TUBE SETTLERS
IN
SECONDARY CLARIFICATION

THE APPLICATION OF TUBE SETTLERS
IN
THE SECONDARY CLARIFICATION OF
DOMESTIC WASTEWATER

By

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ABSTRACT

THE APPLICATION OF TUBE SETTLERS IN THE SECONDARY CLARIFICATION OF DOMESTIC WASTEWATER

by

John Bertrand Mendis

The North-End Water Pollution Control Centre (NEWPCC) in Winnipeg, Manitoba, treats an average annual flow of 60 MGD. During peak periods, spring run-off and storms, flows to the plant exceeds 110 MGD. The capacity of the ten secondary clarifiers limit the capacity of the plant to 65 MGD.

This report discusses the results of an attempt to upgrade the capacity of the existing clarifier at Winnipeg with the use of tube settlers. This experimental programme monitored variables which govern both the biological conditioning within the aeration tank (DO, SRT, F/M, etc.) and the hydraulic effects on the clarifier (Q and Q_R). A 5½ inch ID, 5 feet high transparent plexiglass column was used to monitor the zone settling velocities of various mixed liquor concentrations, from which flux plots were drawn and the limiting hydraulic and solids loadings were determined.

A discontinuous or pulse input of Rhodamine WT dye was used to trace the hydraulic behaviour within the clarifier. The overflow was monitored continuously and the underflow or return was monitored by frequent batch sampling.

The results indicate that under good biological conditioning (clarification limiting), the clarifier with tube settlers treated flows up to 11.5 MGD or 2000 GPD/sq.ft. of clarifier surface area (or 167 GPD /sq.ft. of tube settler area) at solids loadings of 50 to 60 lbs/day/sq.ft. This is 80% greater than the flows handled by a similar clarifier with no tubes. Under upset biological conditions (thickening limiting), the tube settlers could not handle flows in excess of 6.5 MGD or 1100 GPD/sq.ft. of clarifier surface area (or 93.2 GPD/sq.ft. of tube settling area) at solids loadings of 45 lbs/d/sq.ft.

Although no significant short circuiting was found in the overflow, the underflow experienced serious short circuiting problems. As the underflow was increased beyond 2.5 MGD or 430 GPD/sq.ft. of clarifier surface area, short circuiting occurred deteriorating the effluent quality. Since the tracer studies were conducted during periods of thickening limiting, it is not known if the short circuiting in the underflow was singularly responsible for

the deterioration of effluent quality, or if it was an interaction between thickening limitation and the short circuiting in the underflow. This is contradictory to the established design criteria of secondary clarifiers which state that an increase in underflow would increase the ability of the clarifier to handle a greater solids loading.

Now that the ground work has been laid in correlating tube settler performance to thickener design, future studies should pursue the possibility of establishing predictive capability of tube settler performance in secondary clarification of domestic wastewater.

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LIST OF TABLES

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS - contd

NOMENCLATURE

ROMAN

Symbol	Description and Units
A	Surface area (sq.ft)
C_f	Clarifier feed solids conc. (mg/l)
C_i	Suspended solids concentration (mg/l)
C_l	Solids concentration corresponding to limiting flux (mg/l)
C_m	Solids concentration corresponding to limiting flux @ the upper configurate zone (mg/l)
C_u	Clarifier underflow concentration (mg/l)
d	Dispersion index as defined by Levenspeil (49)
d'	Diameter of tube settler (ft)
D	Axial dispersion Co-efficient (sq.ft/hr)
G_a	Applied solids flux (lbs/hr/sq.ft)
G_g	Solids flux due to gravity subsidence (lbs/hr/sq.ft)
G_t	Total solids flux (lbs/hr/sq.ft)
G_u	Solids flux due to bulk downward movement (lbs/hr/sq.ft)

NOMENCLATURE - contd

ROMAN

Symbol	Description and Units
H	Height of slurry @ time t (ft)
H	Find height of compacted slurry (ft)
I_s	Index of short circuiting defined by Murphy(53)
L	Length of tube settler (ft)
L'	Length of tracer (ft)
MLSS	Mixed Liquor Suspended Solids (mg/l)
n	Emperical constant
Q_f	Volumetric clarifier feed rate (MGD or cu.ft/hr)
Q_R	Volumetric recycle rate (MGD or Cu ft/hr)
Q_u	Volumetric underflow rate(MGD or Cu ft/hr)
Q_w	Volumetric wasting rate ($Q_u = Q_R+Q_w$) (MGD or Cu.ft/hr)
R	Hydraulic radius (ft)
r	Recycle ratio (Q_R/Q)
S	Constant as defined by Yao (75)
SS	Suspended solids
SVI	Sludge Volume Index ml/gm
T	Theoretical detention time (mins)
t_{10}	Time for 10% of the injected tracer to pass at the effluent end (mins)
t_{90}	Time for 90% of the injected tracer to pass at the effluent end (mins)

NOMENCLATURE - contd

ROMAN

Symbol	Description and Units
tg	Time to reach centroid of the curve or mean detention time (mins)
t_i	Time interval for the initial appearance of tracer in the effluent (mins)
U	Bulk downward velocity (ft/hr)
U'	Mean flow velocity (ft/hr)
V_i	Hindered solids settling velocity (ft/hr)
V_l	Settling velocity corresponding to the limiting flux (ft/hr)
Vo	Average flow velocity of fluid through the tube settler (ft/hr)
V_s	Fall velocity of discrete particle (ft/hr)
V_{sc}	Critical fall velocity of discrete particle (ft/hr)
x	Length along x axis (ft)
X	Dimensionless ratio $x/_d$
y	Length along y axis (ft)
Y	Dimensionless ratio $y/_d$
ZSV	Zone Settling Velocity (ft/hr)

NOMENCLATURE - contd

GREEK

Symbol	Description and Units
α	Empirical constant
σ	Variance tracer dispersion curve
θ	Angle of Inclination of tube settlers
θ_c	Solids retention time or mean cell retention time (days)
ψ	Solids loading factor as defined by Jennings & Grady (45)

LIST OF APPENDICES

TABLE OF CONTENTS

TABLE OF CONTENTS - cont'd

CHAPTER I

1.0 INTRODUCTION

Although previous workers had aerated wastewater flows for odor control or to provide some treatment, Arden and Lockett (3) are generally credited with discovery of the phenomenon that led to the development of Activated Sludge (62). Aerobic biological treatment has since proved to be a most versatile form of organic removal from wastewater prior to discharge. This process inherently relies on two interdependent characteristics for the production of an acceptable effluent. The first is the assimilation of suspended and colloidal organic material by the active mass of microorganisms to a final end product of carbon dioxide, water and inert material.

$$\text{Organics} + O_2 \xrightarrow[\text{Bacteria}]{\text{Nutrients}} CO_2 + \text{Bacteria} + H_2O$$

This initial phase is commonly referred to as Substrate Utilization and deals primarily with the synthesis of activated microorganisms. There exists at this time a wealth of information concerning this initial carbon utilizing phase.

The second phase and ultimately the most significant in the development of a high quality effluent is the separation of the biological solids for recycle back to the aeration tank. Therefore, like any settling basin, the final settling tank in the activated sludge process has two functions: (a) The production of an effluent which is relatively free of settleable solids; and (b) The production of an underflow which contains, in high concentration, the solids which have been settled in the tank. Both functions, clarification and thickening, must be considered (19) in the design if the basin is to satisfactorily accomplish both of its tasks. The importance of the clarification function of final settling tanks is illustrated in Fig. 1. As shown by the figure, the total B.O.D. of the effluent from an activated sludge treatment plant is dissolved B.O.D. plus the B.O.D. of the suspended solids. While the B.O.D. equivalent of activated sludge solids depends on the treatment plant loading (50) and other factors, each mg/l of solids lost over the weir of the final settling tank commonly increases the effluent B.O.D. by about 0.6 mg/l (19). Thus, if in Fig. 1 the dissolved effluent B.O.D. is 10 mg/l, then with an effluent suspended solids concentration about 20 mg/l, over half of the total B.O.D. leaving an activated sludge plant is in the form of solids which were not removed in the sedimentation process.

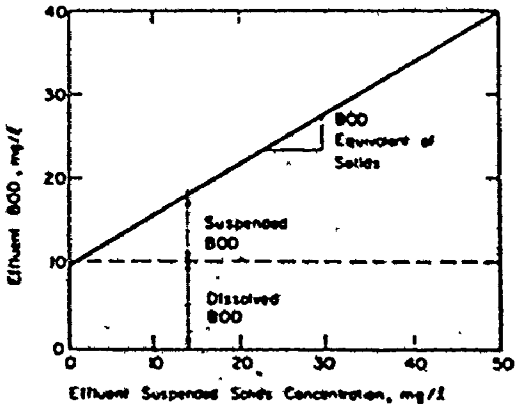

Fig.1-Effect of Clarification on Quality of Effluent from Activated Sludge Plant(19)

The consequence of inadequate thickening is more complex.

For instance it can lead to,

(1) deterioration of effluent by loss of suspended solids caused by an inadequate capacity for conveying solids to the bottom; and

(2) a disruption of the treatment process by altering the conditions in the biological phase of the process because of insufficient solids in the return.

Due to the latter, the concentration of the mixed liquor suspended solids (MLSS) could be reduced. This in turn would increase the organic load intensity. The increase in organic load may adversely alter the flocculating and settling characteristics of the sludge (28) (44) and make it even more difficult to achieve the desired underflow concentration. Hence the problem becomes compounded. In this case, what may appear to be a problem related to the biological aspects of the activated sludge process actually stems from inadequate design in the solids separation phase of the process.

Because clarifiers and indeed treatment plants are designed on the basis of average daily flow, they are subject to hydraulic overloading during peak flow periods. This is particularly characteristic of older treatment plants. Although hydraulic overloading affects all phases of treatment, its principal effect is to strain the solids capture capability of the final settler in secondary biological plants causing excessive loss of solids to the effluent.

To prevent this, many plants by-pass a significant portion of their peak flow either untreated or after primary treatment alone. This is a highly undesirable practice and poses a critical problem to wastewater treatment operators and designers.

Faced with this problem, climbing construction costs and increasingly more stringent effluent quality standards, designers and municipal engineers are being forced to examine more efficient methods for removing solids from wastewaters. High-rate gravity clarification using tube settlers is one process that has been receiving increased attention.

The North End Water Pollution Control Centre* in Winnipeg treats annually an average flow of 72 USMGD**. During peak periods, spring run-off and storms, flows to the plant could exceed 100 MGD. During periods of normal operations (i.e. when SVI values are 100 or less) the hydraulic capacity of the clarifiers has been limited to 60 to 70 MGD.

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* Please refer to Appendix B.1 for description of the North End Plant.

** Unless otherwise stated the values used herein are in Imperial Gallons.

At higher flow rates, short circuiting (as shown in Fig.2) results in a deterioration of the effluent quality(33). In order to increase the clarifier capacity, the City of Winnipeg conducted a preliminary test using a portable tube settler module installed at various locations in one of the existing clarifiers. The effluent quality from the test module was far superior to the basin effluent even at higher hydraulic loadings (13) (see Fig.3).

Since the pattern of short circuiting was such that solids entered the existing launder from the outer weir (nearer to the tank walls), an additional launder was installed in an attempt to equalize the upflow across the surface of the tank. The first phase of testing was to measure improvement in effluent quality caused by the additional launders. Girling (33) reported only a slight improvement in effluent quality. The major indication was however that when short circuiting occurred, it occurred equally over both inner and outer launders. This tends to indicate that an equalization of upflow was in fact achieved.

