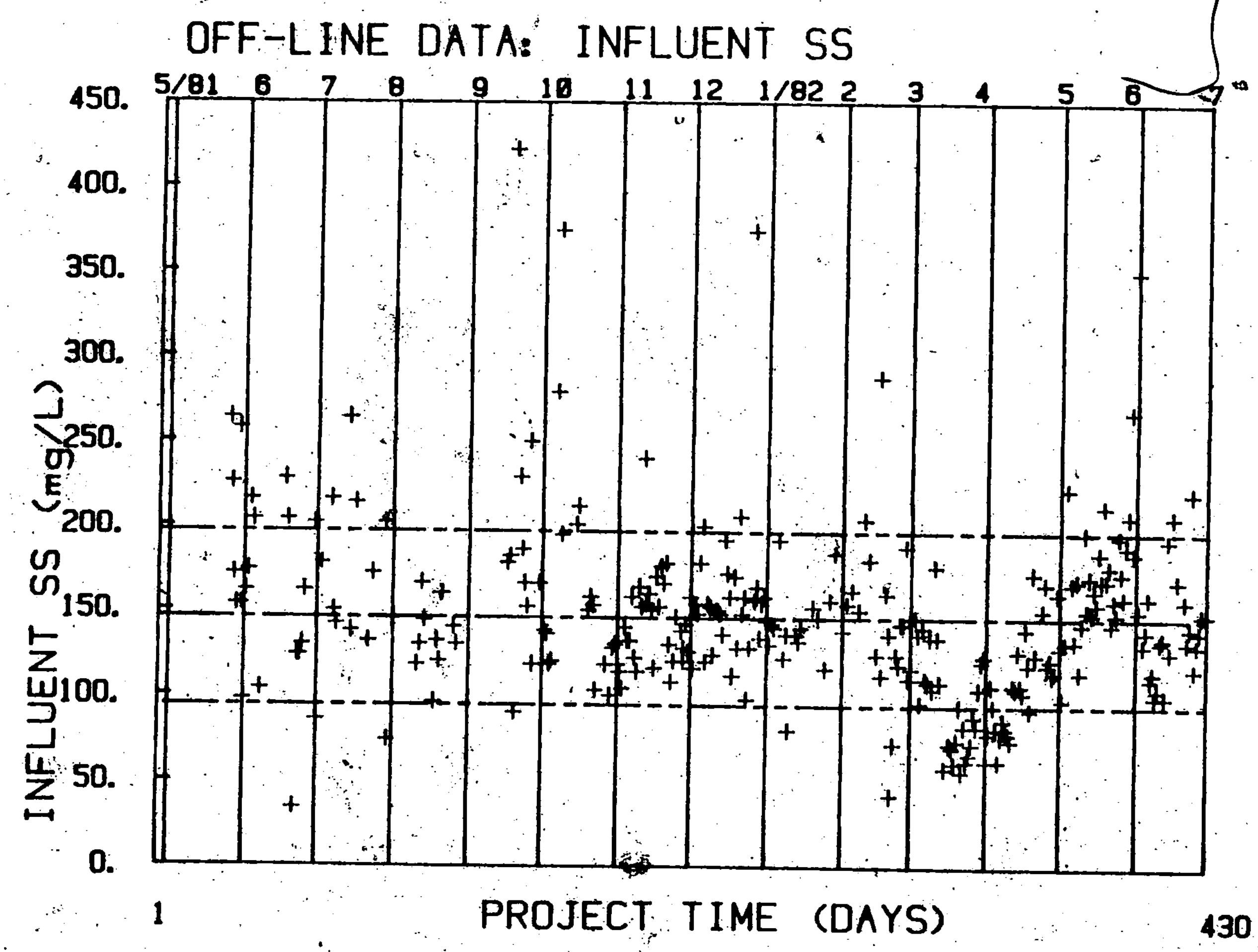

OFF-LINE DATA: EFFLUENT BOD5
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
+ UNFILTERED
▽ FILTERED
70.
60.
50.
40.
30.
20.
10.
0.
BOD5 (mg/L)
1
PROJECT TIME (DAYS)
430


OFF-LINE DATA: INFLUENT & EFFLUENT NH3
INFLUENT
EFFLUENT
NH3 (mg/L)
30.
25.
20.
15.
10.
5.
0.
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
PROJECT TIME (DAYS)
1
430

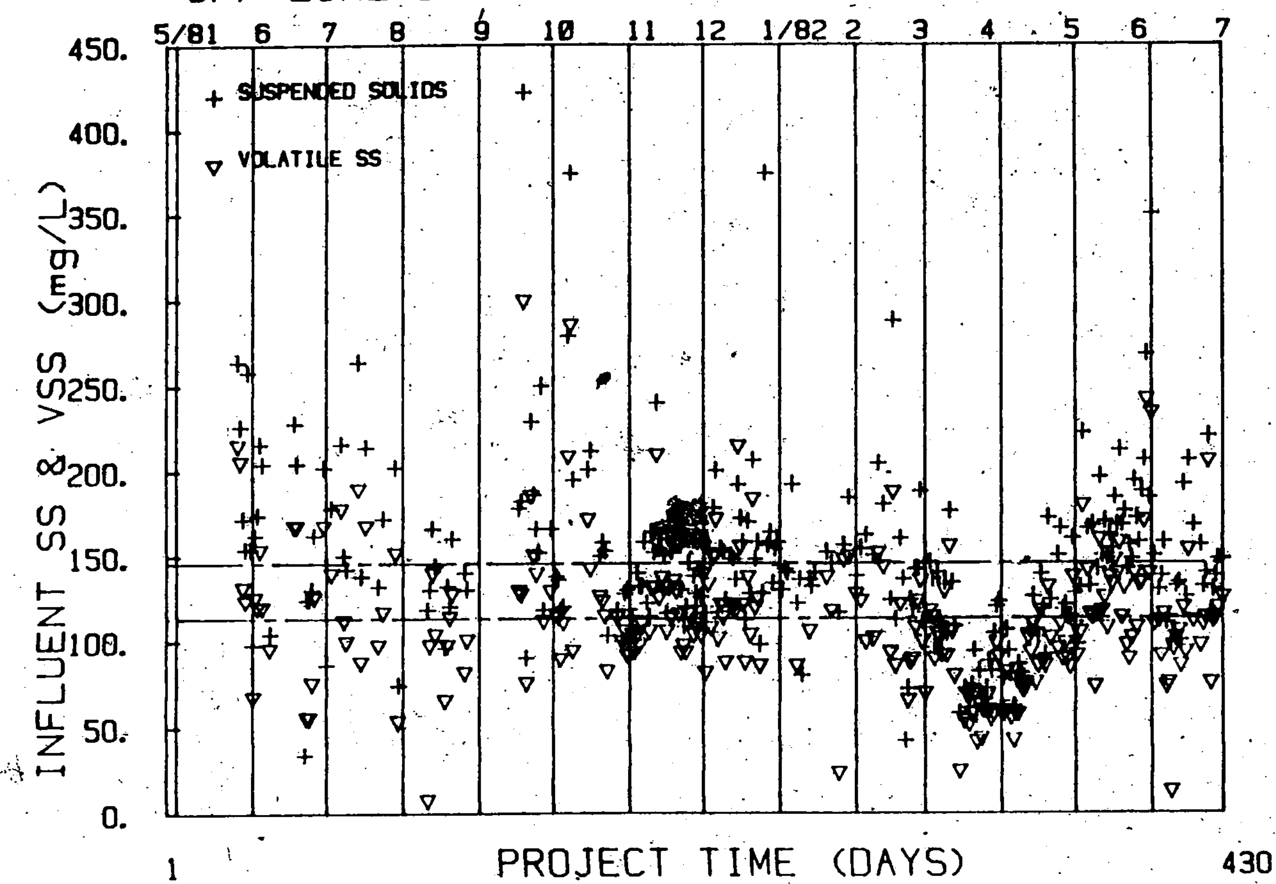

OFF-LINE DATA: SSVI @ 3.5 g/L
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
110.
100.
90.
80.
70.
60.
50.
40.
30.
20.
10.
0.
SSVI @ 3.5 g/L (mL/L)
1
PROJECT TIME (DAYS)
430

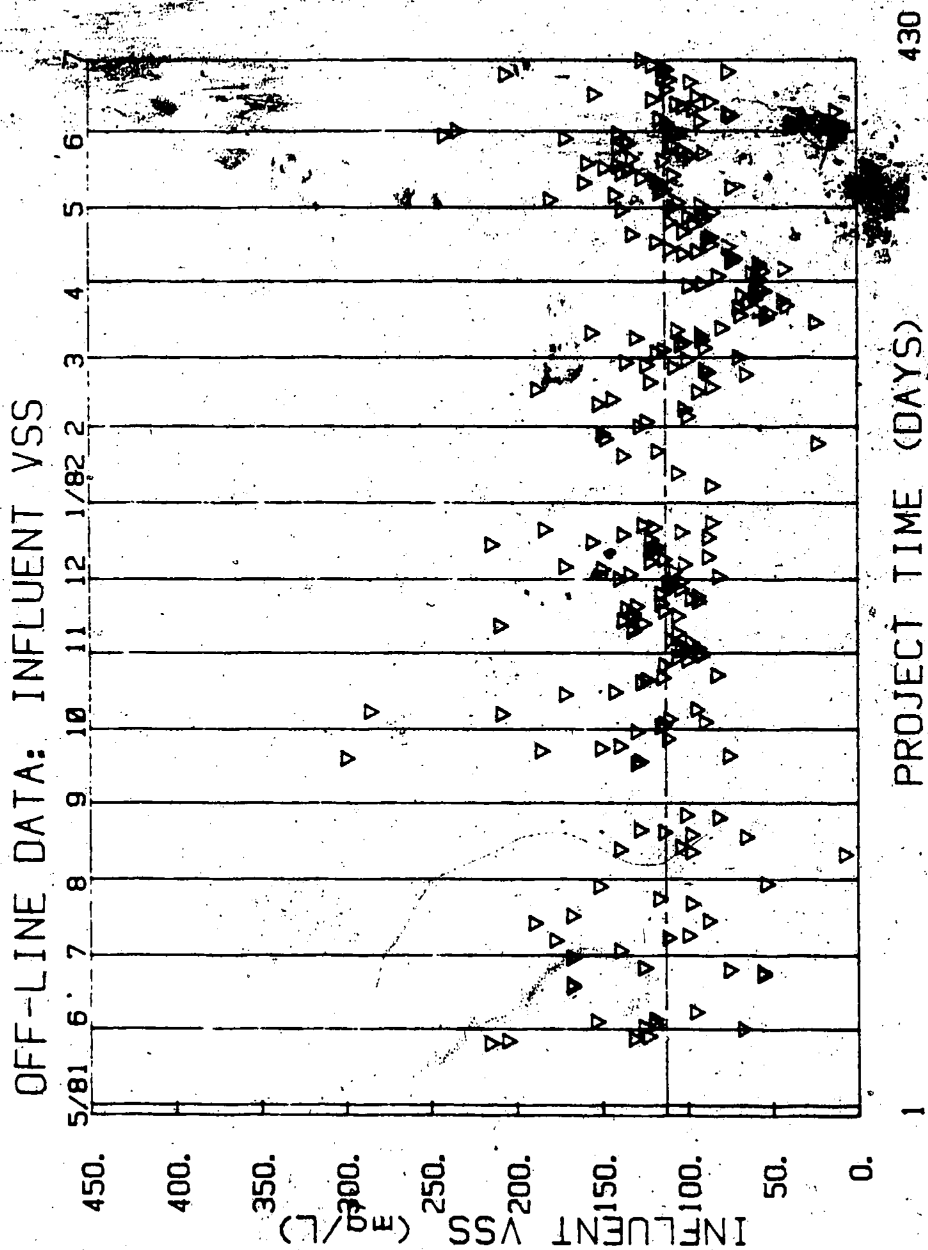


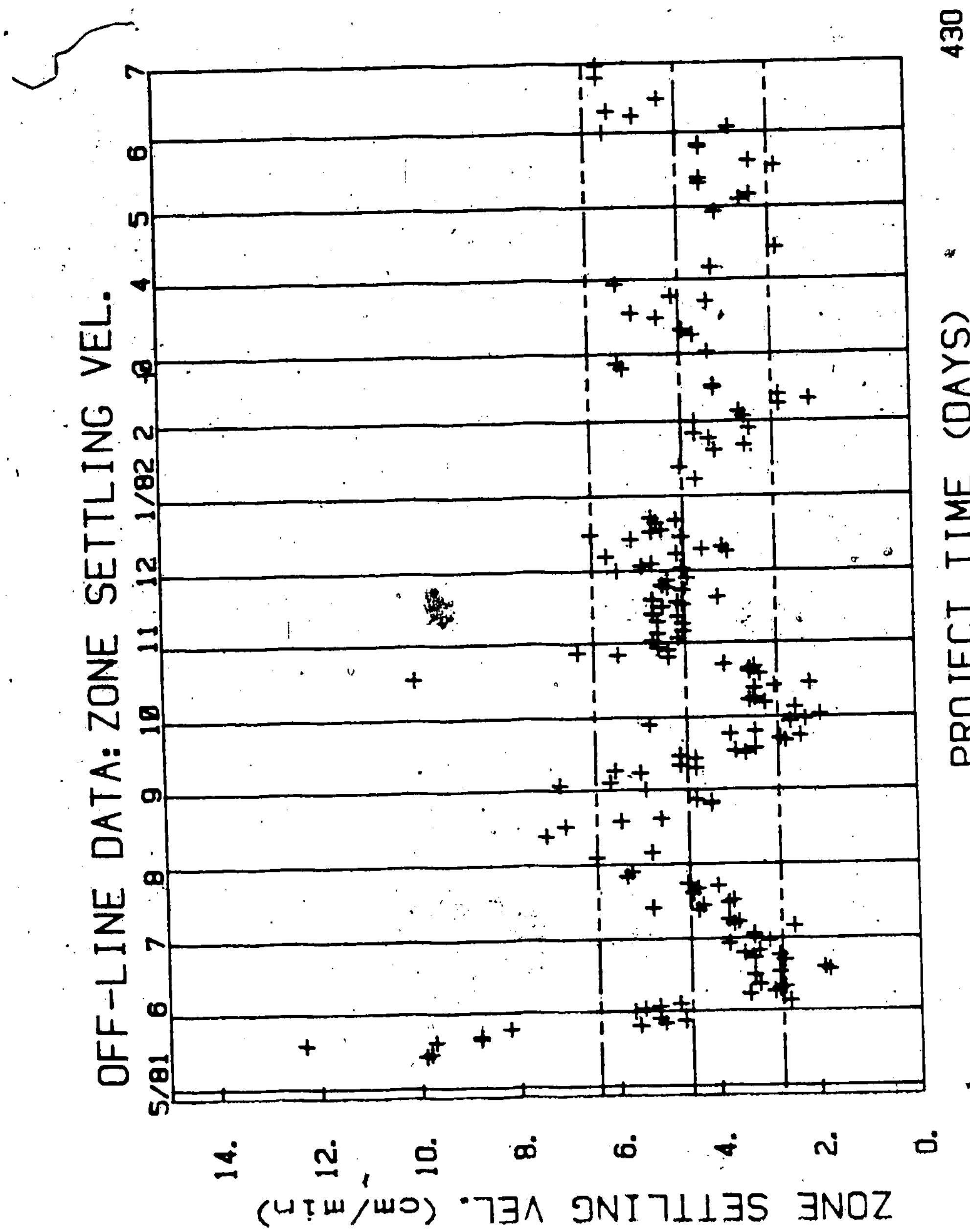
OFF-LINE DATA: OUR
PROJECT TIME (DAYS)
OUR (X10 mg. O2/mg MLVSS/day)
8.
7.
6.
5.
4.
3.
2.
1.
0.
1
430
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7