Settling tubes were installed along the periphery of the tank for the next phase of testing (see Fig.4). Even with a higher solids loading compared to the control basin (1.5:1), the test clarifier with partial tube coverage produced an effluent quality comparable to the control clarifier. Overflow rates to the tube clarifier during this period were 10 to 25 per cent greater than the control

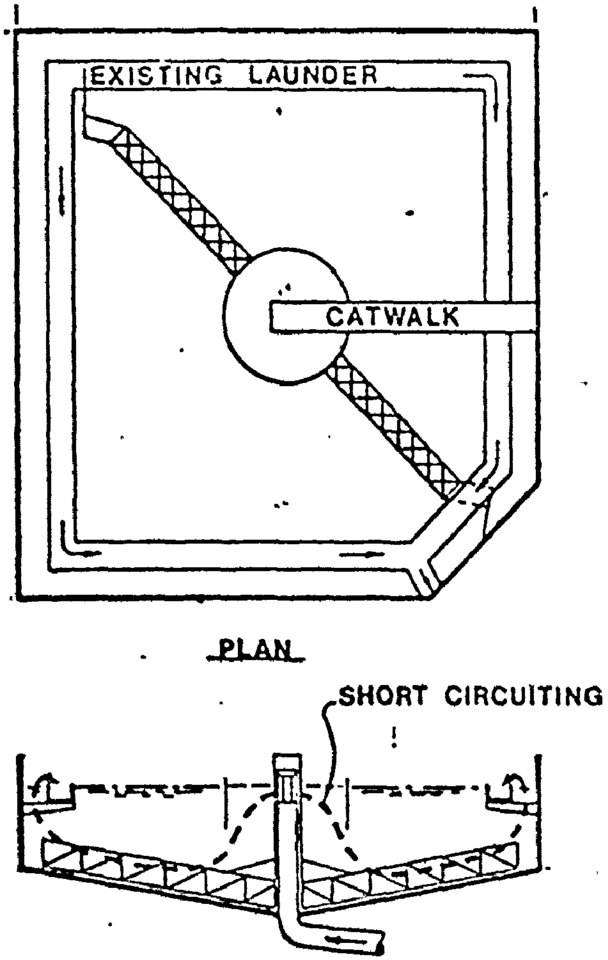

Fig.2- Pattern of Short Circuiting in the Existing Conventional Clarifiers @ NEWPCC, Winnipeg, Manitoba(33).

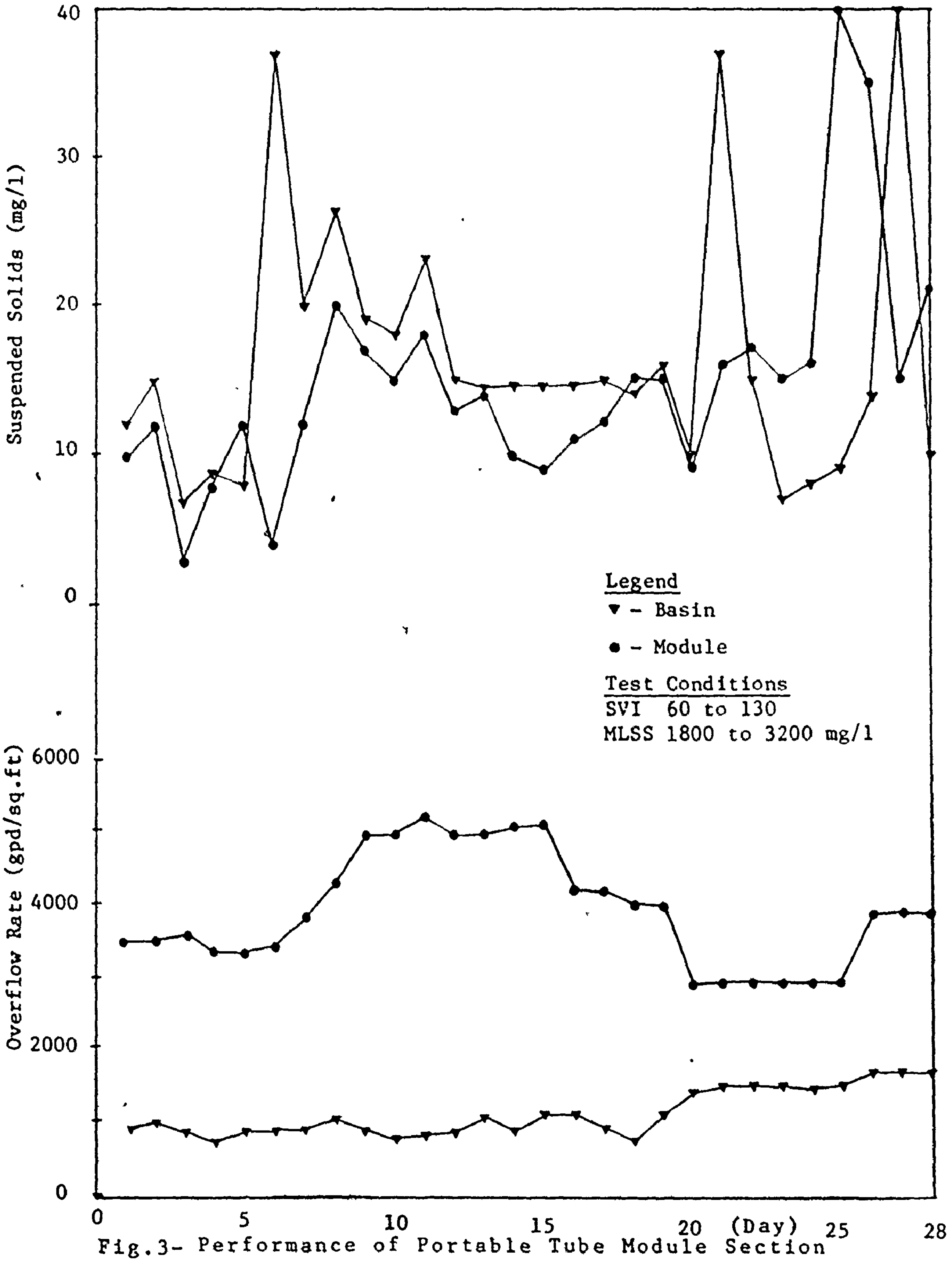

Fig.3- Performance of Portable Tube Module Section Vs Remainder of Clarifier at the NEWPCC, Winnipeg, Manitoba (33).

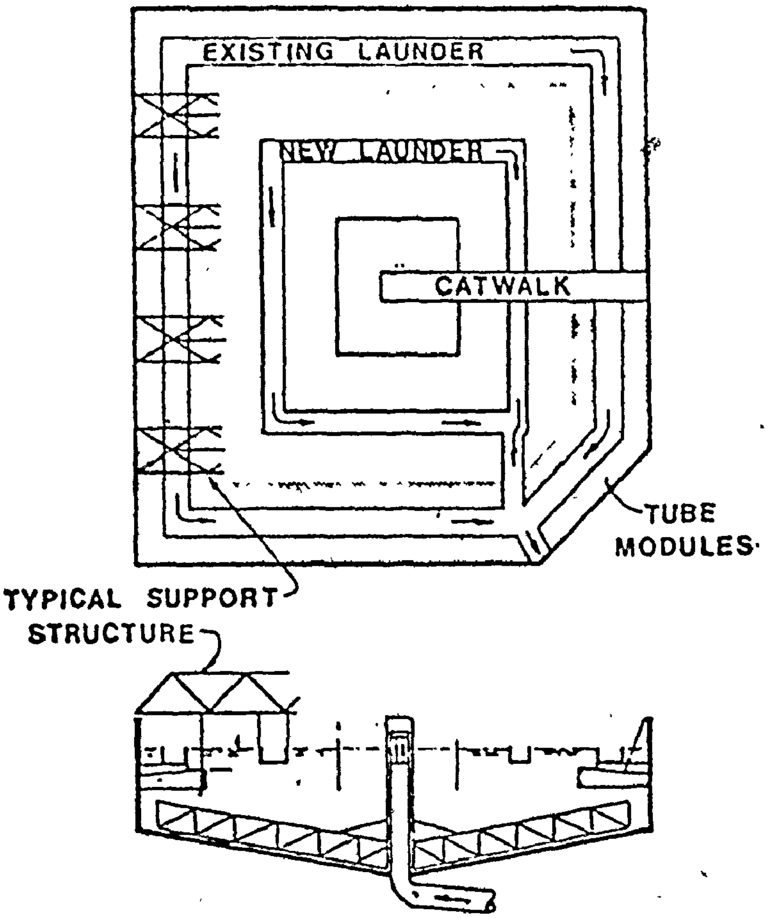

Fig.4-Test Clarifier with Partial Tube Installation (33).

basin. At 50 per cent greater overflow rates the effluent quality was unacceptable (33).

The third phase in the test programme was to evaluate the performance of the clarifier with full tube coverage. Although the installation was completed by August 1975, no meaningful data became available until December 1975. This data (See Table I) confirms an earlier observation that solids in the new launder were greater than that found in the existing launder*.

Previous studies by Culp et al (13) (14) (15), Hennessey (41), Oppelt (53), Slechta and Conley (66) and White (72) report of many instances where the use of tube settlers have either markedly improved effluent quality and/or provided an increase in hydraulic capacity. However, there has been no attempt to relate these performances to existing thickening theory. The present study is a continuation of the third phase, over a four month period from March to June, 1976.

This programme was designed, (a) to ascertain the effects of the more important process variables such as, overflow rate, solids loading, zone settling velocity and SVI on tube settler performance, (b) to establish correlations if any, between these variables and effluent

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* Due to which it was decided to eliminate the use of the new launders for the present study.

TABLE I

OPERATIONAL DATA OF TUBE-CLARIFIER AT THE NEWPCC. (December 1975)

Period (Week)	Aeration Basin		Final Clarifier				Effluent SS(1) / Effluent SS(2)
	MLSS mg/l	SVI	O.R. gpd/sq.ft	S.L. lbs/day/sq.ft	SS(1) mg/l	SS(2) mg/l	
1	1832	106	1416	32.4	21.7	12.6	1.72
2	2188	104	1495	40.8	27.4	21.4	1.28
3	1867	100	1420	33.1	13.4	13.0	1.03
4	1987	114	1153	28.6	-	27.4	-

<u>Note:</u> O.R. -- Overflow Rate; . S.L. -- Solids Loading

SS(1) -- Effluent Suspended Solids of samples withdrawn from inner launder only.

SS(2) -- Effluent Suspended Solids of samples withdrawn from common outfall.

suspended solids, (c) study the effects of tank hydraulics on solids removal, and (d) to develop application guidelines for the use of tube settlers in secondary clarification.

Chapter II deals with historical to present day developments in secondary clarifier design. It also describes shallow depth sedimentation and plant experiences in the use of tube settlers. The third chapter is devoted to describing the experimental programme. Chapter IV contains the results and discussion with a presentation of correlations. The final chapters contain conclusions and application guidelines for the utilization of high-rate tube settlers in activated sludge secondary clarification.

CHAPTER II

2.0 THEORETICAL INVESTIGATION

2.1 Thickening Theory: Bacon and Dalton (4) have called sludge thickening "the largest unsolved research and development problem." The fact that this statement was made in 1966 is indicative of the limited information available for design and operation of secondary clarifiers. Recently, however, Dick (20) (21) (22), Yoshioko (77), Mancini (50), Shannon and Tory (65), Katz(46) and others have established reasonable design criteria and ground work for further research and development.

To the best of the writer's knowledge, the majority of modern theories are based on two classical papers published nearly forty years apart. First, Coe and Clevenger (17) who in 1917 identified several regimes or zones formed during sedimentation of a solid - liquid mixture of pulp and water. Four distinct zones would form, namely (i) feed pulp zone, (ii) flocculated layer of constant consistency, (iii) transition zone, and (iv) compression zone.

They designated zones(ii)and(iii)as free settling zones wherein the slime pulp would fall freely through the liquid without pressing on the flocs below. Coe and Clevenger (17) tacitly assumed that, under the operating conditions, the settling rate would be a function solely of the solids concentration. Therefore, the solids-handling capacity of any layer is a function only of its concentration. In a continuous thickener, the solids must be able to subside through any layer of concentration between the feed and underflow concentrations, at least as rapidly as they are fed to the unit. Otherwise a layer or zone of whatever concentration will limit the solids handling capacity and act as a barrier.

If sufficient area is not presented to handle the solids, such a barrier would build up and all solids in excess of the amount which could subside through this zone would eventually have to overflow the thickener. Therefore, a thickener must have at least enough area to allow the solids to subside through whichever concentration layer would have the least solids handling capacity.

The method Coe and Clevenger (17) used for determining the thickener area needed, was to make a series of batch settling tests on the pulp at varous concentrations (between inlet and required underflow concentrations) and determine the area to handle unit flow of solids for each concentration. The maximum area thus determined was used as a basis of thickener design.

In order to determine the thickener area they presented the following equation :

$$C \doteq \frac{62.35 \quad R}{F-.D} \qquad \text{where,}$$

F = Ratio of fluid to solids in the pulp tested

R = Rate of settling in feet per hour of a free settling pulp of consistency F

D = Ratio of fluid to solids in the underflow

C = Capacity in pounds of solids per square foot per hour which may be discharged with an underflow concentration of D from a layer of pulp of consistency F settling at a rate R.

This may be re-written to suit present day terminology thus,

$$C = \frac{V}{\frac{1}{Ci} - \frac{1}{Cu}} \quad \text{where,} \qquad \ldots (1)$$

C = Capacity of a suspension at concentration Ci, with a velocity V, to transmit solids if the suspension is being thickened to a concentration Cu.

Coe and Clevenger (17) must have been aware of upward propagation of layers of lower solids handling capacity, but did not develop it to explain the ever-decreasing settling rate they observed in the transition between free settling and the compression conditions in a batch test.

Kynch (47) in 1952, as if to explain what Coe and Clevenger failed to describe, presented a rigorous mathematical description of gravity subsidence of concentrated suspensions. Kynch(47) assumed

(a) that all particles in the suspension were similar in size and shape; and

(b) Velocity (V) of any particles is a function only of its local concentration (C).

The nature of Kynch's basic conclusions may be illustrated by using it to describe the shape of the solid-liquid interface subsidence curve during batch sedimentation. The upper line in Fig. 5 represents such a curve (with the brief initial lag period caused by flocculation and residual turbulence). As illustrated by the figure a suspension of initial concentration a, settles at a uniform concentration (V_a) as indicated by the slope of line AB.

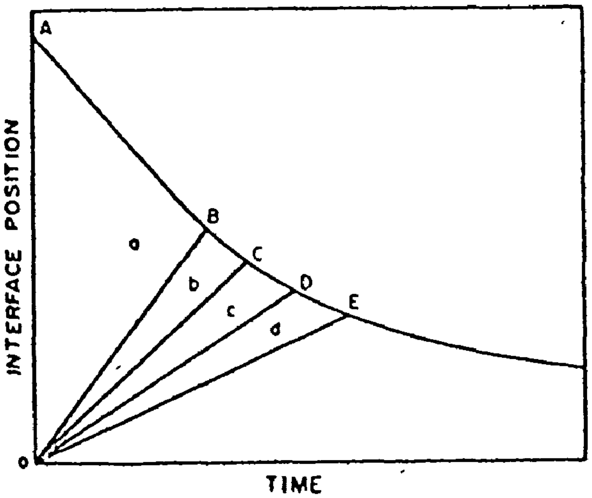


Fig.5- Kynch Interpretation of a Settling Curve.

At the same time, a layer of higher concentration b, is propagated up from the bottom of the vessel at a constant velocity equal to the slope of the line OB. At a point B, the suspension at its initial concentration is expired and the subsidence velocity of the interface is reduced to that of the settling rate (V_b) of the suspension at concentration b (slope of the line BC). At point C, the interface subsides at a rate equal to line CD which is the characteristic settling rate V_c, for the suspension at concentration C, and so forth. Only those concentrations which have a lesser capacity for transmitting solids than that of the overlying layer will appear during batch sedimentation.

Talmage and Fitch (68) showed that applying the Kynch mathematics to thickening eliminates the necessity for multiple batch settling tests for determining the limiting solids-handling capacity as advocated by Coe and Clevenger (17). They also developed a geometric construction method for using a single-batch settling curve to establish the area required for an arbitarily selected, rate limiting layer. Because of their simplicity, these techniques have received considerable attention. However, several authors including Fitch, have subsequently reported better results with multiple batch settling tests. According to Dick (22) several recent authors have used solid flux versus concentration in the analysis of the sedimentation of suspensions.

The solids flux curve approach provides a method by which one can determine the area required to transport solids to the bottom of the tank. Biological solids are transported to the bottom by two mechanisms namely, subsidence due to gravity and bulk downward movement due to sludge withdrawal from the bottom of the tank (19) (21) (22) and (23).

The total solids flux is given by, (See Fig.6)

$$G_t = G_g + G_u \qquad ...(2)$$

$$G_g = C_i V_i \text{ and } G_u = C_i U \qquad ...(3)$$

Where G_g is the solids flux due to gravity subsidence and is the product of suspended solids concentration (C_i) and its corresponding settling velocity (V_i). G_u on the other hand is the flux due to downward bulk velocity (U). If Q_u is the volumetric underflow rate and A is the area of the tank,

$$U = \frac{Q_u}{A} \qquad ...(4)$$

Combining equations (2) and (3) yields,

$$G_t = C_i V_i + C_i U \qquad ...(5)$$

V_i is determined by settling column tests, which results in a plot as shown in Fig. 7.

The possible solids transport due to gravity subsidence at each concentration of activated sludge is a product of the settling velocity (V_i) and the corresponding concentration (C_i) (see Fig. 8).

Fig. 8 is also called a batch flux curve. It is apparent from the plot that dilute sludges settle rapidly, but as concentration increases, settling velocity rapidly decreases and approaches zero at extremely high concentrations.

The second term in equation (5) is determined by

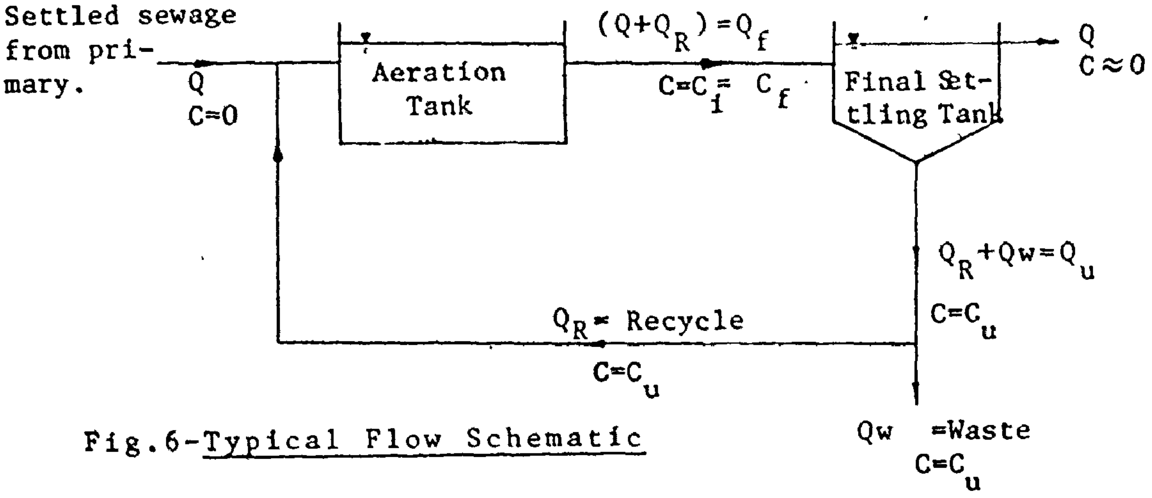


Fig.6-Typical Flow Schematic

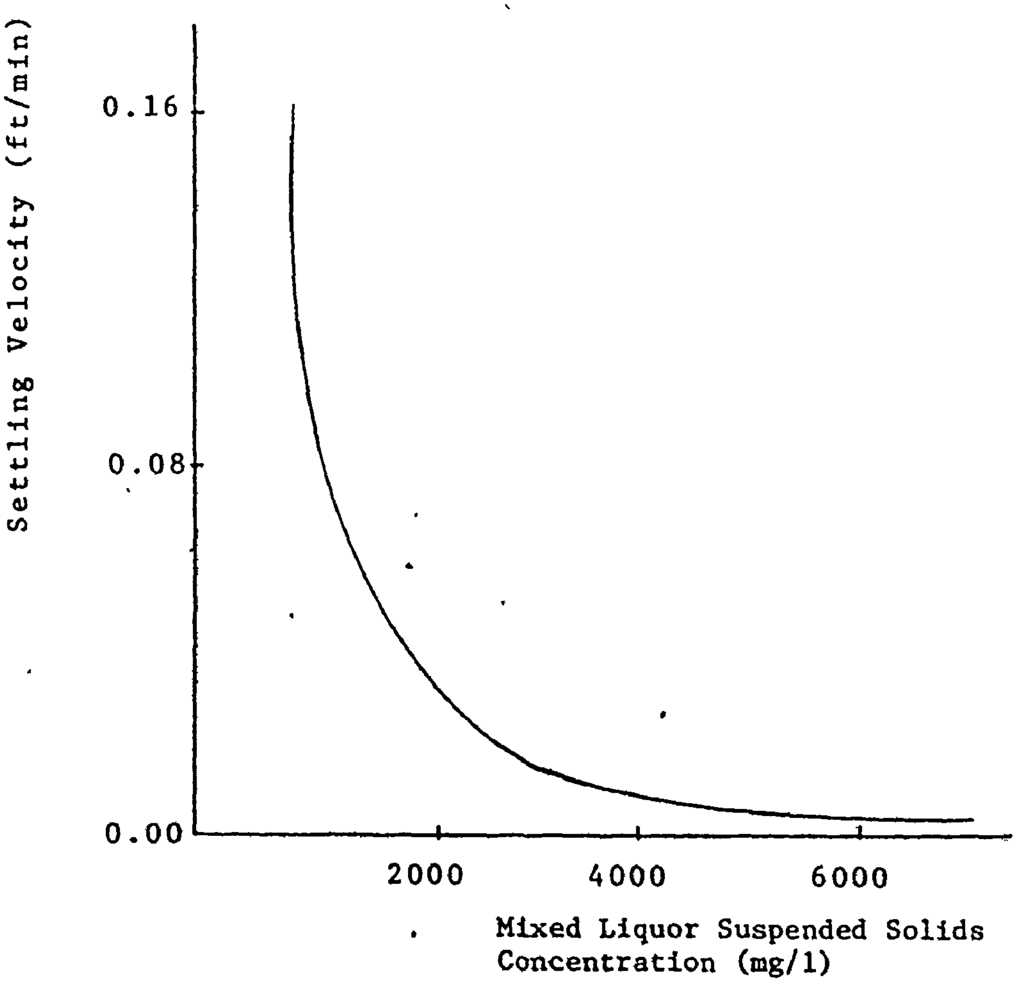


Fig.7-Typical Activated Sludge Solids Settling Curve.

using the following arguments; (Please refer to Fig.6)

$$U = \frac{Q_u}{A} = \frac{rQ}{A} , \quad \text{where } r = \frac{Qu}{Q}$$

Since, $Q_u \, C_u = Q_f \, C_f$ (Total solids capture)

Therefore $$U = \frac{(Q_R+Q)\,C_f}{C_u \; A} \qquad \ldots(6)$$

Plot C_iU versus C_i to cover all values of concentration between the influent (C_f) and underflow C_u) (see Fig.9).

The total possible flux for any concentration then will be the sum of Figures (8) and(8A) or equation (5) (see Fig.9).

Assume that a sludge is to be thickened from C_f to C_u Of the intervening concentrations, C_l, has the least total capacity for transmitting solids to the bottom.

In a design, therefore, sufficient area must be provided so that the applied solids flux G_a does not exceed the limiting total flux G_l. Assuming complete removal of solids,

$$G_a = \frac{C_f Q_f}{A} \leqq G_l$$

Therefore the area to be provided is,

$$A = \frac{C_f\,(Q+Q_R)}{G_l} = \frac{C_f\,(1+r)\;Q}{G_l} \qquad \ldots(7)$$

The values of Q,r and G_l are variable and therefore could be controlled for optimum operations.