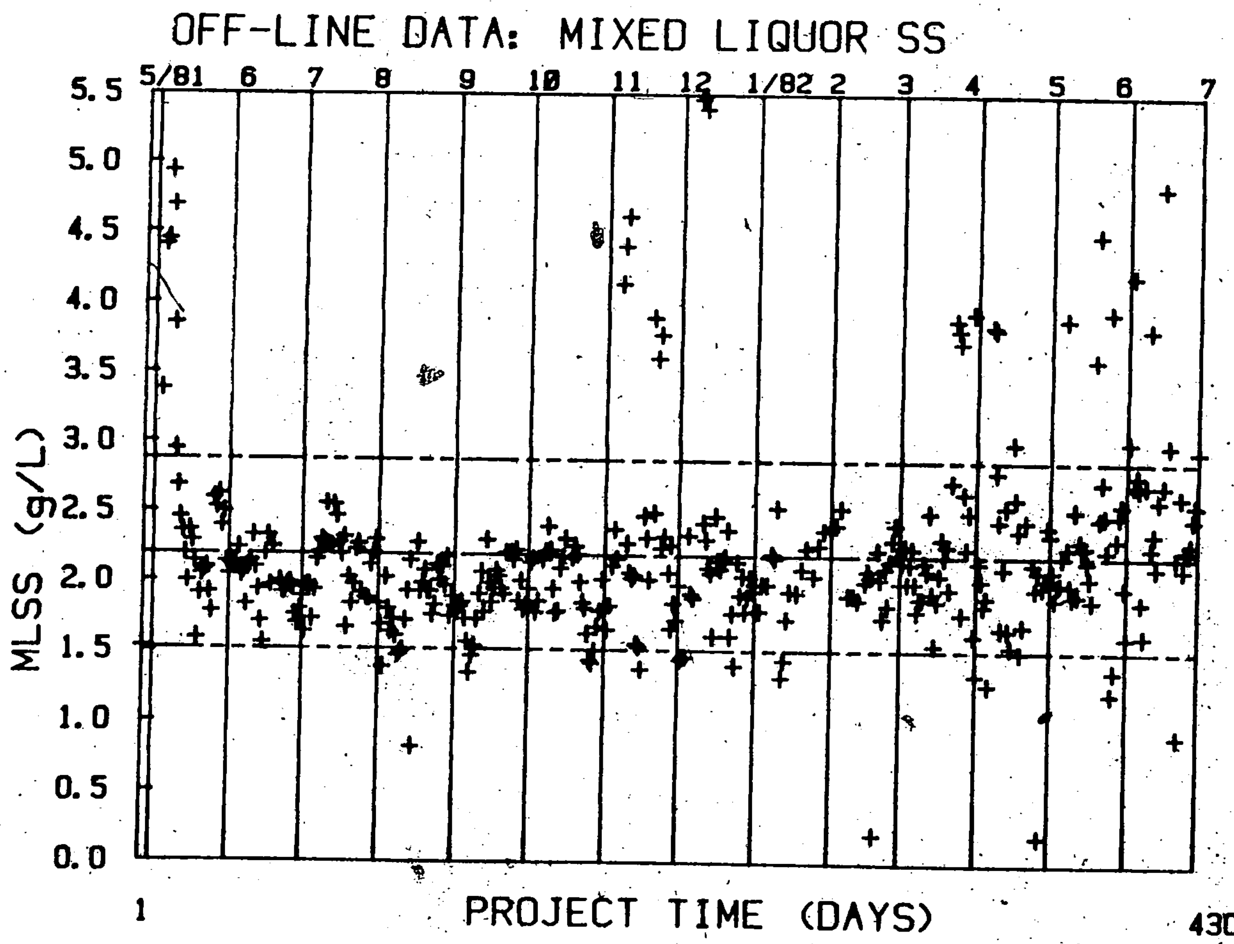

OFF-LINE DATA: INFLUENT SS
5/81 6 7 8 9 10 11 12 1/82 2 3 4 5 6 7
450.
400.
350.
300.
250.
200.
150.
100.
50.
0.
INFLUENT SS (mg/L)
1
PROJECT TIME (DAYS)
430

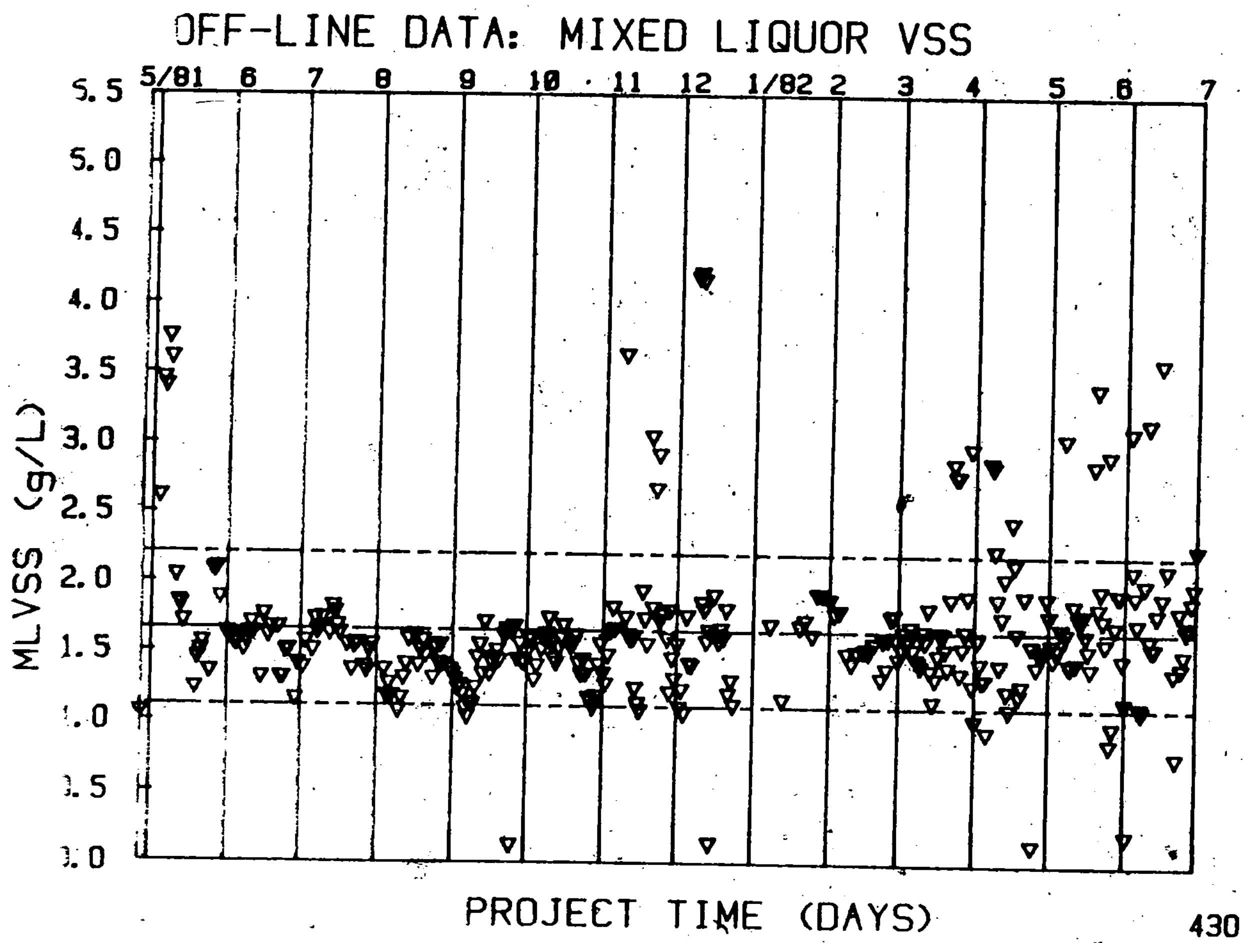
OFF-LINE DATA: INFLUENT SS & VSS



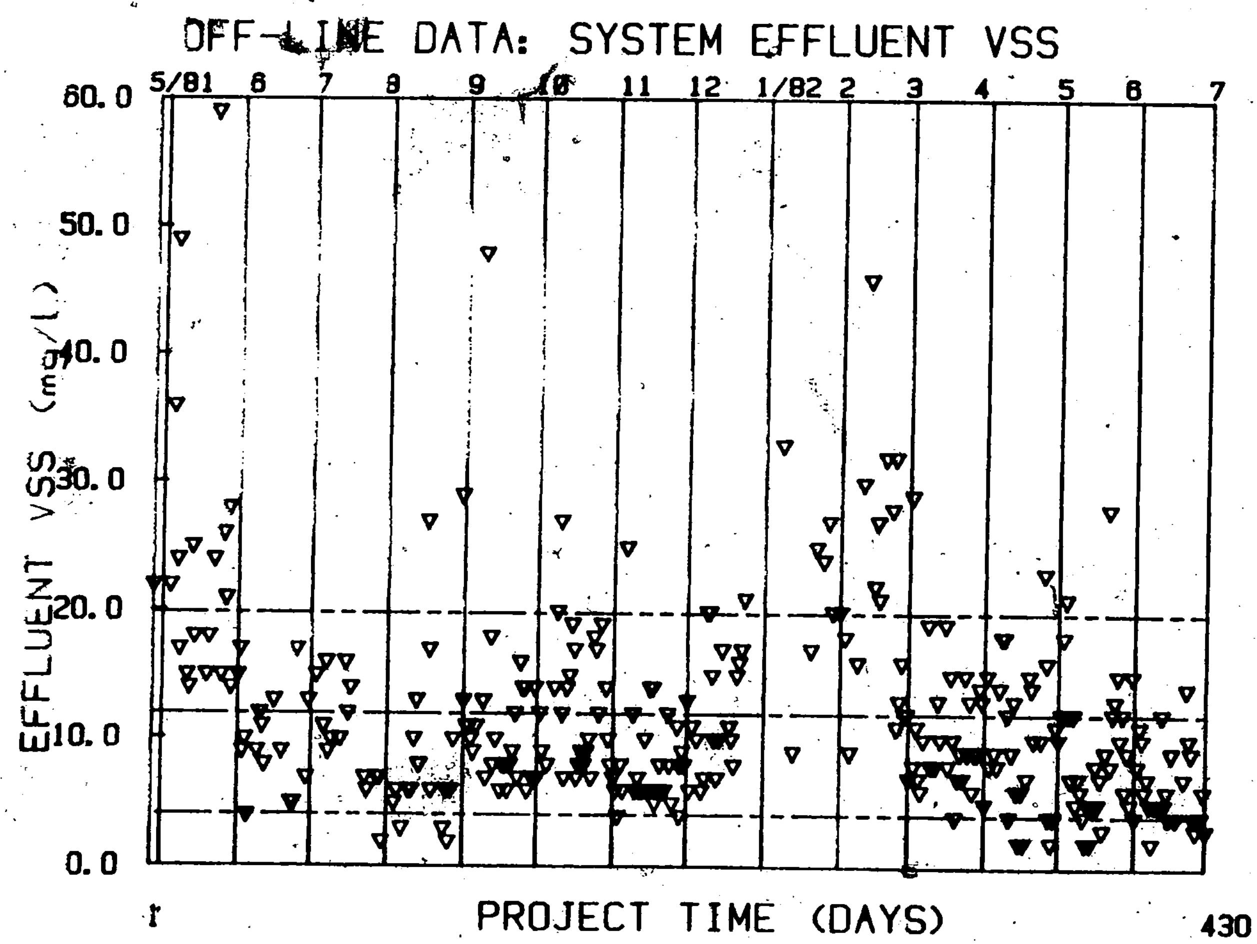

OFF-LINE DATA: INFLUENT VSS
INFLUENT VSS (mg/L)
450.
400.
350.
300.
250.
200.
150.
100.
50.
0.
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
1
430
PROJECT TIME (DAYS)