Re-writing (5) for the limiting concentration,

$$G_l = C_l\,V_l + C_l\,\frac{(rQ)}{A} \qquad \ldots(8)$$

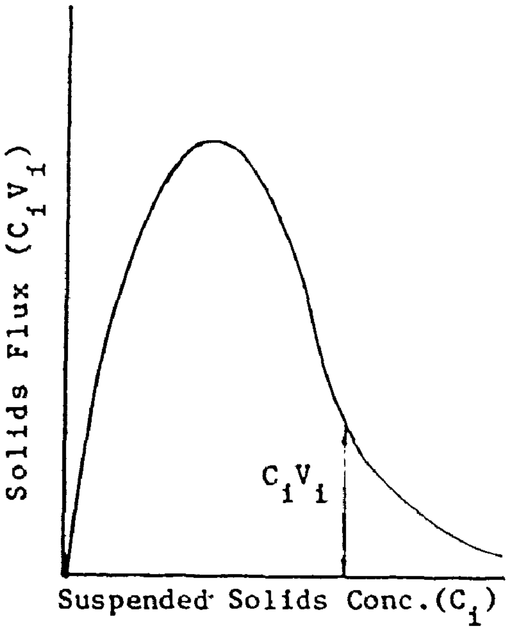


Fig.8-Typ.Solids Transport Curve Due to Gravity Subsidence (Batch Flux Curve)

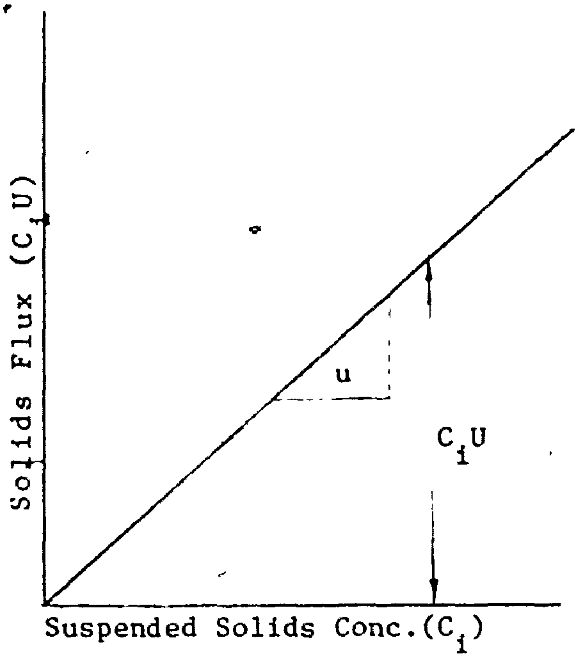


Fig.8A-Typ.Solids Transport Curve due to Bulk Downward movement caused by sludge withdrawal

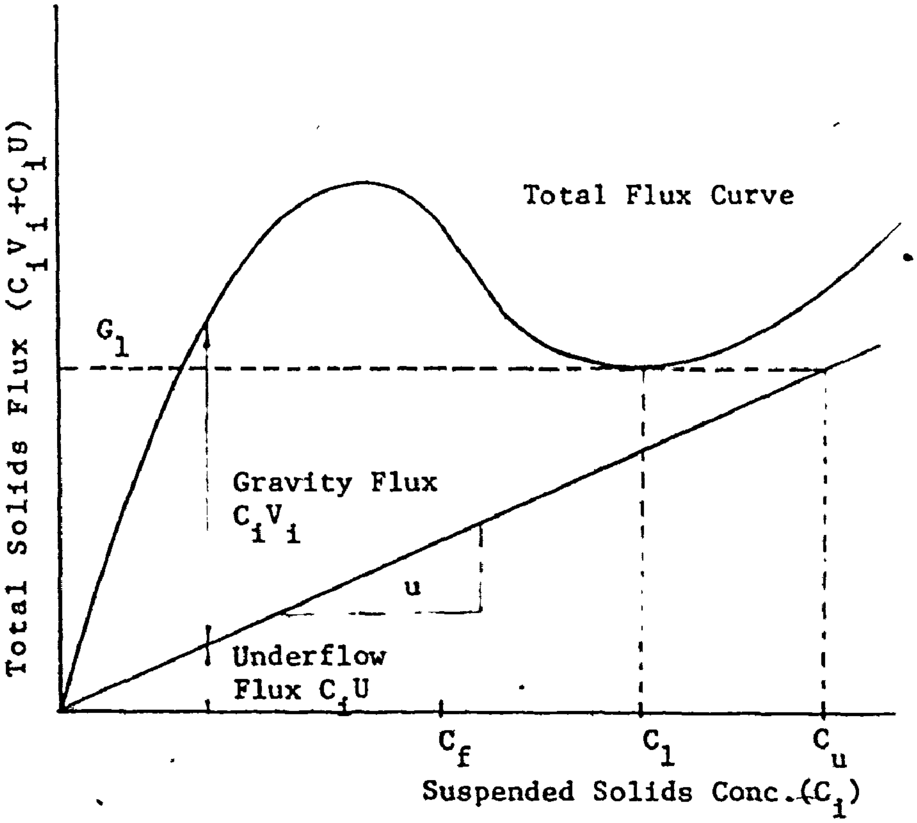


Fig.9 - Typical Total Flux Plot in a Continuous Thickener.

At higher values of r, a larger solids handling can be achieved, but only by sacrificing the degree of sludge thickening accomplished,

$$C_u = \frac{(Q_R+Q)\,C_f}{Q_R} = (\frac{1}{r} + 1) \quad C_f \qquad \ldots(9)$$

It is seen therefore that C_u is maximised by minimising r. In the case of operational control of an existing final settling tank where A is fixed, the operator exercises control over the tank by varying the wasting rate and recycle to give the best values of G_1 and C_u.

If solids are applied at a rate higher than G_a, they will not be able to reach the tank bottom because of the bottleneck created by the limiting solids handling capacity of the sludge at concentration C_1. In these circumstances, a quantity of solids $(G_a - G_1)$ would propagate upwards and pass over the weir. The preceding design approach was advocated by Hassett (39) and used by Dick (19) to demonstrate the role of final settling tanks in the activated sludge process.

An alternative final settling tank design has been published by Yoshioko et al (78) They described when a tangent to the batch flux plot is drawn with the abscissa intercepted at the desired underflow concentration, the line intercepts the ordinate at the corresponding limiting flux, G_1 (see Fig.10).

The tangent or "operating line" was used by Yoshioko

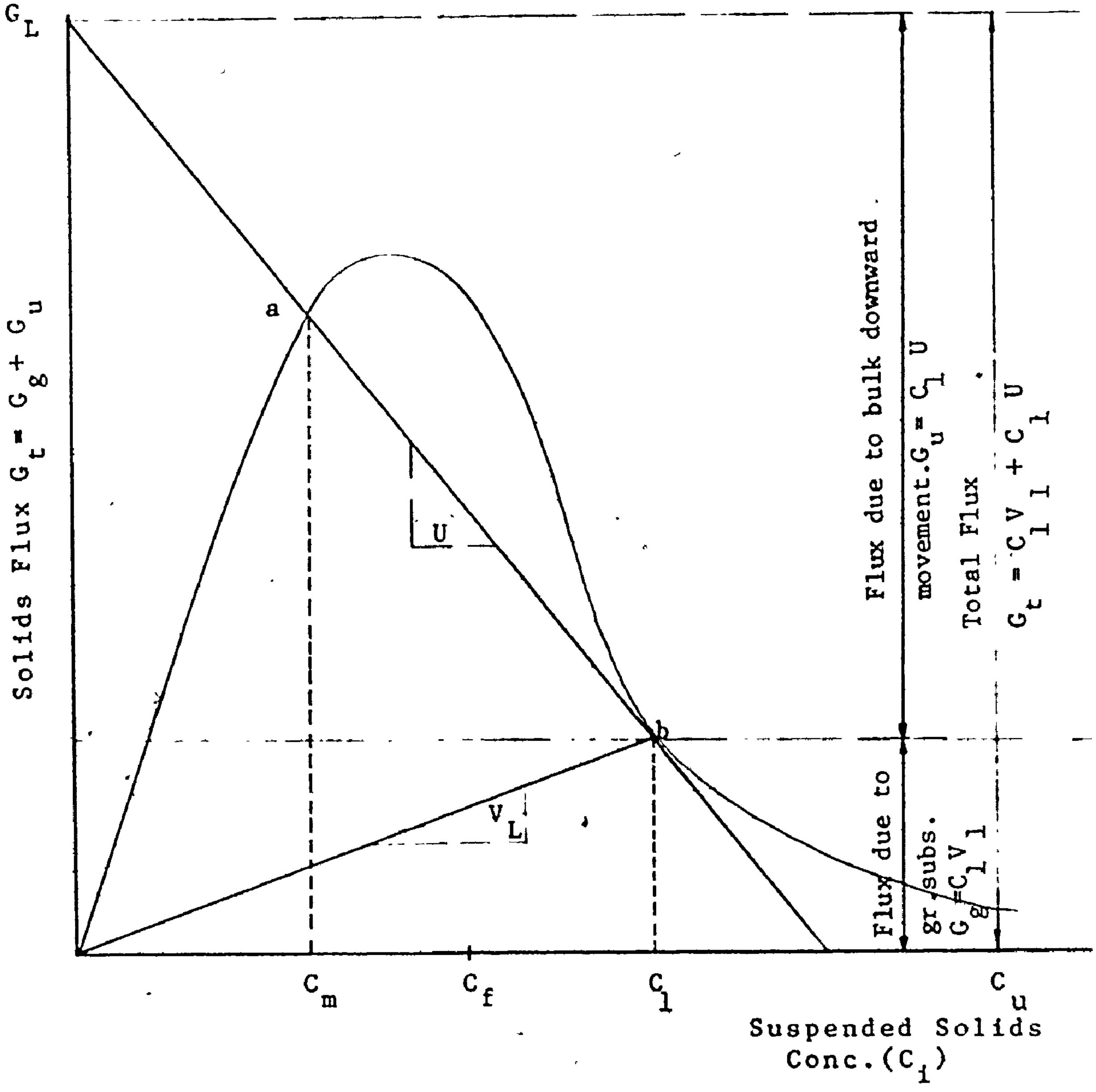


Fig.10 Graphical Determination of Settling Tank Performance.

to predict performance and has several important characteristics.

1. The slope of the operating line has the same absolute value as the underflow flux line, but is of opposite sign. Thus, the downward velocity of the fluid is fixed once the operating line is fixed.
2. The slope of the operating line is equal to the limiting flux divided by underflow concentration. Consequently, the underflow is also given by the intersection of the underflow flux line with a horizontal line drawn through the limiting flux.
3. Fig.10 shows that limiting flux can occur at two concentrations, a and b. These were termed as the upper and lower conjugate points by Hassett(39). Dick (19) clearly indicated that it was the lower conjugate point that is of importance in activated sludge final settler design. For more complicated flux curves, Shannon and Tory (65) have shown that more than one limiting concentration may occur in the lower conjugate zone.
4. The limiting flux for the lower conjugate zone occurs at the concentration corresponding to the point of tangency of the operating line with the batch flux curve,(b).
5. The limiting flux at the upper conjugate zone occurs at the concentration (C_m) given by the intersection of the operating line with the batch flux

curve (a). If the concentration of solids entering (C_f) the settler is greater than that concentration (C_m), the upper conjugate zone will not form unless dilution of the solids occur.

6. The concentration (C_m) represents the minimum inlet solids concentration for which the limiting flux will be determined by the intersection of the operating line with the ordinate.

However, should the influent solids concentration be less than (C_m), the area requirements of the settler has little effect as the flux curve is almost linear in that region. Solids will ~~be~~ handled by the clarifier up to a concentration (C_m) without requiring any change in the underflow rate.

7. Once the operating line is fixed, the optimum use of the settler area will be used only if the input solids concentration (C_f) is equal to or greater than (C_m). Should the operating line be below the tangential line, the underflow concentration will be lower in magnitude. Should the operating line be above the tangent line, it must cut the batch flux curve and this would mean the limiting flux will be exceeded and a system failure will occur.

Previous efforts, as indicated by published design standards (63) had isolated the settler from the aeration tank in design consideration. Since the sizes of both aeration and settler tanks are related to the MLSS concentration, the opportunity exists for optimising them to arrive at the most economic design, Dick (23), Jennings and Grady (45).

Mathematical analyses have been developed to describe the preceding graphical determinations of the final settling tank area.

The graphical analysis has been modified (23) to relate settling velocity to concentration. The model which seems to be most acceptable being,

$$V_i = \alpha C_i^{-n} \qquad \ldots(10)$$

Where α and n are emperical constants which depend on the physical characteristics of the sludge. Values of α and n, from literature are shown in Table I In cases where equation (10) had not been used they were replotted. Large values of α represent "good" settling sludges. Large values of n represent sludges with settling properties very sensitive to changes in MLSS concentration. Dick and Young (23) performed a series of laboratory determinations of α and n, and used them to predict the performance of the plants from which the samples were originally treated. The deviation of the predictions from the actual performance was ± 20 per cent. Because of (1) inevitable experimental error, (2) assumptions made in derivation of equations, and (3) loading conditions observed under one loading condition were applied to all other loadings, they considered their findings to be excellent.