OFF-LINE DATA: ZONE SETTLING VEL.
ZONE SETTLING VEL. (cm/min)
14.
12.
10.
8.
6.
4.
2.
0.
PROJECT TIME (DAYS)
1
430
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7


OFF-LINE DATA: MIXED LIQUOR SS
5/81 6 7 8 9 10 11 12 1/82 2 3 4 5 6 7
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
MLSS (g/L)
1
PROJECT TIME (DAYS)
430



OFF-LINE DATA: MIXED LIQUOR VSS
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
MLVSS (g/L)
PROJECT TIME (DAYS)
430

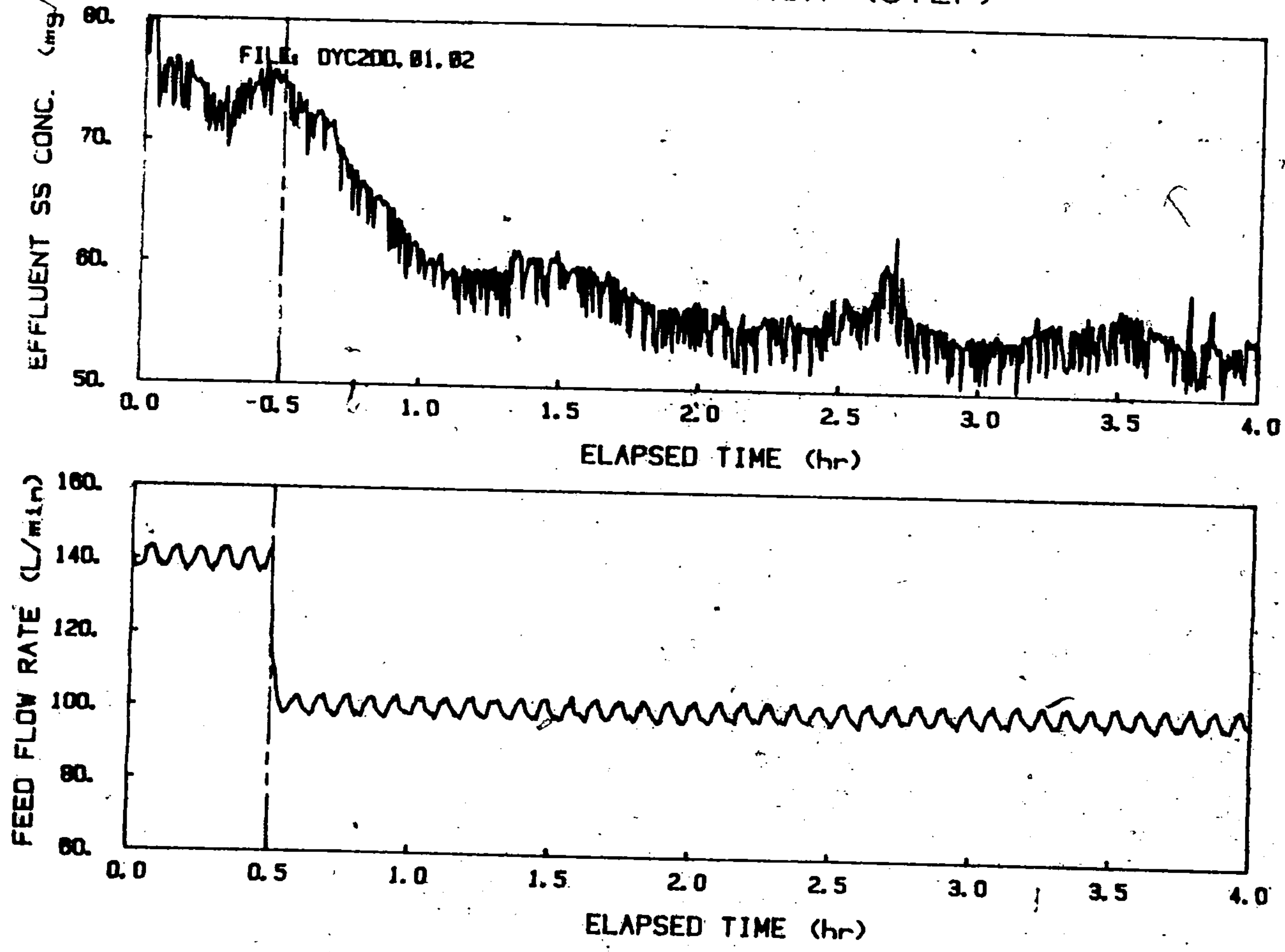


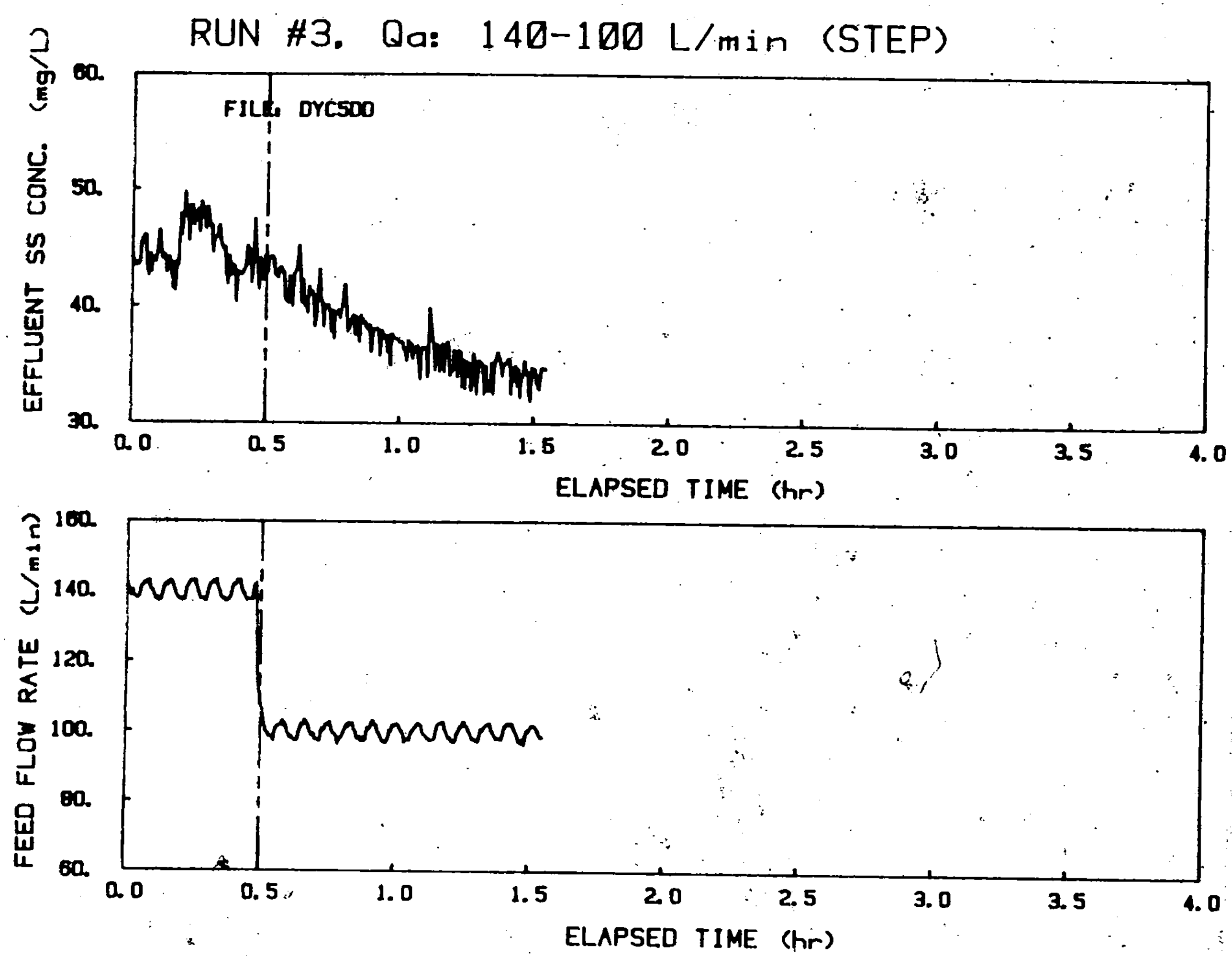
OFF-LINE DATA: SYSTEM EFFLUENT VSS
5/81
6
7
8
9
10
11
12
1/82
2
3
4
5
6
7
60.0
50.0
40.0
30.0
20.0
10.0
0.0
EFFLUENT VSS (mg/L)
PROJECT TIME (DAYS)
430

APPENDIX D

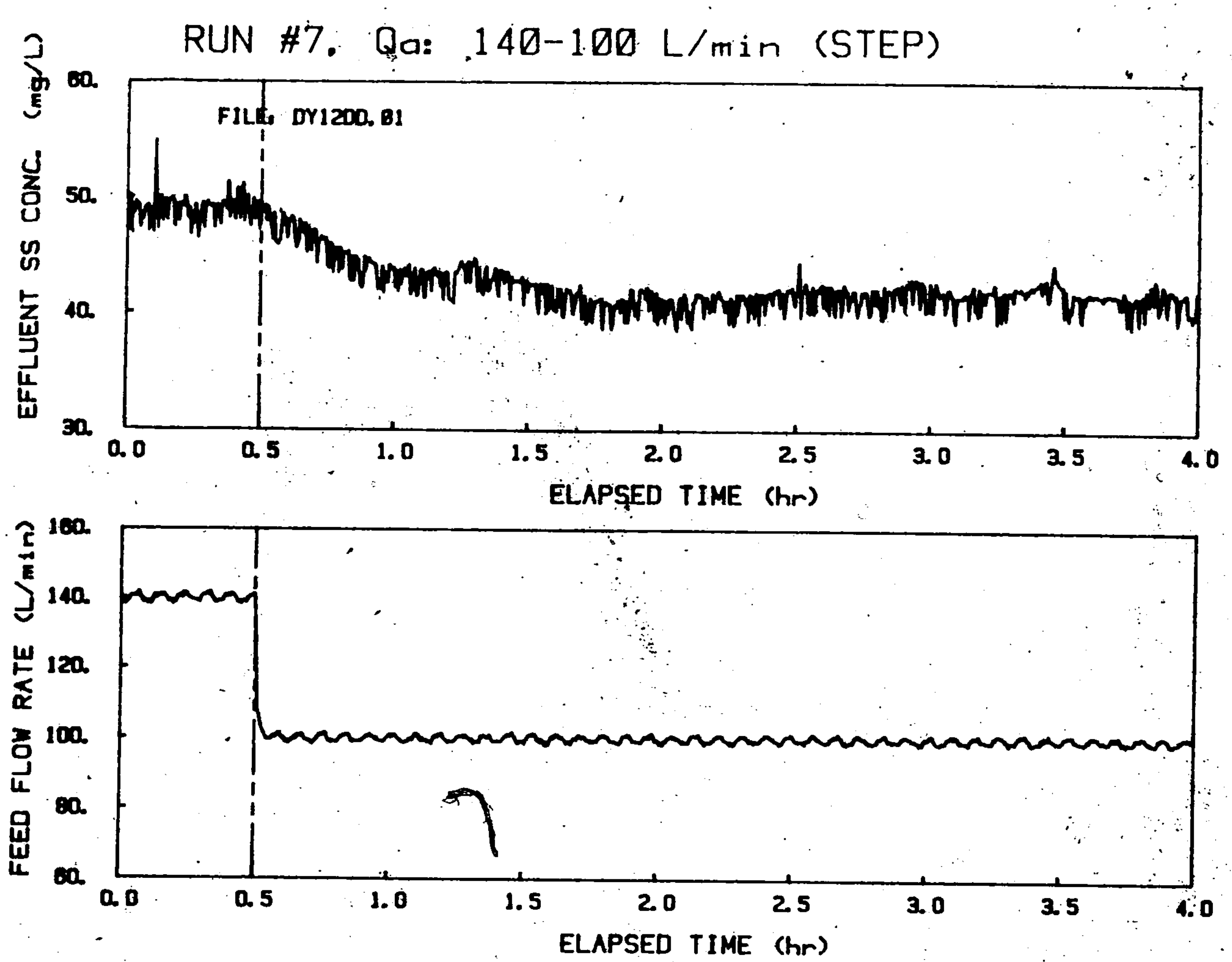
STEP TEST PLOTS

RUN #1, Qa: 140-100 L/min (STEP)

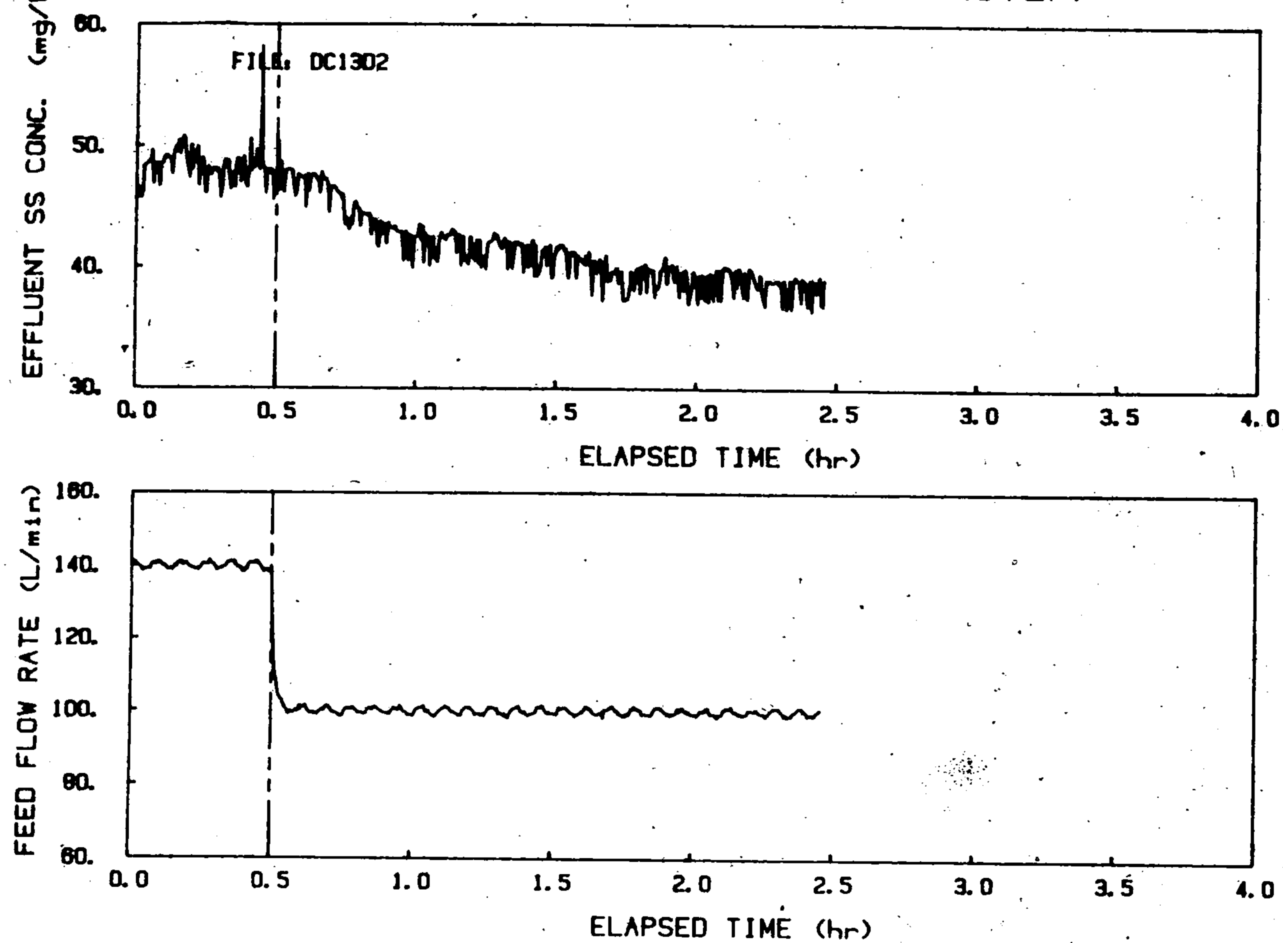


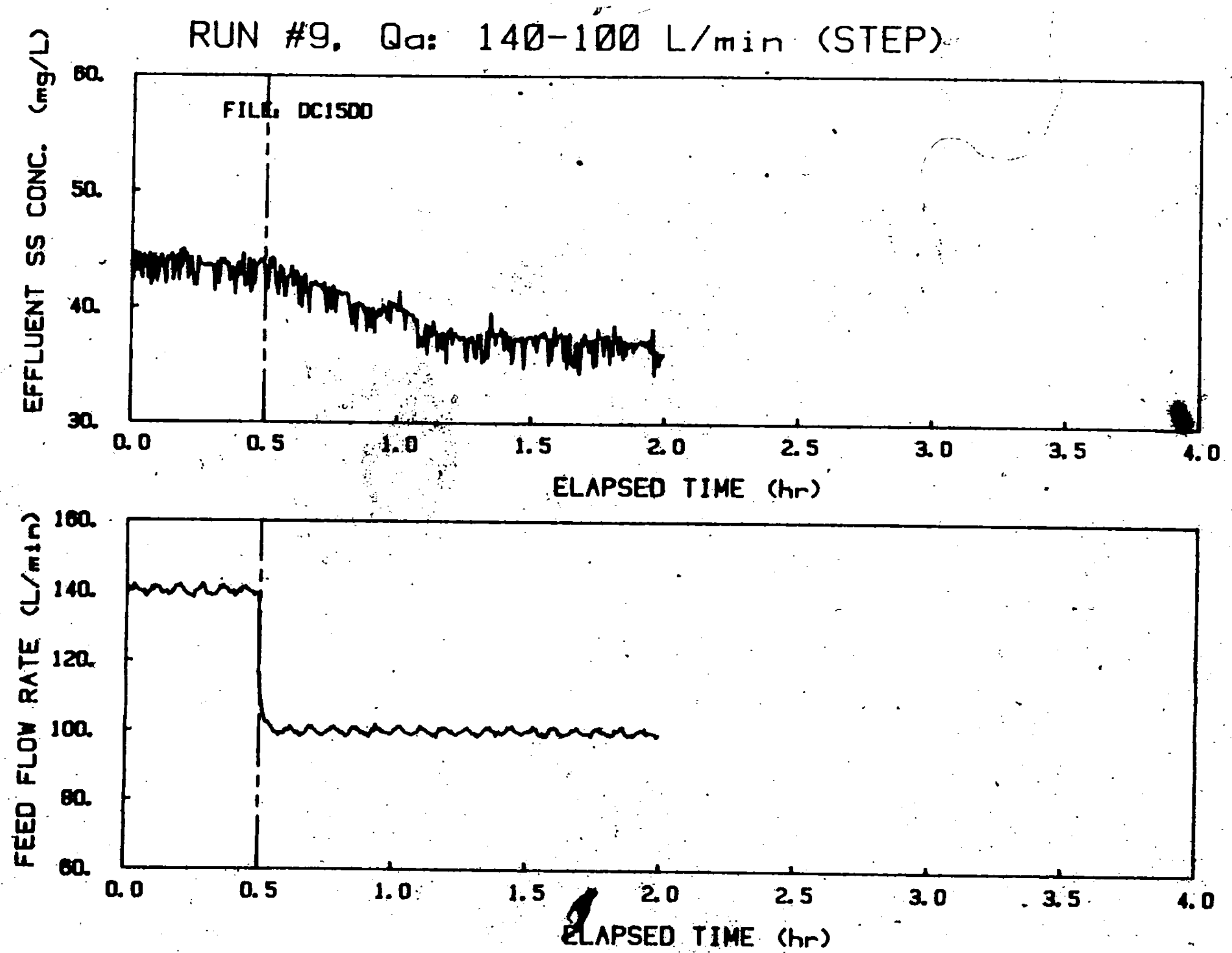


RUN #3. Qa: 140-100 L/min (STEP)
FILE: DYC500
EFFLUENT SS CONC. (mg/L)
60.
50.
40.
30.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)
FEED FLOW RATE (L/min)
160.
140.
120.
100.
80.
60.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)

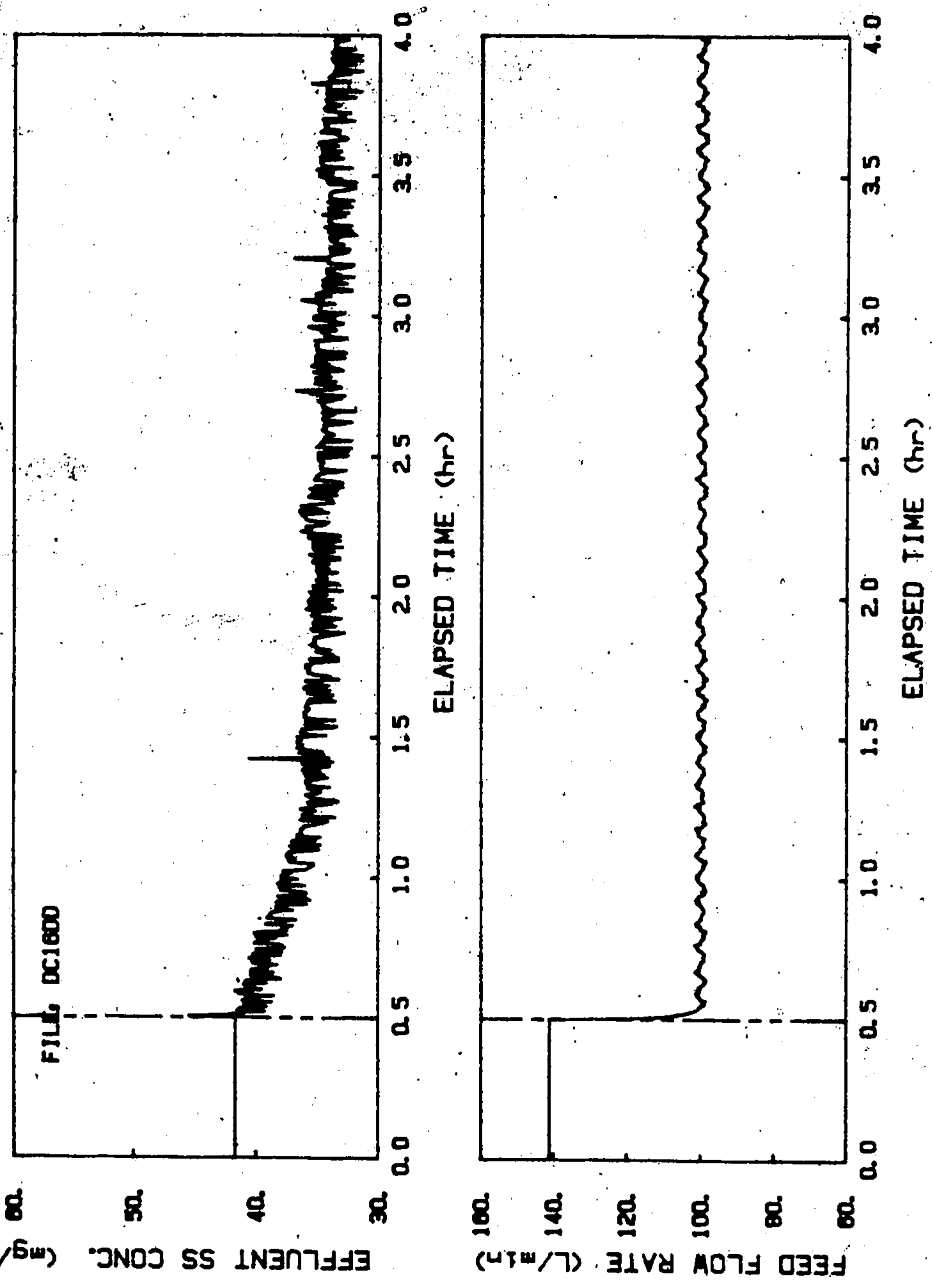

RUN #7. Qa: 140-100 L/min (STEP)
FILE: DY1200.81
EFFLUENT SS CONC. (mg/L)
60.
50.
40.
30.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)
FEED FLOW RATE (L/min)
160.
140.
120.
100.
80.
60.
ELAPSED TIME (hr)

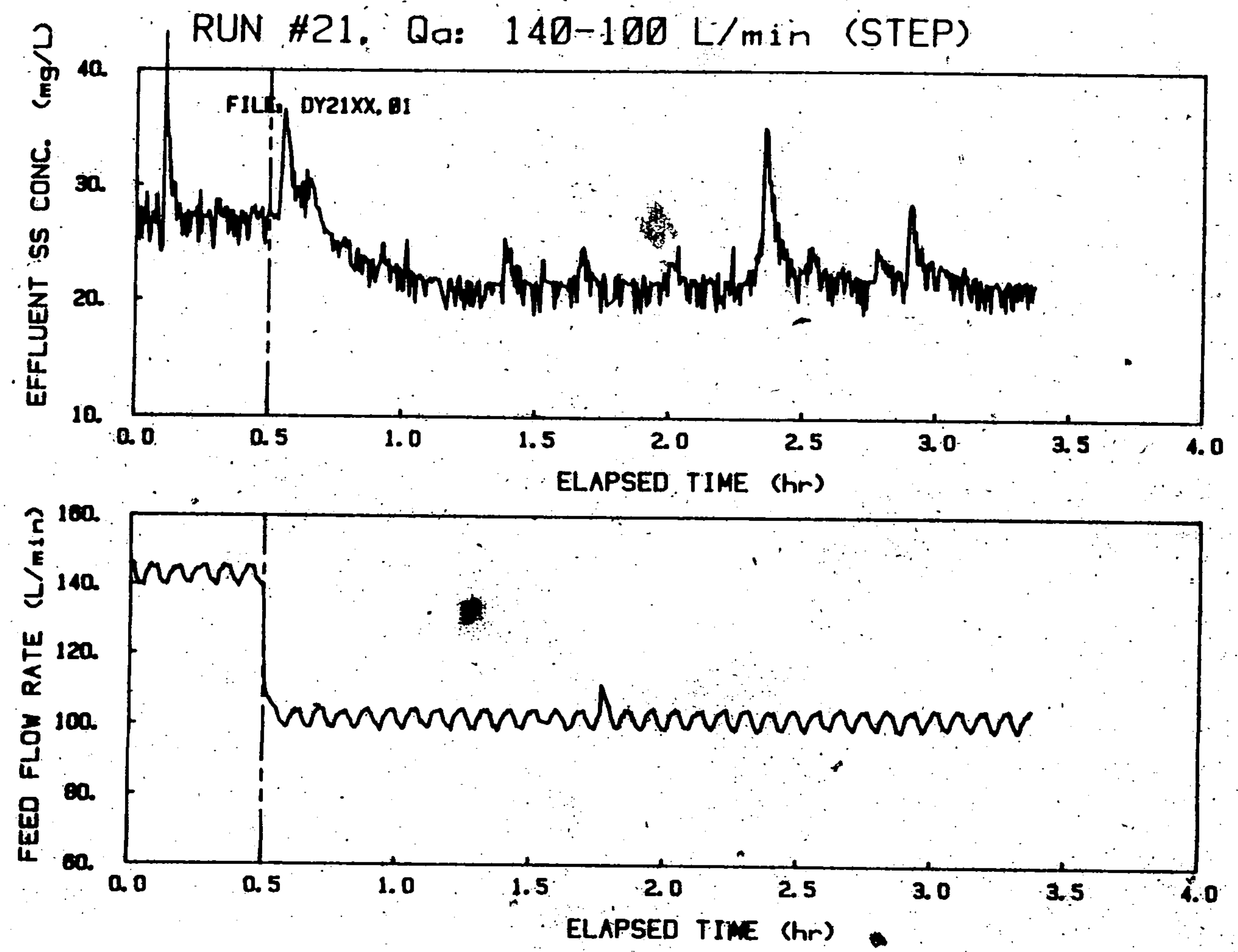
RUN #8 (b), Qa: 140-100 L/min (STEP)