Substituting equation (1) into (5)

$$G_t = \alpha C_i^{(1-n)} + \frac{rQ}{A} C_i \qquad \ldots(11)$$

The value of the limiting flux may be formed by taking the first derivative of equation (1) and equating it to zero.

i.e. $$\frac{\partial G_t}{\partial C_i} = \alpha(1-n)\ C_i^{-n} + \frac{rQ}{A} = 0 \qquad \ldots(12)$$

from which $$C_1 = \alpha \frac{(n-1)}{r} \ \frac{A}{Q} \ ^{1/n} \qquad \ldots(13)$$

This corresponds to the total flux curve (i.e. G Vs C) when the second derivative is positive.

Differentiating equation (12)

$$\frac{\partial^2 G_t}{\partial C_i^2} = \frac{\alpha n \ (n-1)}{C_i^{(1-n)}} \qquad \ldots(14)$$

As long as α is positive and n is greater than 1, equation (14) will be positive and hence, C_1 in equation (13) is the limiting concentration.

Note, however, that equation (14) cannot have a negative value and hence will not recognise the existence of the maximum value shown in the total flux curve (Fig. 9). Equation (10) does not adequately describe settling at low velocities. Fortunately the magnitude of C_1 is ordinarily within the range of concentrations for which equation (10) will be satisfied. Hence the deficiency in the model is not of serious concern.

Substituting equation (13) into equation (11),

TABLE II

TYPICAL VALUES OF α & n FROM LITERATURE

a, in feet per minute	n	System type	Process Loading Factor		Refs.
			Value	Basis	
0.43×10^{-8}	2.62	conventional activated sludge	0.5	Pounds BOD/ Pounds MLVSS	(23)
2.62×10^{-6}	1.70	conventional activated sludge	0.24	Pounds BOD/ Pounds MLSS	(23)
13.5×10^{-8}	2.58	laboratory	-	-	(23)
3.8×10^{-6}	2.53	Conventional activated sludge	0.2 to 0.4	Pounds TOC/ Pounds MLVSS	(35)

$$G_1 = \left[\alpha(n-1)\right]^{1/n} \left[\frac{n}{n-1}\right] \left[\frac{rQ}{A}\right]^{\frac{n-1}{n}} \quad \ldots(15)$$

For the settler to be adequate in thickening, the flux applied to it must not exceed the limiting flux.

The applied flux, $G_a \geq \frac{Q\,(1+r)C_f}{A}$ ...(16)

Assuming $G_a = G_1$, combining equation (15) and (16) we have,

$$\frac{Q}{A} = \frac{\alpha(n-1)\left(\frac{n}{n-1}\right)^{n} (r)^{n-1}}{C_f^{\,n}(1+r)^{n}} \quad \ldots(17)$$

This equation permits the calculation of the overflow rate required to prevent the settler from being limited in thickening.

Re-writing (17),

$$\frac{Q\,(1+r)^{n}}{A\,(r)^{n-1}} = \frac{\alpha(n-1)\left(\frac{n}{n-1}\right)^{n}}{C_f^{\,n}}$$

Let the R.H.S. be equal to ψ

then, $\frac{Q}{A} = \psi\,\frac{(r)^{n-1}}{(1+r)^{n}}$ ...(18)

Where ψ is called the "Solids loading factor".

Jennings and Grady (45) summarised a number of potential weaknesses in conventional activated sludge design. They show particular concern over the fact that conventional design ignores the interactions between the aeration chamber and the final settling tanks and designs the two units independently.

Jennings and Grady (45) presented a design approach integrating aerator and final settling tank performance.

They included individual models for the costs of aerators and final settling tanks.

Although the area requirement of a thickener can be determined with reasonable accuracy, as described earlier, the volume requirement is determined by the time needed for further compaction beyond the concentration of the limiting layer. Analysis of compaction has been largely empirical. Coe and Clevenger recognised this and recommended laboratory tests to determine the time required to accommodate compaction and sufficient storage to compensation for fluctuations in the feed and discharge.

Compaction or settling of solids in the compression zone was assumed to be a first order function of time by Roberts (59).

$$\frac{-dH}{dt} = K\,(H-H_\infty) \qquad \ldots\ (19)$$

Where H is the height of slurry at time t and H_∞ is the final compacted height. Integration yields,

$$\ln\left[\frac{H - H_\infty}{H_c - H_\infty}\right] = -Kt \qquad \ldots\ (20)$$

H_c is the height of the interface at the start of compression, at which point, time t is taken as zero. H_∞ is determined by trial and error. If an assumed H_∞ is correct a plot of $(H-H_\infty)$ Vs t should be a straight line on a semi-log paper.

The volume of the compression zone could be assumed to consist of volume occupied by the solids and the volume occupied by the associated liquid, and the resulting equation is evaluated graphically from results obtained in batch tests. The height of the compression zone is determined by dividing the volume by the area for clarification. The total depth is then estimated by adding the calculated compression zone to the following :

Bottom Pitch : 1 to 2 feet
Storage Capacity : 1 to 2 feet
Submergence of feed : 1 to 3 feet

Related design procedures for evaluation of thickener volume requirements have not enjoyed the theoretical development and research that clarification has received, and are usually dictated by experience.

2.2 Process Variables

2.2.1 Biological Flocculation

The second phase in the development of a high quality effluent, as explained earlier, is the flocculation of the microorganisms and the other suspended or colloidal components into a readily settleable mass. In fact the effectiveness of the clarifier depends upon the degree of bio-flocculation and the settling characteristics of the activated sludge. The settling characteristics of the sludge are usually defined by its sludge volume index (SVI), zone settling velocity (ZSV), and percent of microorganisms which remain dispersed after long settling periods.

A review of the literature Bisogni and Lawrence(7), Ford and Eckenfelder (28), Pipes (56), Pavoni et al (58) pertaining to bacterial flocculation indicates that several distinct theories have been supported.

2.2.1(a) Interparticle Forces: Early investigators believed that physical attraction of particles (surface charges) was responsible for bioflocculation. Recent workers have however shown that surface charge reduction alone cannot explain bacterial flocculation.

2.2.1(b) Zoogloea ramigera: This theory postulates that bioflocculation was solely dependent on this particular group of bacteria possessing a gelatinous matrix. Support for this theory eroded when numerous other bacterial species were found capable of floc formulation.

2.2.1(c) P.H.B. Theory: The poly-beta-hydroxybutyric acid (P.H.B.) theory of bacterial agglutination directly correlates PHB content of cell and bioflocculation (11). Other authors have however concluded that the primary function of PHB is to act as reserve food to be utilised during endogenous growth.

2.2.1(d) Extracellular Polysaccharides: Pavoni, et al(58) extended previous observations which correlated the accumulation of extracellular polysaccharides to bacterial flocculation.

Their experimental results clearly indicate that bioflocculation takes place when the microorganisms have entered the endogenous phase. This is in agreement with previous work. Of major importance was their finding a decrease in culture turbidity with increase in extracellular polymeric content (see Fig.11).

Bisogni and Lawrence (7) demonstrated the existence of a similar relationship between mean cell retention time or solids retention time and the settling of activated sludge. In fact they found that the polysaccharide content increases with increase in θ_c (see Fig.12). They made the following general conclusions :

1) A relationship between SVI, ZSV and percent dispersed solids and θ_c exist.
2) Best overall solids removal occurred at values of θ_c in the range of 4 to 9 days.
3) The physical character of the solids in the effluent was dependent on the value of θ_c i.e. short θ_c effluent contained dispersed growth, while long θ_c effluents contained pin point floc and small deflocculated particles.
4) At longer θ_c values settling and bioflocculation is accompanied by an accumulation of polysaccharidic materials.

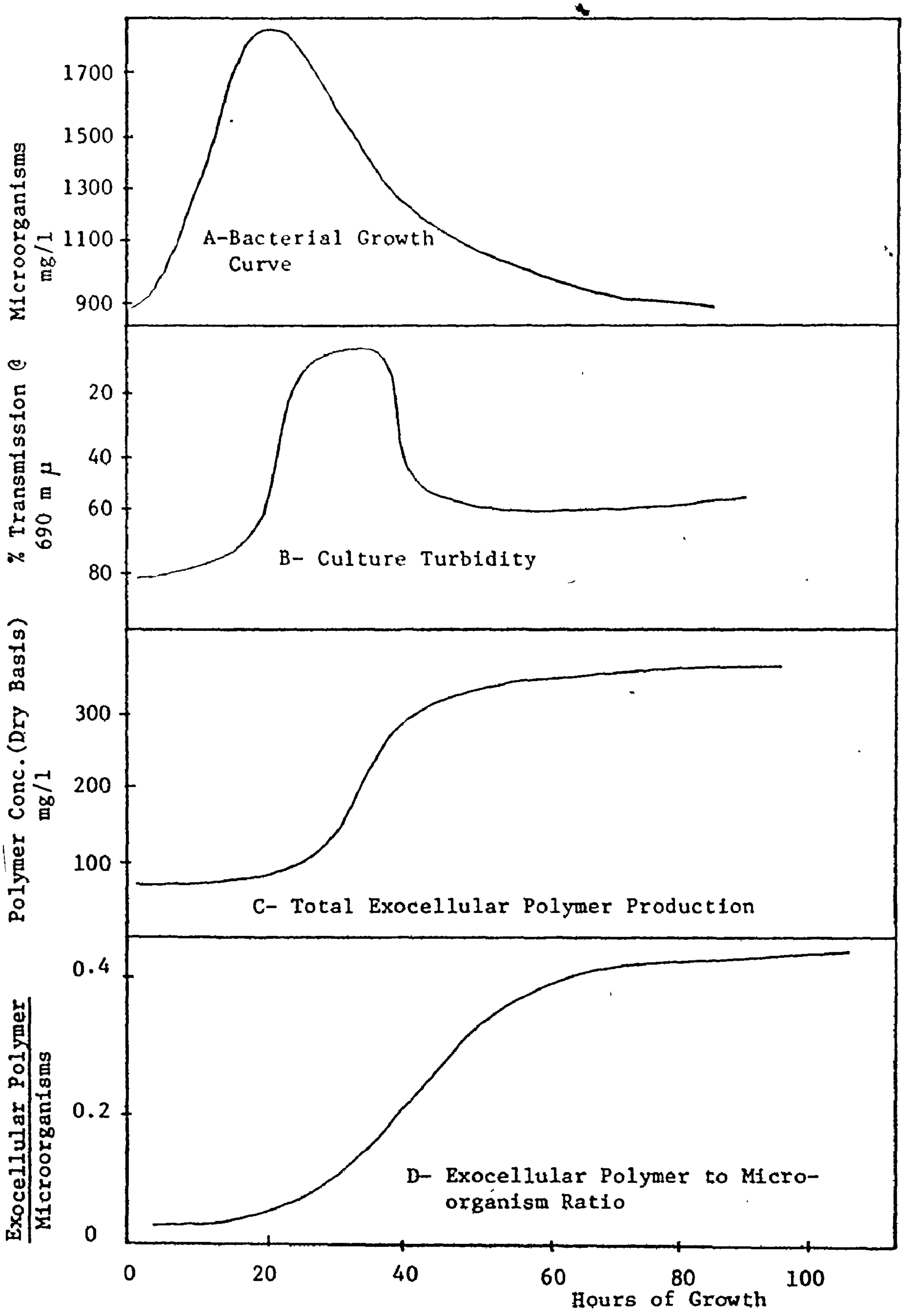


Fig.11- Factors affecting biological flocculation II, Pavoni, et al. (58)