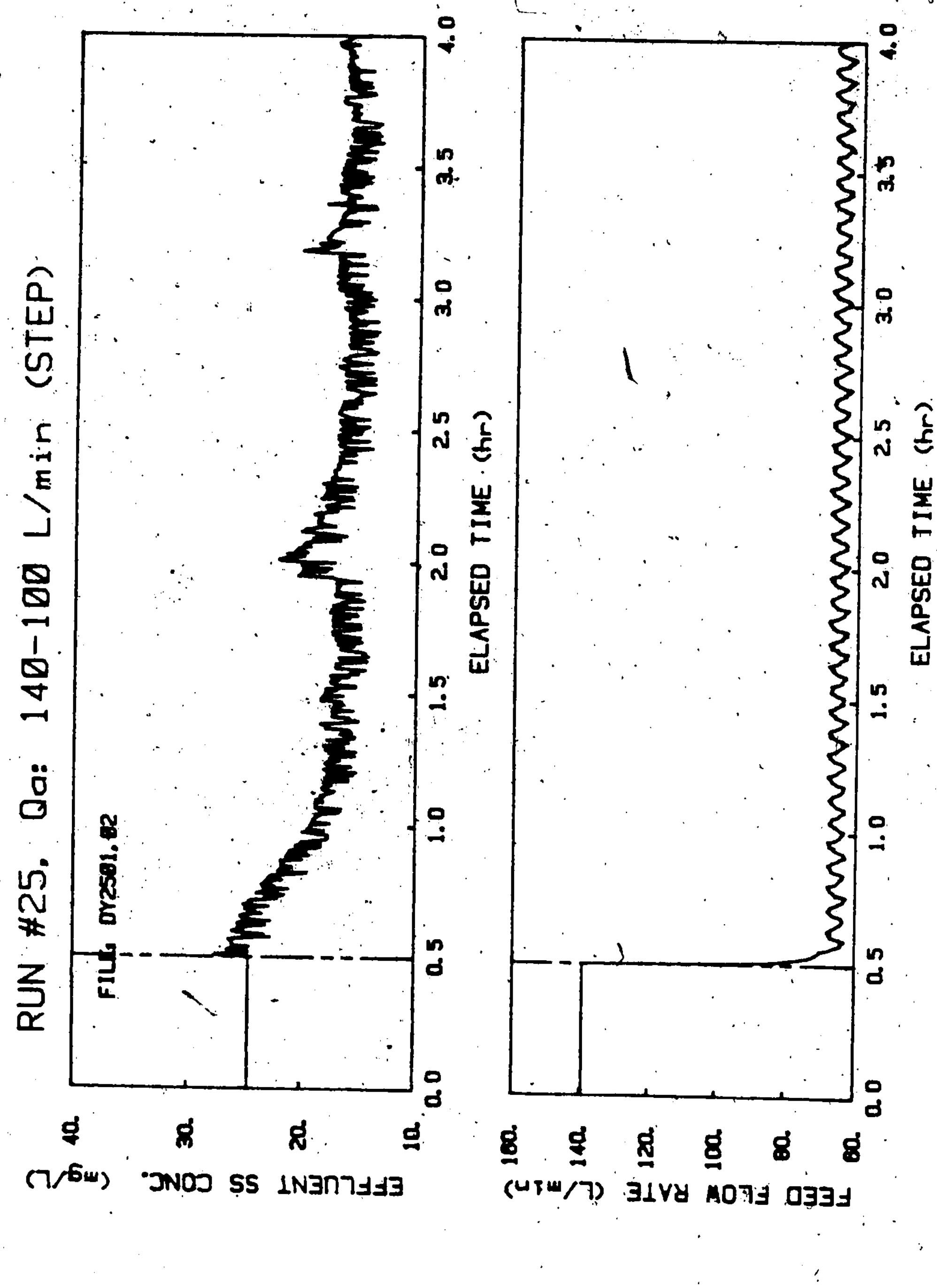

RUN #9, Qa: 140-100 L/min (STEP)
FILE: DC15DD
EFFLUENT SS CONC. (mg/L)
60.
50.
40.
30.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)
FEED FLOW RATE (L/min)
160.
140.
120.
100.
80.
60.
ELAPSED TIME (hr)

RUN #10, Qa: 140-100 L/min (STEP)