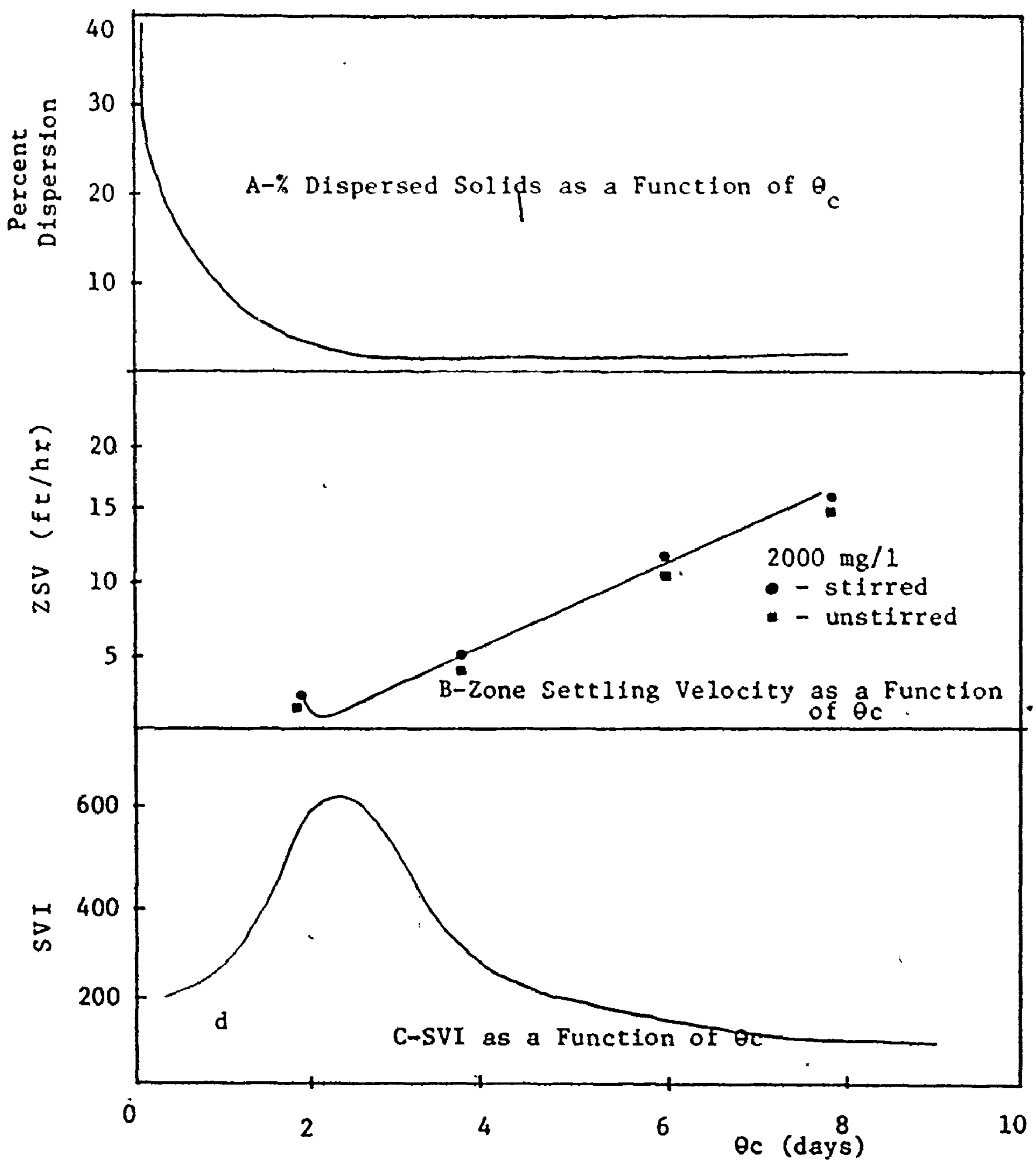


Fig.12- Factors affecting Biological Flocculation, I
Bisogni & Lawrence (7)

2.2.2 Effects of Physical Variables

2.2.2(a) Temperature Effects : There are several possible explanations for differences in settling rates under various temperature conditions;

(a) Change in viscosity of the liquid,

(b) Change in surface tension,

(c) Effects of currents, and

(d) Ability of material to flocculate.

A rise in temperature decreases the viscosity of the liquid and causes thermal currents. These were found to be fairly significant by Rudolfs and Lacy (60). They found that when sludges were aerated at different temperatures than settling, better settling and compacting occurred at the higher temperatures. When settling was carried out at the same temperature as aeration, similar results were obtained. White (73) reports that a 0.2°C temperature difference between the influent and the bulk liquid in the clarifier can be detrimental to the use of tube settlers. In general, flocculation has been found to be more effective at higher temperatures.

2.2.2(b) Pressure Effects : Rudolfs and Lacy (60) showed that settling is independent of the variations in pressure normally experienced in secondary plants.

2.2.2(c) Density Currents : Density current is a gravity flow produced in one fluid by another fluid of slightly greater density seeking the lower level. This happens when mixed liquor of an activated sludge plant enters the final settling tanks. The high concentration of solids in the mixed liquor causes it to behave as a liquid of slightly greater density. This causes the mixed liquor at the inlet of the clarifier to plunge to the bottom and flow along the bottom of the tank until deflected by some obstruction, usually the side of the tank, inducing a counter-current in the upper levels back towards the influent. Anderson(2) reported that bottom velocities of up to 7 ft. per min. and return velocities of up to 4 ft. per min. can co-exist in one tank depending on tank geometry and influent velocity. Although Green (32) substantiated that bottom velocities of 7 ft. per min. exist, he also reported an instance when the bottom velocity was as low as 2 ft. per min. This agrees with a recent report by Heinke and Qazi (43).

Anderson (2) made the following general suggestion for design, in order to minimize the effects of density currents:

(a) for circular tanks with center inlet, the radius should not exceed 5 times the side water depth;

(b) for rectangular tanks, where there is a better opportunity to reduce inlet velocity, the flow length should not exceed 7 times the depth, and

(c) the depth below the weirs should not be less than 10 feet in order to avoid any disturbance caused by the scraper movement and density currents, not less than 12 feet if the weirs are located at the upturn of the density currents. (Please refer to Appendix B.1 for details of the clarifier understudy)

2.3 Tank Hydraulics

2.3.1 Tracer Studies

In an ideal basin, fluid travels smoothly from inlet to outlet without lateral dispersion. The particles of fluid move in parallel paths analogous to soldiers marching in line. Under such conditions if a given quantity of dye is added in a short interval of time at the inlet and the dye concentration in the effluent is monitored, the dye (tracer) will appear without having mixed after a period of time equal to the theoretical detention time. Ideally, there are no dead zones or short circuiting. The effluent tracer concentration vs time curve (dispersion curve) (see Fig. 13) will have a standard deviation and variance equal to zero as there is only one observation.

In a completely mixed tank, the tracer added would instantaneously and homogenously mix, and the dispersion curve will proceed according to an exponential pattern (see Fig, 13). The hydraulics in "real" sedimentation tanks lie somewhere between these two extremes as shown in Fig. 14.

An almost complete description of the hydraulic

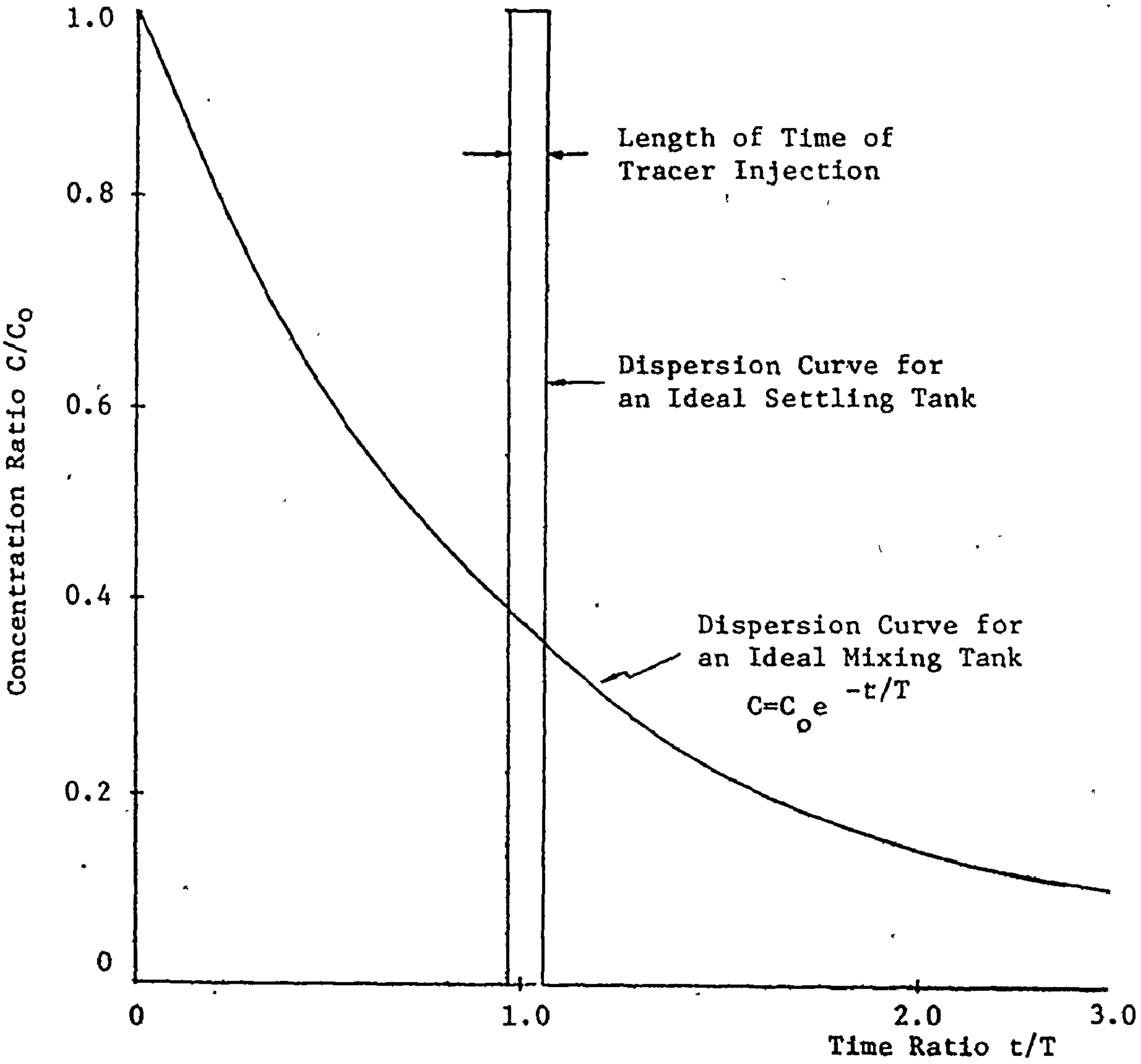


Fig.13- Dispersion Curves for Ideal Tanks

behaviour pattern within a tank could be obtained by the tracer dispersion curves. The name "dispersion" suggests a spreading of the tracer in terms of time, ahead of or behind the time position it would occupy in an ideal basin i.e. theoretical detention time. A number of workers, Morrill (52) Levenspeil (49), Murphy (53), Thirumurthi(70) and Villemonte and Rohlich (71), have introduced various non-dimensional ratios that one could use to describe hydraulic behaviour within the sedimentation tank from the dispersion curve. The following is a summary of these ratios: (Please refer to Fig.14.)

t_i/T : Indicates the worse case of short circuiting. (For an ideal tank it will be 1.0)

t_p/T : Measures average short circuiting.

t_g/T : Represents the effective basin volume. Values less than 1.0 indicates existence of stagnant areas.

t_{90}/t_{10} : This is a measure of the ratio of mixing to short circuiting. It is 1.0 for an ideal basin.

$I_s = \frac{t_g - t_p}{t_g}$: Index of short circuiting. Introduced to envelope the observation scatter and the variation in the tail i.e. measures the relative skewness of the tracer curve. It will be 0.0 for ideal tanks and 1.0 for ...