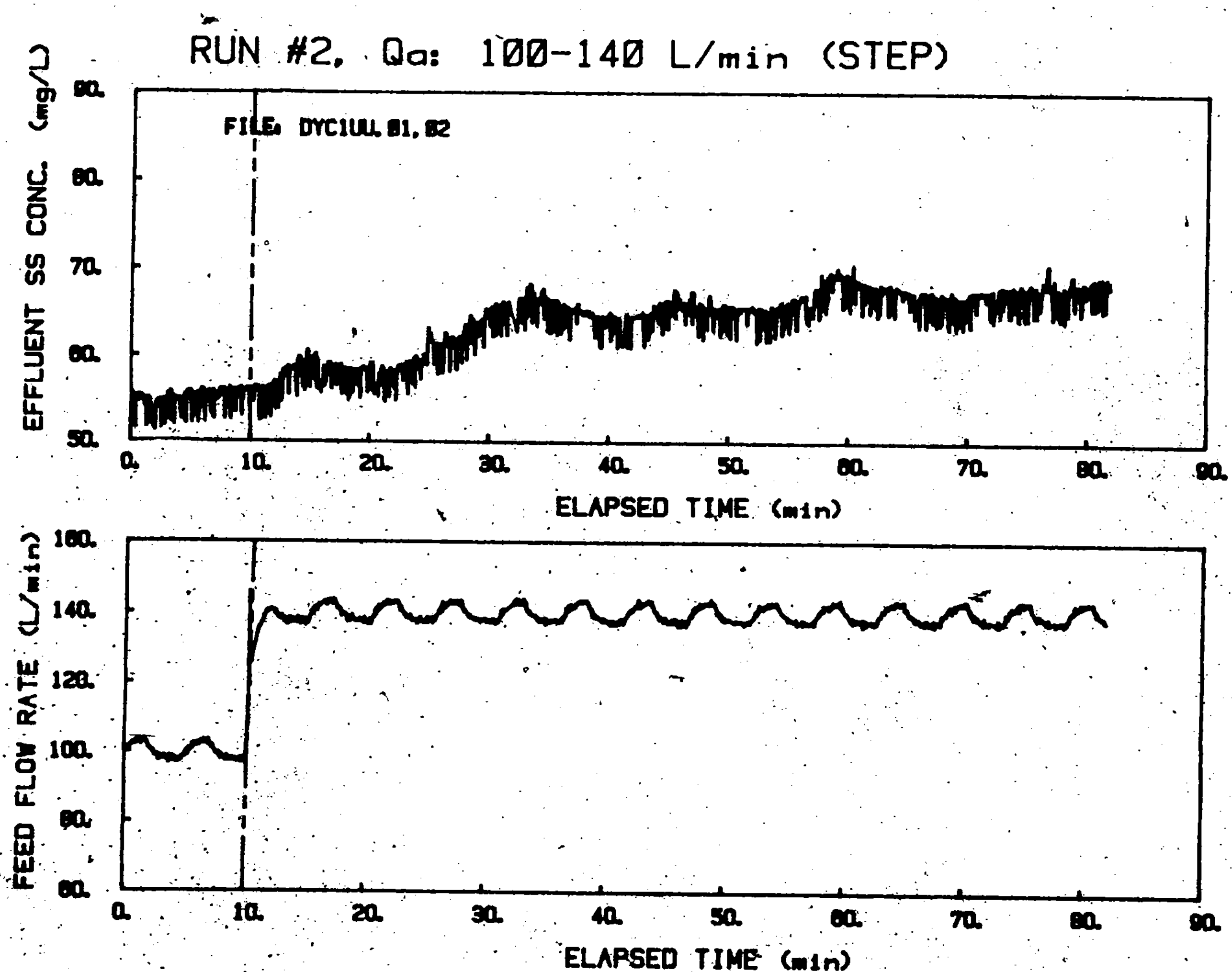


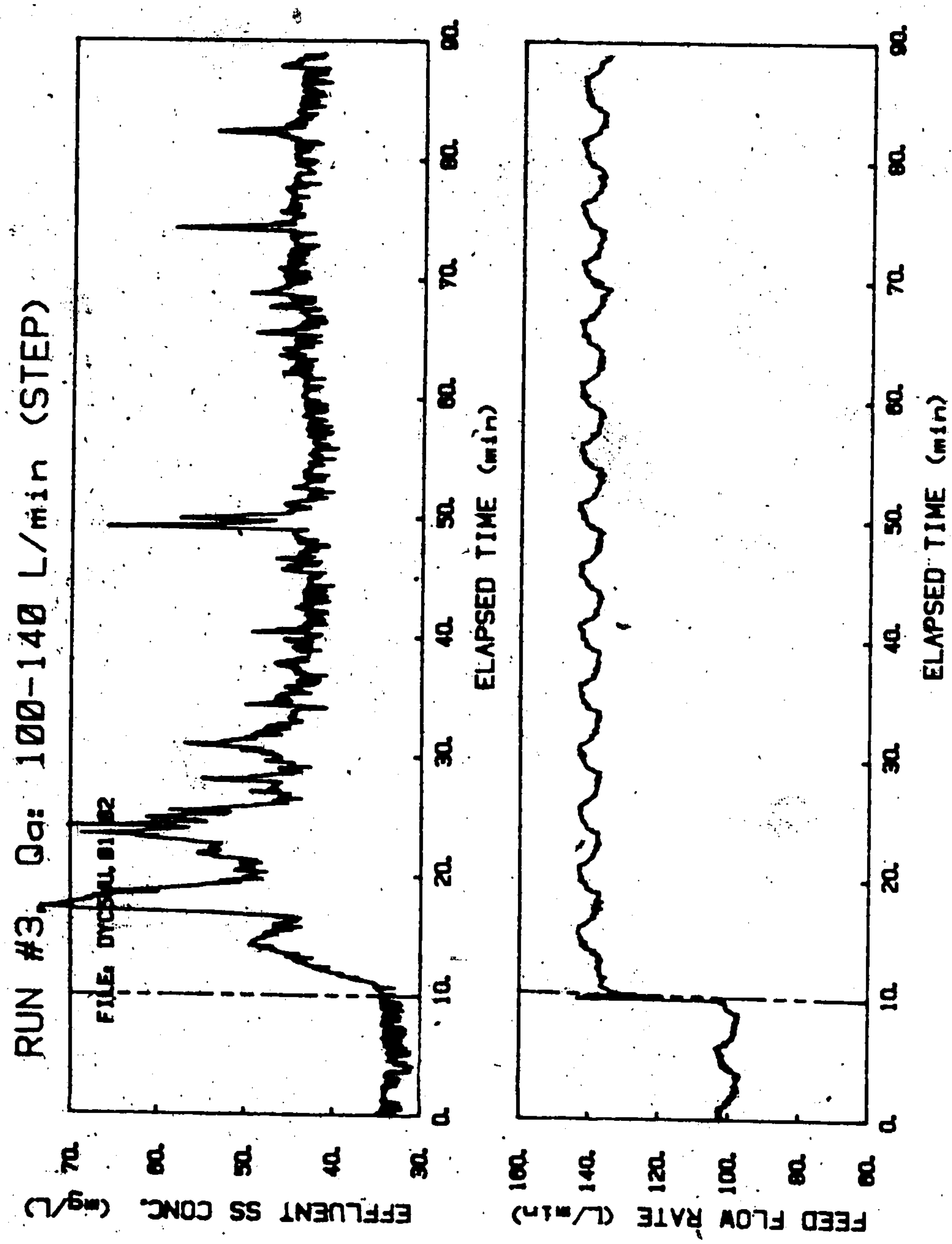


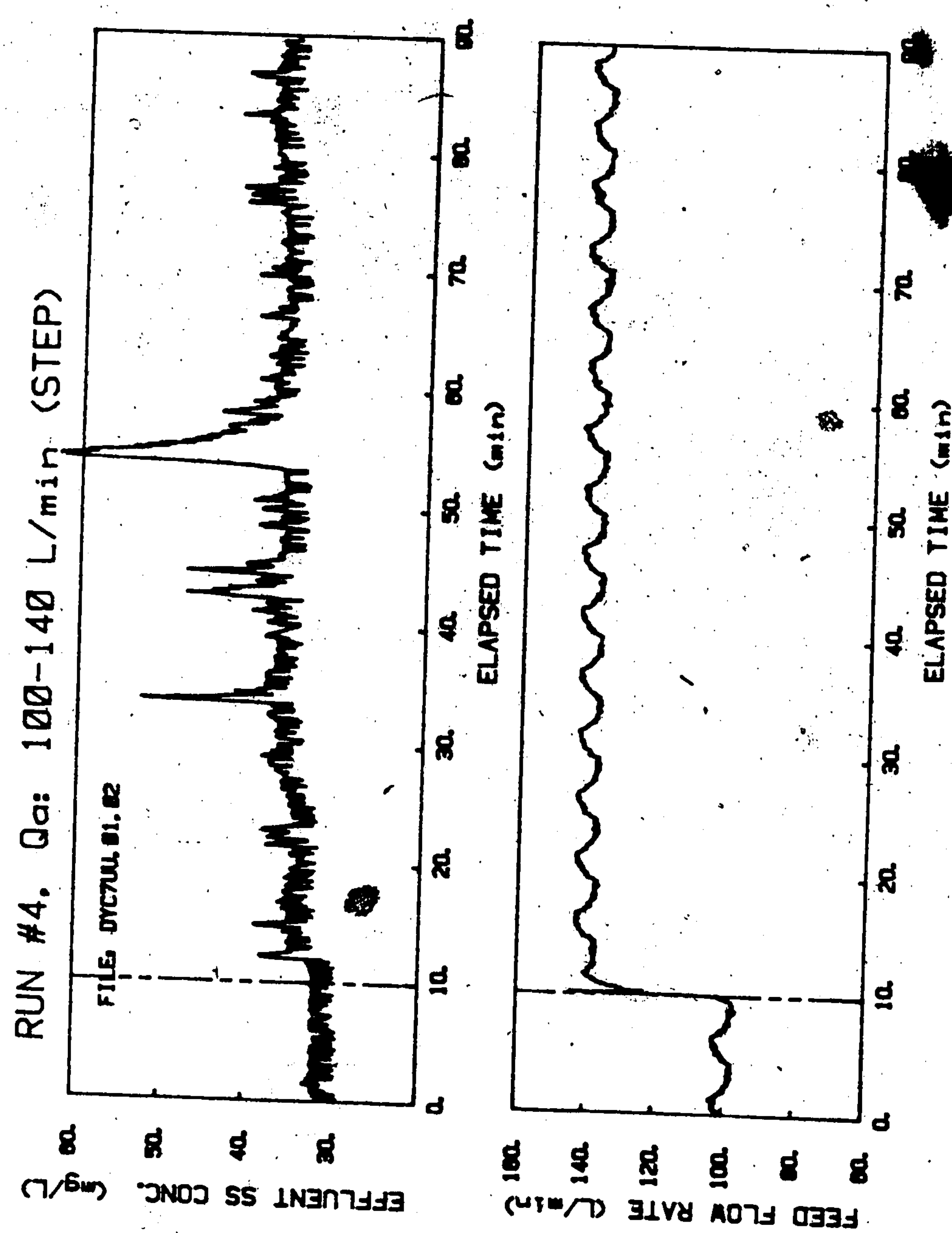
RUN #21, Qa: 140-100 L/min (STEP)
FILE, DY21XX.01
EFFLUENT SS CONC. (mg/L)
40.
30.
20.
10.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)
FEED FLOW RATE (L/min)
160.
140.
120.
100.
80.
60.
ELAPSED TIME (hr)



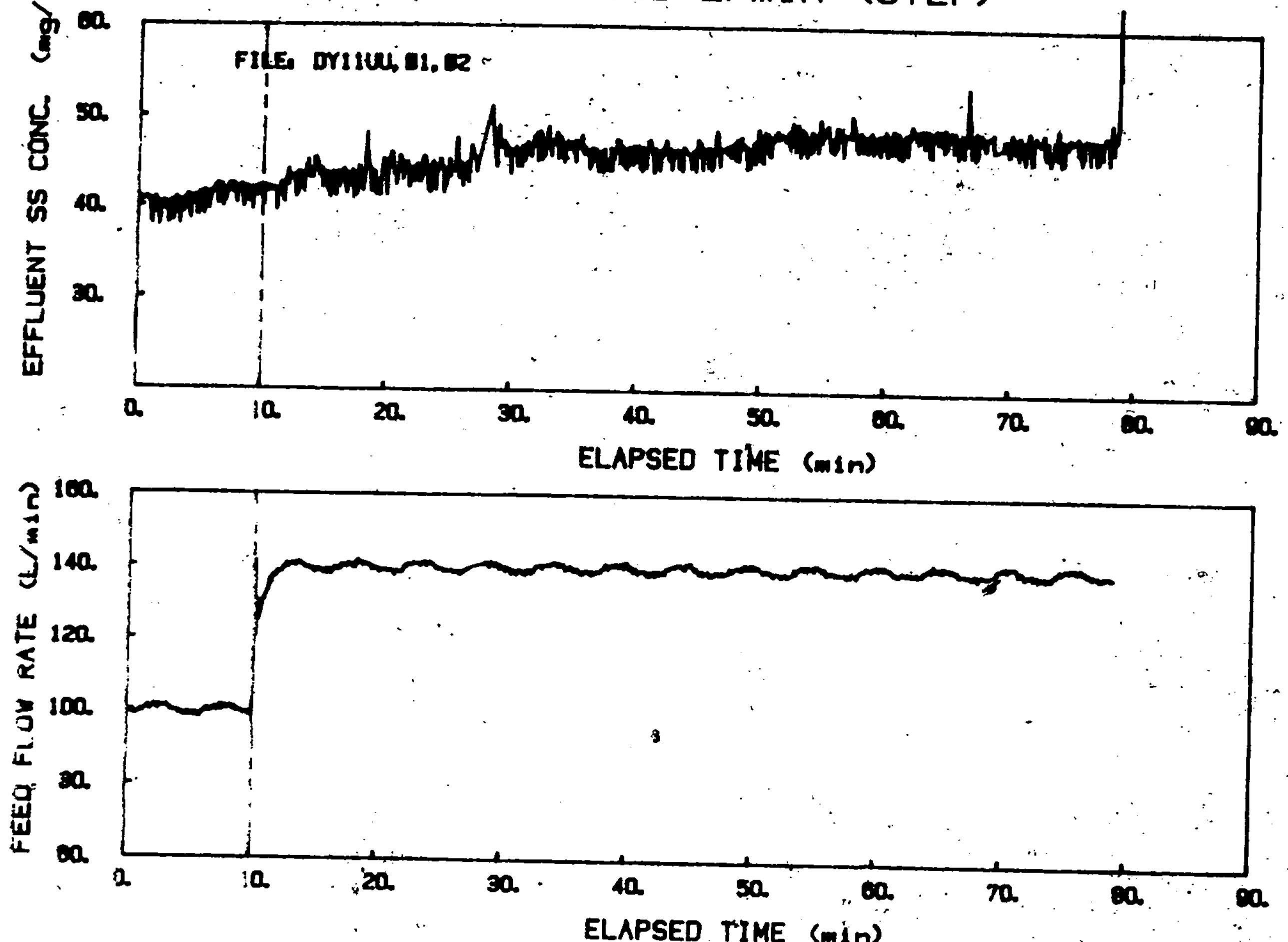
RUN #25. Qa: 140-100 L/min (STEP)
FILE DY2501.02
EFFLUENT SS CONC. (mg/L)
40.
30.
20.
10.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)
FEED FLOW RATE (L/min)
160.
140.
120.
100.
80.
60.
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
ELAPSED TIME (hr)


RUN #2, Qa: 100-140 L/min (STEP)
FILE: DYC1UU.01,02
EFFLUENT SS CONC. (mg/L)
ELAPSED TIME (min)
FEED FLOW RATE (L/min)
ELAPSED TIME (min)

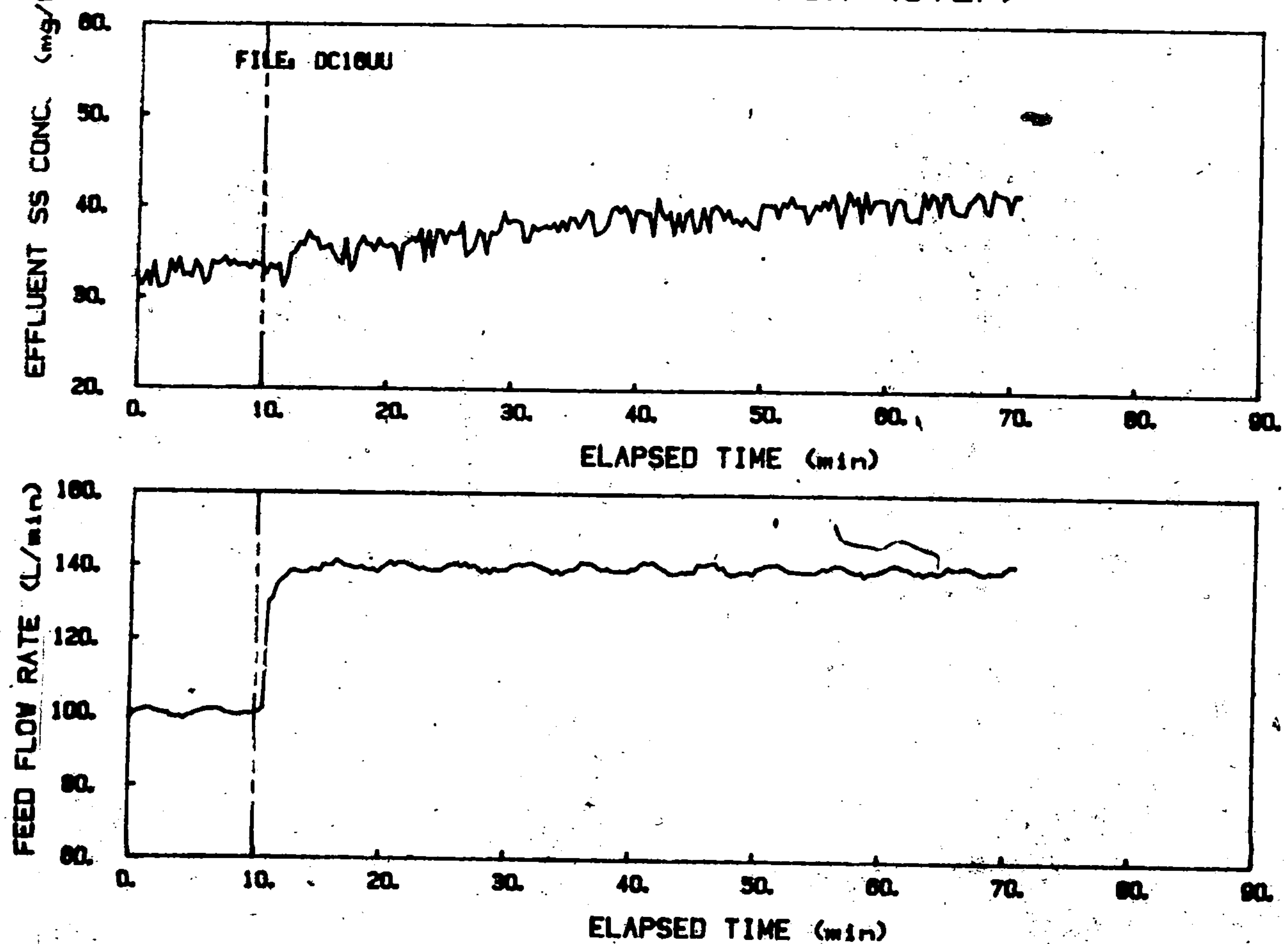

RUN #3 Qa: 100-140 L/min (STEP)
EFFLUENT SS CONC. (mg/L)
ELAPSED TIME (min)
FEED FLOW RATE (L/min)
ELAPSED TIME (min)


RUN #4. Qa: 100-140 L/min (STEP)
FILE: DYC7UUL81.82
EFFLUENT SS CONC. (mg/L)
FEED FLOW RATE (L/min)
ELAPSED TIME (min)
ELAPSED TIME (min)

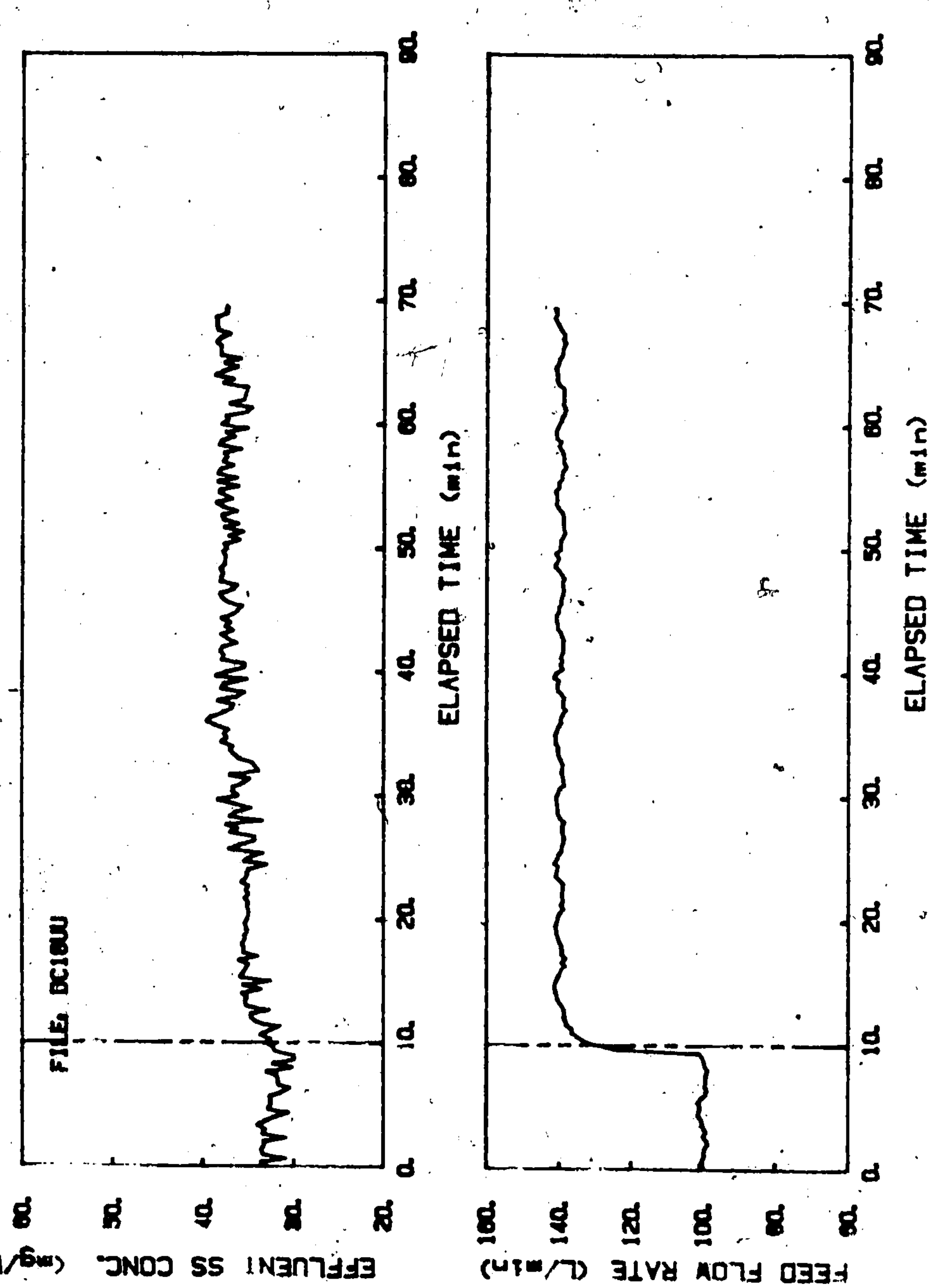
RUN #7. Qa: 100-140 L/min (STEP)