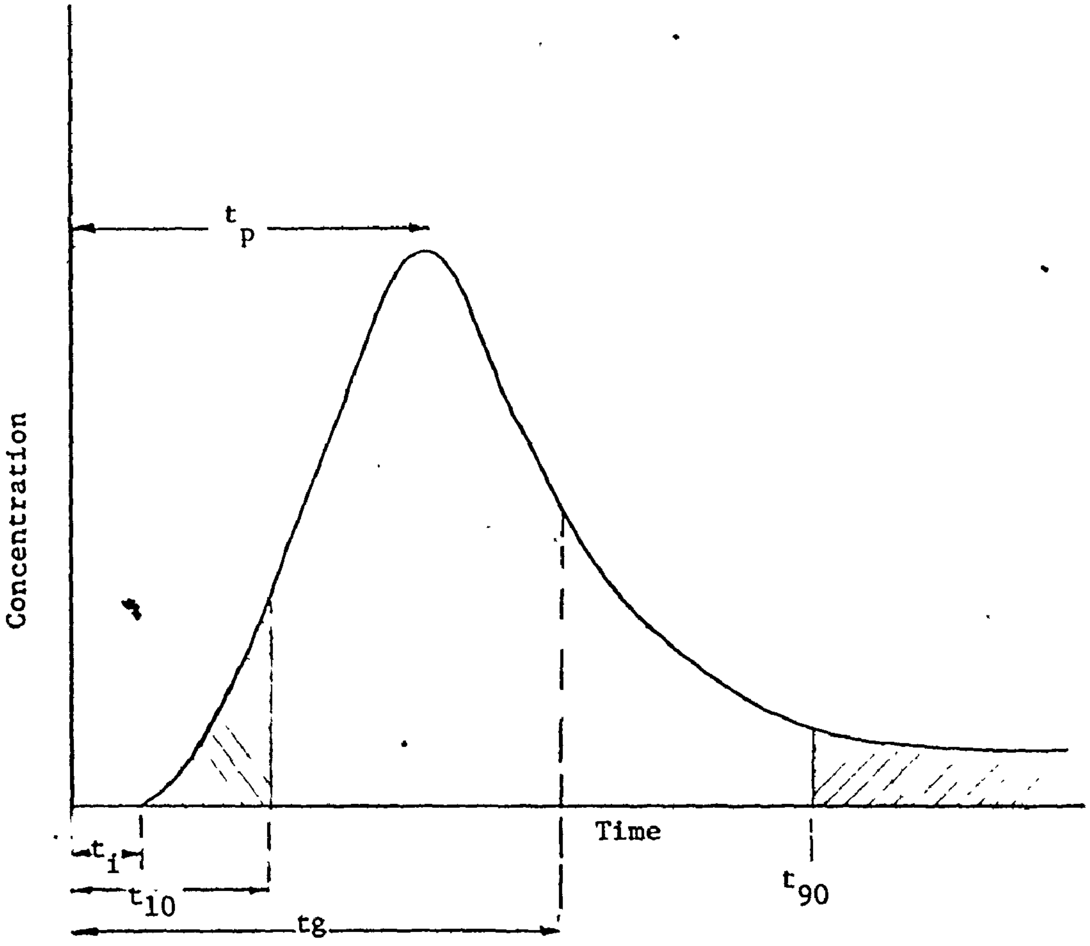


Fig.14- Dispersion Curve of a Real Basin

t_i = Time to initial indication of tracer. Indicates most severe short circuiting.

t_p = Time to peak concentration. Indicates average short circuiting and this is a function of dead spaces.

t_{10}= Time for 10 per cent of tracer to pass. A function of large scale turbulence.

t_g = Time to center of gravity of the area under the curve.

t_{90}= Time for 90 per cent of tracer to pass. A function of large scale turbulence or eddies and thus mixing.

T = Theoretical detention time.

completely mixed basins.

$d=\frac{D}{U'L'}$: Dispersion Index. This is calculated from the variance of the C curve and includes all points in the curve. It will be 0.0 for ideal tanks and infinity for completely mixed basins (see Fig.15).

Morrill (52) in 1932 introduced statistical analysis of dispersion curves. He replotted time on a logarithmic scale and the cumulative tracer concentration on a probability scale. Except for the ends of the curve, a straight line relationship existed between 10 per cent and 90 per cent recovery. Relating the slope of this line to the degree of dispersion (steeper the slope, the worse the dispersion) he introduced the ratio t_{90}/t_{10} and called it the dispersion index.

According to Murphy(53) and Villemonte and Rohlich (71) this ratio will be affected by the differences in the "tail" of the curve. They introduced the index of short circuiting which meausres the interaction between short circuiting and dispersion while at the same time accounting for variation in the "tail". In effect, it envelopes both t_p/T and t_{90}/t_{10}.

Based on the argument that statistical reliability of one single point in the dispersion curve is less than that of several points Thirumurthi(70) explained the applicability of the dispersion index d, as defined by Levenspeil (49).

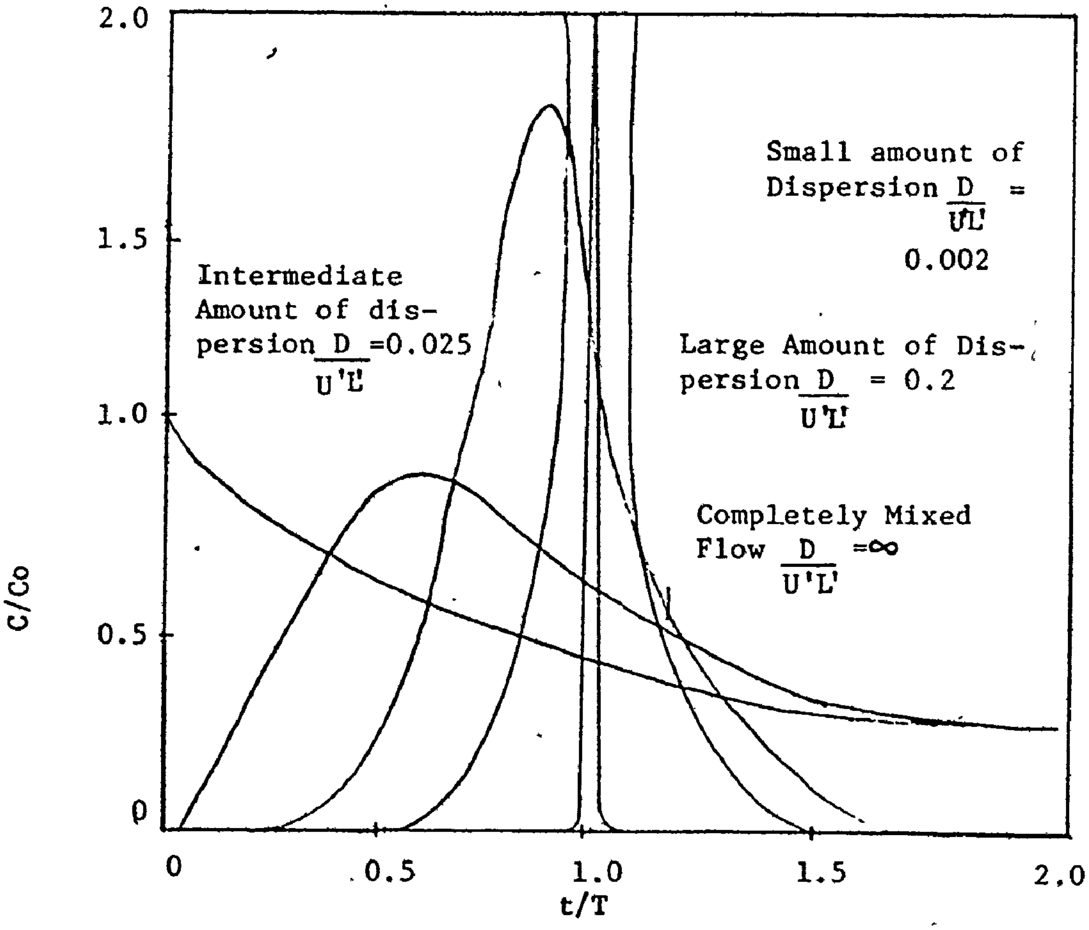


Fig.15- Extent of Mixing as Predicted by the Dispersion Model, Levenspeil (49)

The ratios t_i/T and t_p/T are widely used (70) (43) in describing hydraulic dispersion. But, since only a small quantity of tracer is recovered at time ti, it is not considered to be as a good measure of overall short circuiting as t_p/T.

Fiedler and Fitch (26), Wallace (72), Thirumurth(70) and Heinke (43) attempted to correlate hydraulic behaviour with efficiency of solids removal.

The universal applicability of such a correlation, especially in secondary clarification, is still suspect. Murphy (53) reported that if t_g/T tends towards unity, i.e. no stagnant areas, there should be a relationship between dispersion index t_{90}/t_{10} and modal time ratio t_p/T. He in fact showed a reasonable linear correlation to exist.

Wallace (72) summarised the necessary conditions that must be met in order to achieve a reasonable analysis of the sedimentation basin through tracer techniques -

a) Curves obtained from the same basin at identical operating conditions should be in close agreement;

b) Curves obtained from the same basin at different operating conditions should be comparable;

(c) Comparisons with other basins operated under identical conditions must be possible; and

(d) Tracer recovery should be within 70-110%.

A study of the hydraulic regime of a sedimentation basin describes :

a) design anomalies with respect to inlet and outlet design;

b) the existence of stagnant areas;

c) anomalies of the position and loading rates of weirs and launders (short circuiting), and

d) inadequacies, if any, in the method of sludge withdrawal.

It does not however distinguish between density current short circuiting or streaming. Also it does not reveal the behaviour pattern of the solids in the tank. Priesing (57) showed that when both liquid and solid particles were tagged with two different tracers (Rhodamine WT dye for liquid and Radioactive Iron-59 for solids) and monitored simultaneously, the return solids curve showed very sharp ascending and descending slopes which indicates an almost plug-flow condition probably due to stratification of the particulate matter in the clarifier. The return dye curve exhibits (see Fig. 16) a narrower peak and a broader base. This is probably caused by entrapment of the liquid within the solids blanket. The peak radioactivity concentration appeared almost 30 minutes before the peak dye concentration, indicating that solids travel faster through the sludge blanket than liquid does.

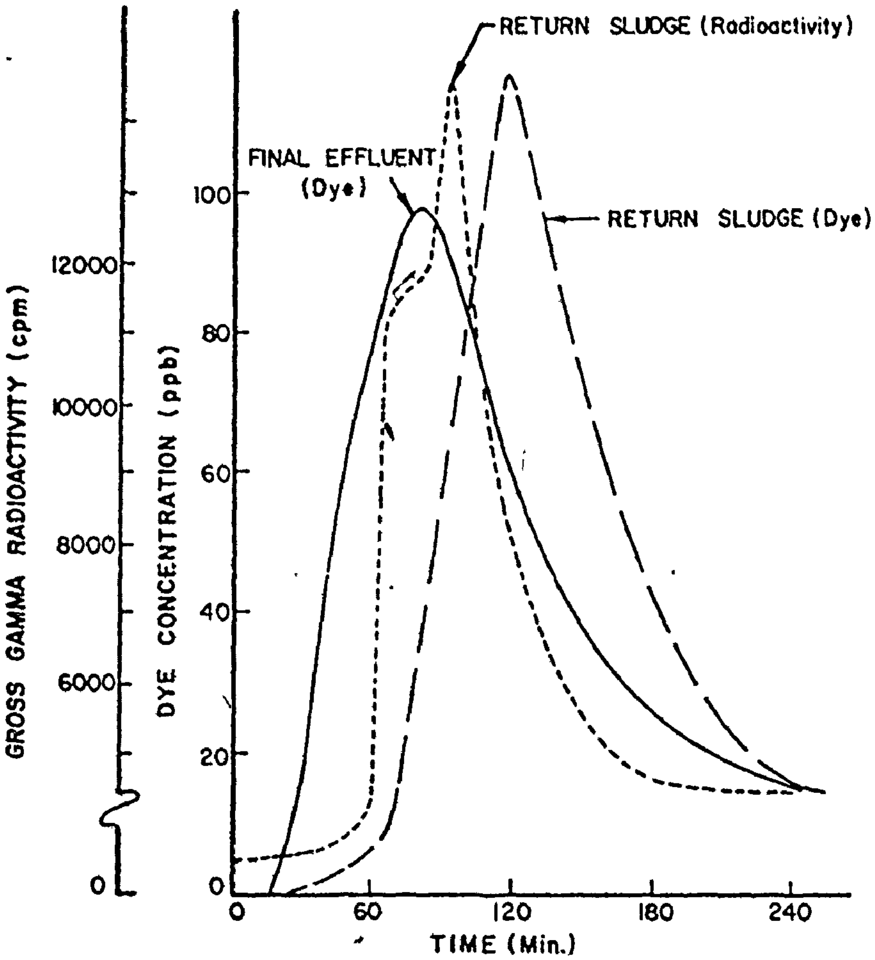
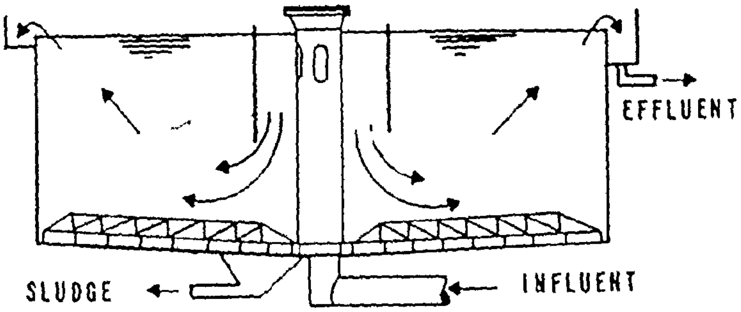


Fig. 16 . Detention of Solids and Liquids in a Final Clarifier (57).