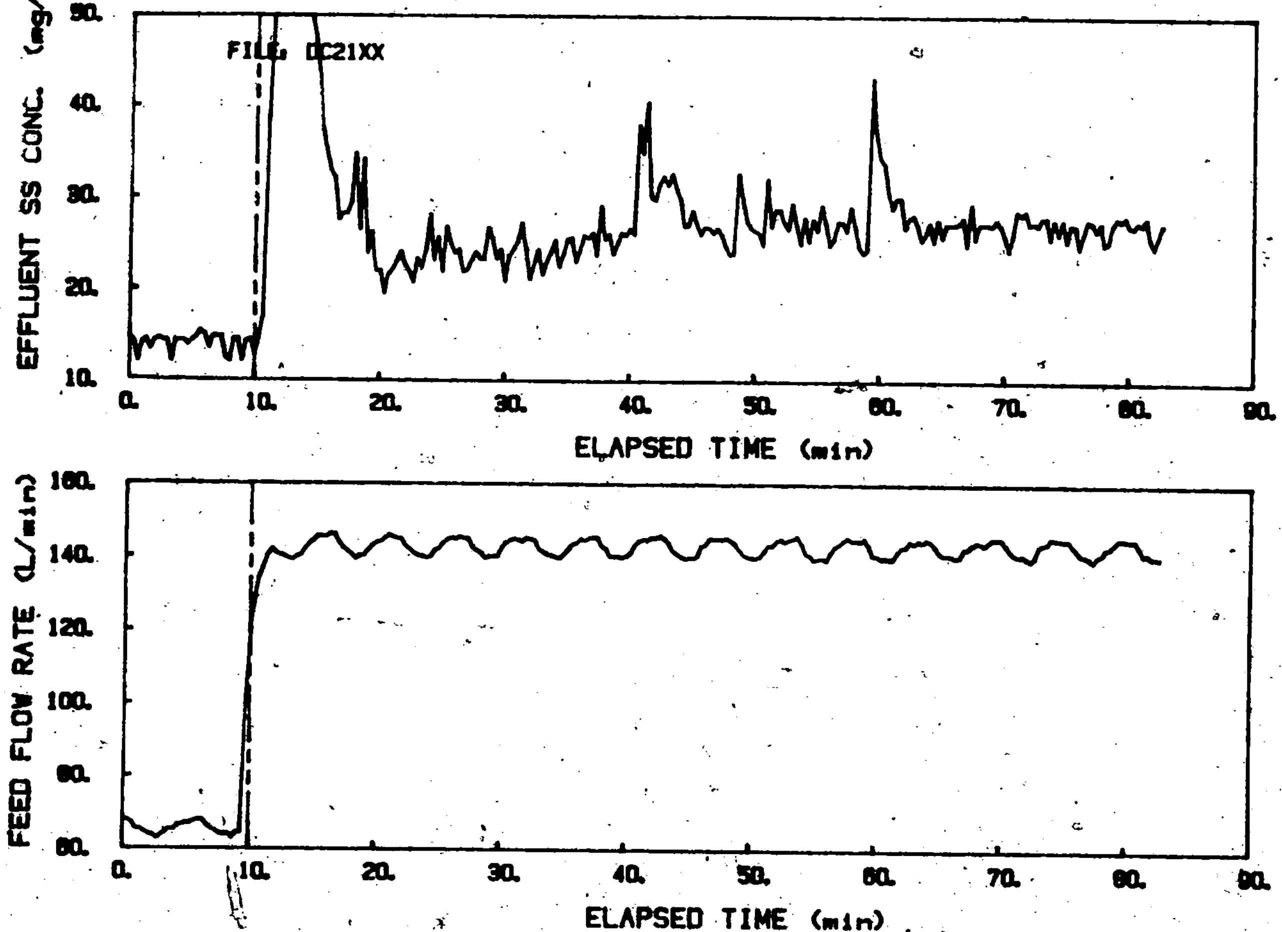
RUN #10. Qa: 100-140 L/min (STEP)



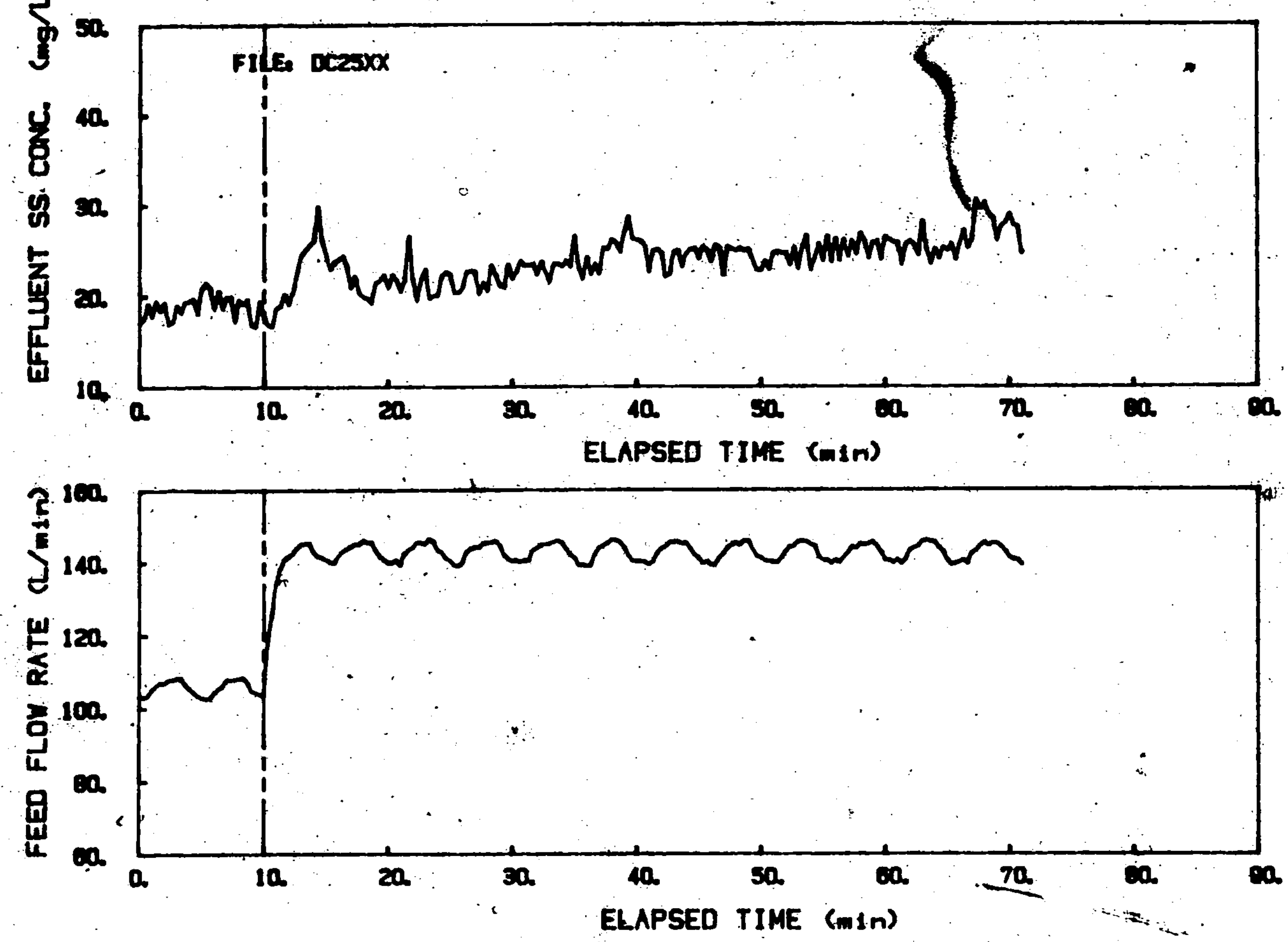
RUN #12, Qa: 100-140 L/min (STEP)



RUN #21. Qa: 100-140 L/min (STEP)



RUN #25. Qa: 100-140 L/min (STEP)



APPENDIX E

FRACTIONAL FACTORIAL DATA

	Length of Run				Levels: Design and Actual								Effluent Solids		
	start		finish		ML	FD	RS	FF	UF	SD	AF	T	Gravi.	Sensor	
Run #	day	time	day	time	(g/L)	(m)	(rph)	(L/min)	(L/min)	(m)	(L/s)	(°C)	(mg/L)	Mean (mg/L)	s (mg/L)
								Block #1:							
CP-1	5/11	11:00	6/11	11:45	2.33 (0)	1.27 (0)	5.0 (0)	120.0 (0)	40.0 (0)	1.94 (+)	58.3 (0)	17.8	18.0	36.5	2.2
R-1	7/11	8:00	9/11	12:00	4.19 (+)	1.72 (+)	8.0 (+)	139.8 (+)	59.8 (+)	1.94 (+)	[illegible].0 (+)	16.5	34.3	37.5	7.4
CP-2	10/11	8:00	11/11	20:00	2.36 (0)	1.27 (0)	5.0 (0)	120.0 (0)	40.0 (0)	1.94 (+)	59.6 (0)	16.6	19.9	25.5	2.2
R-2	14/11	8:00	16/11	9:15	1.69 (−)	0.81 (−)	2.0 (−)	140.0 (+)	60.0 (+)	1.94 (+)	50.1 (−)	16.7	26.5	31.7	4.8
CP-3	17/11	8:00	18/11	9:15	2.75 (0)	1.27 (0)	5.0 (0)	120.0 (0)	40.0 (0)	1.94 (+)	59.0 (0)	17.0	20.9	24.5	1.3
R-3	19/11	8:00	20/11	0:00	3.57 (+)	0.81 (−)	2.0 (−)	100.0 (−)	20.0 (−)	1.94 (+)	69.4 (+)	14.6	19.5	28.8	2.5
R-3*	22/11	8:00	23/11	9:15	3.53 (+)	0.81 (−)	2.0 (−)	100.4 (−)	19.8 (−)	1.94 (+)	67.4 (+)	16.4	21.7	31.9	8.0
CP-4	24/11	8:00	25/11	9:30	2.62 (0)	1.27 (0)	5.0 (0)	119.9 (0)	39.9 (0)	1.94 (+)	60.5 (0)	14.9	17.4	22.1	0.8
R-4	26/11	[illegible]	[illegible]	[illegible]	2.09 (−)	1.72 (+)	8.0 (+)	100.0 (−)	20.0 (−)	1.94 (+)	51.2 (−)	15.3	17.7	27.2	4.3
								Block #2:							
CP-5	28/11	8:00	1/12	9:15	2.26 (0)	1.27 (0)	5.0 (0)	120.0 (0)	40.0 (0)	1.48 (−)	60.9 (0)	14.7	26.8	30.7	5.4
R-5	2/12	8:00	3/12	10:15	1.38 (−)	0.81 (−)	8.0 (+)	139.0 (+)	19.9 (−)	1.48 (−)	52.2** (+)	14.7	55.5	60.1	5.8
CP-6	5/12	8:00	7/12	9:15	2.00 (0)	1.27 (0)	5.0 (0)	120.0 (0)	40.0 (0)	1.48 (−)	61.2 (0)	14.4	32.0	38.0	5.4
R-6	8/12	8:00	10/12	9:15	6.18 (+)	0.81 (−)	8.0 (+)	100.0 (−)	60.0 (+)	1.48 (−)	52.6 (−)	14.1	50.0	53.7	7.2
CP-7	11/12	8:00	14/12	9:00	2.47 (0)	1.27 (0)	5.0 (0)	120.0 (0)	39.9 (0)	1.48 (−)	61.3 (0)	13.8	29.2	37.3	5.1
R-7	14/12	14:00	14/12	20:00	3.28 (+)	1.72 (+)	2.0 (−)	140.2 (+)	19.9 (−)	1.48 (−)	53.5 (−)	13.5	46.4	50.3	9.9
CP-8	19/12	8:00	21/12	9:00	2.18 (0)	1.27 (0)	5.0 (0)	120.0 (0)	40.0 (0)	1.48 (−)	62.7 (0)	12.5	47.0	42.	2.5
R-8	22/12	8:00	24/12	8:45	1.61 (−)	1.7 (+)	2.0 (−)	100.0 (−)	60.0 (+)	1.48 (−)	69.1 (+)	13.5	37.6	39.8	39.7

*Factorial Run #3 was repeated a second time.
**The airflow rate was set incorrectly; it should have been 66.5 L/s.

	Length of Run				Levels: Design and Actual								Effluent Solids		
	start		finish		ML	FD	RS	FF	UF	SD	AF	T	Gravi.	Sensor	
Run #	day	time	day	time	(g/L)	(m)	(rph)	(L/min)	(L/min)	(m)	(L/s)	(°C)	(mg/L)	Mean (mg/L)	s (mg/L)
								Block #1:							
CP-A	20/3	14:00	22/3	9:00	(0) 1.79	(0) 1.27	(0) 5.0	(0) 100.0*	(0) 40.0	(+) 1.94	(0) 58.9	9.6	34.0	37.5	2.2
R-9	23/3	7:00	24/3	8:45	(+) 3.83	(-) 0.81	(+) 8.0	(+) 140.0	(0) 40.0	(+) 1.94	(-) 51.3	10.0	44.5	54.6	3.8
R-10	25/3	7:00	26/3	9:00	(+) 3.77	(+) 1.72	(-) 2.0	(-) 100.0	(+) 60.0	(+) 1.94	(-) 52.0	10.3	26.5	29.5	1.4
								Block #2:							
CP-B	27/3	7:00	29/3	8:45	(0) 2.22	(0) 1.27	(0) 5.0	(0) 120.0	(0) 32.8**	(-) 1.48	(0) 62.1	9.0	49.0	42.0	2.9
R-11	30/3	7:00	31/3	8:00	(+) 3.94	(-) 0.81	(-) 2.0	(+) 139.9	(+) 54.0	(-) 1.48	(+) 68.2	10.9	74.5	76.7	33.9
R-12	1/4	7:00	2/4	8:45	(-) 1.21	(-) 0.81	(-) 2.0	(-) 100.0	(-) 20.0	(-) 1.48	(-) 52.8	9.9	44.5	38.9	3.7
CP-C	3/4	7:00	4/4	11:45	(0) 1.93	(0) 1.27	(0) 5.0	(0) 120.1	(0) 39.9	(-) 1.48	(0) 62.2	9.5	47.0	43.3	6.8
R-13	6/4	7:00	7/4	9:30	(-) 1.19	(+) 1.72	(+) 8.0	(+) 139.9	(+) 60.0	(-) 1.48	(-) 54.2	8.8	48.0	49.9	4.9
R-14	8/4	7:00	9/4	9:15	(+) 3.69	(+) 1.72	(+) 8.0	(-) 100.0	(-) 20.0	(-) 1.48	(+) 69.6	10.3	33.0	32.6	3.9
								Block #3:							
CP-D	12/4	7:00	13/4	9:00	(0) 2.15	(0) 1.27	(0) 5.0	(0) 119.9	(0) 40.0	(+) 1.94	(0) 59.8	10.8	37.0	32.4	4.8
R-15	14/4	7:00	16/4	9:00	(-) 1.44	(+) 1.72	(-) 2.0	(+) 140.0	(-) 20.0	(+) 1.94	(+) 68.0	11.3	38.0	38.5	2.8
CP-E	17/4	7:00	19/4	8:45	(0) 2.26	(0) 1.27	(0) 5.0	(0) 119.9	(0) 40.0	(+) 1.94	(0) 60.6	11.8	44.0	48.1	2.4
R-16	20/4	7:00	21/4	9:00	(-) 1.54	(-) 0.81	(+) 8.0	(-) 100.0	(+) 60.0	(+) 1.94	(+) 69.4	12.4	30.0	27.8	2.6

* incorrect setpoint.
**valve plugged on test settler underflow.

Run #	Length of Run: start day	start time	finish day	finish time	Levels: Design and Actual: ML (g/L)	FD (m)	RS (rph)	FF (L/min)	UF (L/min)	SD (m)	AF (L/s)	T (°C)	Effluent Solids: Grav. (mg/L)	Sensor Mean (mg/L)	Sensor s (mg/L)
Block #1:															
CP-F	15/5 & 16/5	7:00 7:00	15/5 17/5	13:00 9:00	0 2.16	0 1.27	0 5.0	0 122.3	0 37.8	+ 1.94	0 56.2	15.9	23.5	34.7	2.7
R-17	18/5 & 18/5	7:00 18:00	18/5 19/5	10:45 8:45	- 1.86	+ 1.72	+ 8.0	+ 143.0	- 17.1	+ 1.94	- 49.1	16.1	27.0	35.7	2.1
R-18	20/5	7:00	21/5	8:30	+ 4.84	+ 1.72	+ 8.0	- 101.7	+ 58.6	+ 1.94	+ 65.1	16.9	45.0	50.2	4.6
Block #2:															
CP-G	22/5	7:00	24/5	10:15	0 2.72	0 1.27	0 5.0	0 122.3	0 37.9	- 1.48	0 58.3	15.2	62.5	54.6	9.7
R-20	27/5	7:00	28/5	9:00	- 1.41	- 0.81	+ 8.0	- 101.7	- 17.1	- 1.48	+ 65.9	16.4	31.0	35.0	3.0
CP-H	29/5	7:00	30/5	16:15	0 2.29	0 1.27	0 5.0	0 122.3	0 37.8	- 1.48	0 57.4	16.3	53.5	46.9	4.3
R-21	1/6	14:00	2/6	7:30	- 1.48	+ 1.72	- 2.0	+ 143.0	+ 58.6	- 1.48	+ 66.4	17.1	80.0	70.2	17.1
R-22	3/6	7:00	4/6	9:00	+ 4.34	+ 1.72	- 2.0	- 99.6	- 20.2	- 1.48	- 49.8	16.5	23.0	34.6	2.3
Block #3:															
CP-I	6/6	7:00	7/6	9:00	0 2.61	0 1.27	0 5.0	0 120.2	0 41.0	+ 1.94	0 57.3	16.0	31.0	33.1	2.6
R-23	8/6	7:00	9/6	9:00	- 1.54	- 0.81	- 2.0	- 99.6	+ 61.7	+ 1.94	- 49.1	16.6	13.0	16.1	2.2
R-24	10/6	7:00	11/6	9:15	+ 3.90	- 0.81	- 2.0	+ 140.9	0 41.0	+ 1.94	+ 65.1	17.4	31.0	36.2	3.8
Block #4:															
CP-J	12/6	12:00	14/6	9:15	0 2.21	0 1.27	0 5.0	0 120.2	0 41.0	- 1.48	0 57.0	17.3	21.0	31.9	7.7
R-19(R)	15/6	7:00	16/6	9:00	+ 4.72	- 0.81	+ 8.0	+ 140.9	+ 60.6	- 1.48	- 49.2	18.1	81.0	78.7	9.7
R-20(R)	17/6	7:00	18/6	9:00	- 1.87	- 0.81	+ 8.0	- 99.6	- 20.3	- 1.48	+ 64.9	18.1	22.0	20.7	2.4
CP-K	19/6	7:00	21/6	10:00	0 2.91	0 1.27	0 5.0	0 120.2	0 41.0	- 1.48	0 57.4	17.7	27.8	52.6	12.1

APPENDIX F

REGRESSION RESULTS FROM DRAPER AND SMITH (1966) AND PROGRAM "BAKLM"

Results of Step-Wise Regression as Obtained by Draper and Smith (1966).

Input Data Matrix and Correlation Matrix:

	Inputs:				Outputs:
	X_1	X_2	X_3	X_4	X_5
1	7.00000000	26.00000000	6.00000000	60.00000000	78.50000000
2	1.00000000	29.00000000	15.00000000	52.00000000	74.30000000
3	11.00000000	56.00000000	8.00000000	20.00000000	104.30000000
4	11.00000000	31.00000000	8.00000000	47.00000000	87.60000000
5	7.00000000	52.00000000	6.00000000	33.00000000	95.90000000
6	11.00000000	55.00000000	9.00000000	22.00000000	109.20000000
7	3.00000000	71.00000000	17.00000000	6.00000000	102.70000000
8	1.00000000	31.00000000	22.00000000	44.00000000	72.50000000
9	2.00000000	54.00000000	18.00000000	22.00000000	93.10000000
10	21.00000000	47.00000000	4.00000000	26.00000000	115.90000000
11	1.00000000	40.00000000	23.00000000	34.00000000	83.80000000
12	11.00000000	66.00000000	9.00000000	12.00000000	113.30000000
13	10.00000000	68.00000000	8.00000000	12.00000000	109.40000000

Means of Transformed Variables

1	7.46153830	48.15384500	11.76923000	29.99999900	95.42307500

Std. Deviations of Transformed Variables

1	5.88239440	15.56087900	6.40512590	16.73817800	15.04372400

Correlation Matrix

1	.99999991	.22857948	-.82413372	-.24544512	.73071745
2	.22857948	1.00000010	-.13924238	-.97295516	.81625268
3	-.82413372	-.13924238	.99999991	.02953701	-.53467065
4	-.24544512	-.97295516	.02953701	1.00000010	-.82130513
5	.73071745	.81625268	-.53467065	-.82130513	.99999999

Results from Draper and Smith (con't):

Predictions and Residuals:

Obs. No.	Observed Y	Predicted Y	Residual
1	78.5000000	78.4952410	.0047590
2	74.3000000	72.7887950	1.5112050
3	104.3000000	105.9709300	-1.6709300
4	87.6000000	89.3270940	-1.7270940
5	95.9000000	95.6492470	.2507530
6	109.2000000	105.2745500	3.9254500
7	102.7000000	104.1486600	-1.4486600
8	72.5000000	75.6749840	-3.1749840
9	93.1000000	91.7216450	1.3783550
10	115.9000000	115.6184400	.2815600
11	83.8000000	81.8090130	1.9909870
12	113.3000000	112.3270100	.9729900
13	109.4000000	111.6943300	-2.2943300

Basic ANOVA Table:

Source	d.f.	Sums sqs.	Mean sq.	Overall F
Total	12	2715.7635000		
Regression	4	2667.9000000	666.9750000	111.4795200
Residual	8	47.8634980	5.9829372	

Table of Partial F-Values:

Var No.	Mean	Decoded B Coefficient	Limits Upper/ Lower	Standard Error	Partial F-test
4	29.9999990	-.1440588	1.4909970 -1.7791144	.7090441	.0412794
3	11.7692300	.1019111	1.8422494 -1.6384272	.7547901	.0182345
2	48.1538450	.5101700	2.1792063 -1.1588665	.7237799	.4968402
1	7.4615383	1.5511043	3.2685233 -.1663147	.7447811	4.3375858

Output from Program "BAKLM" for Input Data from Draper & Smith (1966).

LU=1 LL=6 N=13 NI=5 NC=-1 NR=0

FILE CONTAINING CALIBRATION DATA: HALD01

********** LINEAR LEAST SQUARES **********

--------- X MATRIX & Y VECTOR ----------

.100D+01	.700D+01	.260D+02	.600D+01	.600D+01	.785D+02
.100D+01	.100D+01	.290D+02	.150D+02	.520D+02	.743D+02
.100D+01	.110D+02	.560D+02	.800D+01	.200D+02	.104D+03
.100D+01	.110D+02	.310D+02	.800D+01	.470D+02	.876D+02
.100D+01	.700D+01	.520D+02	.600D+01	.330D+02	.959D+02
.100D+01	.110D+02	.550D+02	.900D+01	.220D+01	.109D+03
.100D+01	.300D+01	.710D+02	.170D+01	.600D+01	.103D+03
.100D+01	.100D+01	.310D+02	.220D+02	.440D+02	.725D+02
.100D+01	.200D+01	.540D+02	.180D+02	.220D+02	.931D+02
.100D+01	.210D+02	.470D+02	.400D+01	.260D+02	.116D+03
.100D+01	.100D+01	.400D+02	.230D+02	.340D+02	.838D+02
.100D+01	.110D+02	.660D+02	.900D+01	.120D+02	.113D+03
.100D+01	.100D+02	.680D+02	.800D+01	.120D+02	.109D+03

COLUMNS MEANS:

.746D+01	.482D+02	.118D+02	.300D+02	.945D+02

COLUMN STANDARD DEVIATIONS:

.588D+01	.156D+02	.641D+01	.167D+02	.150D+02

CORRELATION MATRIX:

1.00	.23	-.82	-.25	.73
.23	1.00	-.14	-.97	.82
-.82	-.14	1.00	.03	-.53
-.25	-.97	.03	1.00	-.82
.73	.82	-.53	-.82	1.00

Output from Program "BAKLM" (con't):

b-ESTIMATES WITH 95% CONFIDENCE LIMITS:

62.40537	-99.17783 -	223.98857
1.55110	-.16633 -	3.26854
.51017	-1.15888 -	2.17922
.10191	-1.63844 -	1.84226
-.14406	-1.77913 -	1.49101

COEFFICIENT OF MULTIPLE DETERMINATION= 98.2%

PREDICTED VALUES:	RESIDUALS:
78.49524	.00476
72.78880	1.51120
105.97094	-1.67094
89.32710	-1.72710
95.64924	-.25079
105.27456	3.92544
104.14867	-1.44867
75.67499	-3.17449
91.72165	1.37835
115.61845	.28155
81.80902	1.99098
112.32701	.97299
111.69434	-2.29434

********** BASIC ANOVA TABLE **********

SOURCE:	SS:	DF:	MS:	F:
MODEL:	2667.900	4.	666.975	111.480
RESIDUAL:	47.863	8.	5.983	
MEAN (BO):	118372.327	1.		
TOTAL (CORR.):	2715.763	12.		
TOTAL (UNCORR.)	121088[illegible]0	13.		

********** EXTRA SUM OF SQUARES **********

ADDED LAST:	SS:	DF:	MS:	F:
B(1)	25.951	1.	25.951	4.338
B(2)	2.973	1.	2.973	.497
B(3)	.109	1.	.109	.018
B(4)	.247	1.	.247	.041