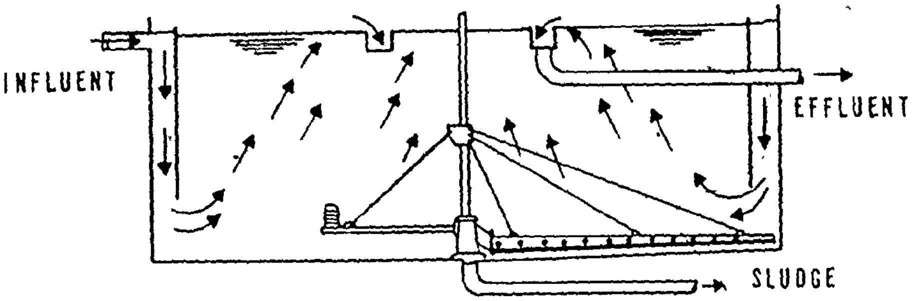
The degree to which the hydraulic regime will influence the efficiency of biological solids separation, depends on the velocity of the particles, the rate of particle growth or flocculation, the rate at which settled matter can be removed from the basin and the solids content of the underflow. Katz (46) reported that effects of underflow hydraulics to be relatively small when the sludge return rates are low. For example, 10-15 per cent. At return rates greater than 25 per cent, however, the method of withdrawal will effect the deposition pattern of the sludge and the solids concentration of the underflow.

Therefore, a complete description of a clarifier cannot be achieved by studying only the hydraulic regime, but must also include a study of the solids behaviour pattern. Recognising the preceding shortcomings, equipment manufacturers have established various basin designs and methods of sludge withdrawal (see Fig. 17).

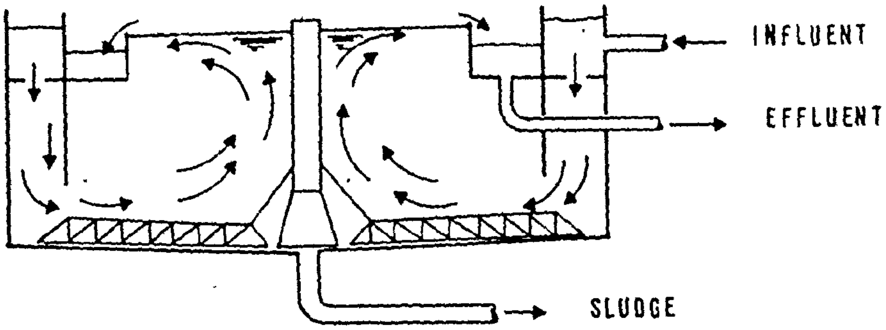
An introduction to fluoresence, tracer selection and a description of tracer detection instrumentation can be found in Appendix C.



CIRCULAR CENTER-FEED CLARIFIER WITH
A SCRAPER SLUDGE REMOVAL SYSTEM



CIRCULAR RIM-FEED. CENTER TAKE-OFF CLARIFIER WITH A
HYDRAULIC SUCTION SLUDGE REMOVAL SYSTEM



CIRCULAR RIM-FEED, RIM TAKE-OFF CLARIFIER

Fig.17- Typical Clarifier Configurations

2.4 Shallow Depth Sedimentation

2.4.1 Application of Idealised Sedimentation Theory :

The basic theory of gravity clarification in settling basins was first proposed by Hazen (40) in 1904. Fig.18 shows the theoretical settling pattern that would exist in an idealized continuous horizontal-flow basin. Settling paths of discrete particles are determined by the vector sums of the particle settling velocity v_s and the displacement velocity of the basin v_d. All particles having settling velocities v_s greater than v_o will fall through the entire depth h_o and will be removed, v_o being the velocity of a particle falling through h_o in settling time t_o. It can be seen that, $v_o = h_o/t_o$, $t_o = C/Q$, and $C/h_o = A$, where Q is the basin flow rate, C is the volumetric capacity of the settling zone, and A is the basin surface area. Substitution reveals that $v_o = Q/A$, is the surface loading or overflow velocity of the basin. The proportion of particles having settling velocities v_s less than v_o which will be removed is equal to the ratio of the velocities:

$$V_S / v_o = v_s/(Q/A) \qquad (1)$$

Recognising this fact. Hazen proposed that for discrete particles and unhindered settling conditions, basin efficiency is primarily a function of the surface area of the basin, and is independent of basin depth and displacement time or detention period (40). Thus, doubling the surface area will double settling efficiency.

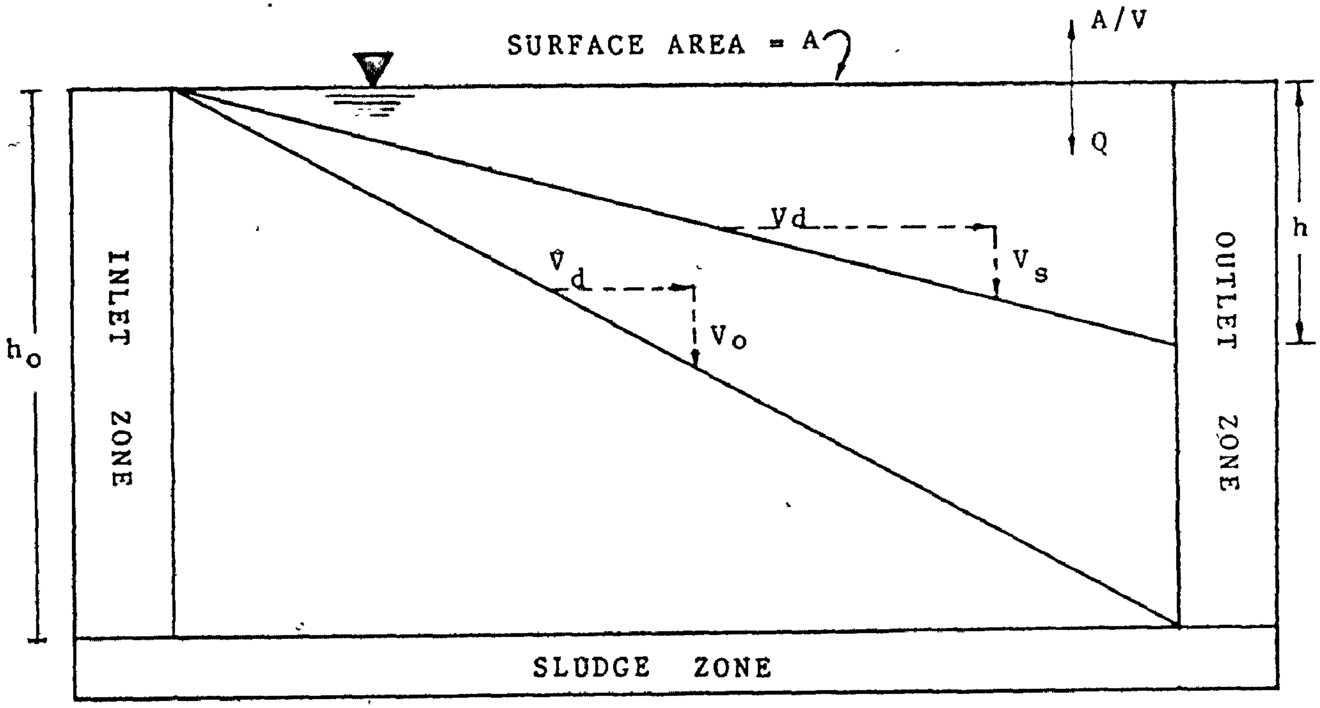


Fig.18 - Settling Paths of Discrete Particles in A Rectangular Sedimentation Basin

From Fig. 18 it can be seen that the effective settling surface of a basin can be increased by inserting horizontal trays at various depths. It follows that particles with settling velocity $v_s = v_o$ can be fully captured in a horizontal-flow basin if the trays are placed at intervals $h = v_s\ t_o$. The greater the number of trays the smaller the settling velocity of the particles may be. Structurally the possible number of trays is limited by the required clearances needed to remove the collected sludge.

A number of attempts have been made to practically apply the shallow depth gravity sedimentation principles proposed by Hazen. These were recently reviewed (38). One of the earliest attempts appeared as a 1915 patent(5). Here, several conical circular trays nested above each other formed a series of shallow settling compartments. Solids collected were to be scraped into a central sludge transport tube. Several patents for similar schemes appeared in the next few years.

Interest in shallow depth clarification increased during the 1940's and early 1950's. At least two companies were commercially marketing multi-story tray-settling tanks, (64) and several full-scale designs were reported (9) (29) (61) and (24). In spite of these efforts and general recognition of the theoretical advantages of the shallow depth process, the method still did not receive widespread acceptance in the water and wastewater treatment field. To some extent this may have been due to the reluctance of engineers to stray from traditional design, but primarily it resulted from an acknowledgement of the difficulties encountered in proper operation of tray settlers, namely: unstable hydraulic conditions encountered with the wide shallow trays commonly used which made equal flow distribution difficult, and the limitations of tray spacing mandated by sludge removal requirements.

In the mid 1950's, Camp (10) and Fischerstrom (27) restated the case for tray settling, noting the many advantages that could be realized if basins could be designed with the maximum number of horizontal trays while still permitting expedient removal of sludge. Fischerstrom stressed the need for maintaining proper hydraulic conditions for clarification and suggested that settling would be significantly improved if the Reynolds number were lowered into the laminar range.

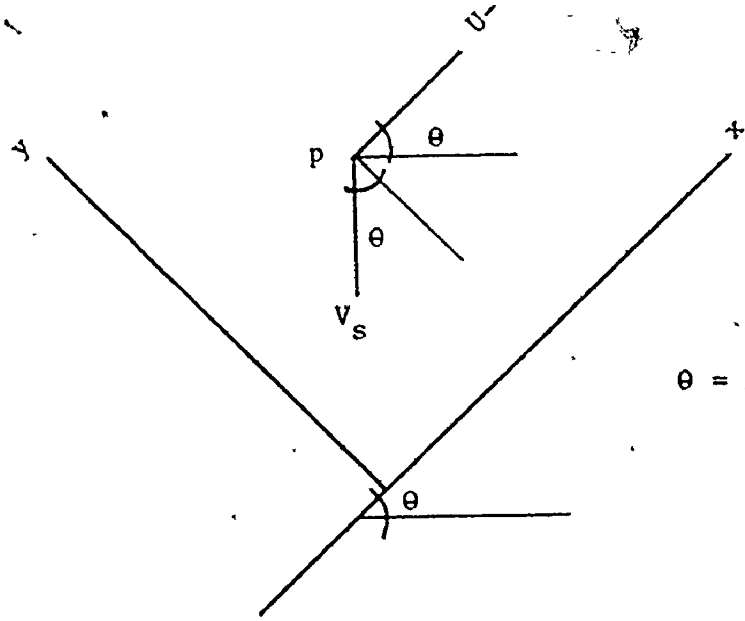
The limit of laminar flow is approximately a Reynolds number of 500. Most conventional settling basins are operated at Reynolds numbers ranging between 1,000 and 25,000. The Reynolds number can be practically reduced only by increasing the wetted perimeter. Fischerstrom recognised that this could be effected quite easily by inserting horizontal or vertical baffles into the clarifier. An overflow rate of 2 US gpm/sq.ft. of clarifier surface area corresponds to a Reynolds number of 41.3 for MicroFloc tube settlers (see Appendix A.3). Gray and Churchill (36) reported that the efficiency of removal of suspended particles is favoured by the simultaneous occurrence of a high Froude number and a low Reynolds number. Culp (12), Culp, Hansen & Richardson (16), and Hansen, Culp and Stukenberg (37) established by means of laboratory, pilot plant and plant scale studies the optimum values of the essential physical parameters such a tube length, tube diameter and angle of inclination for gravity sludge withdrawal. They did not however propose a design equation but recommended overflow rates for various waste streams based on tube entrance area.

The first real attempt to arrive at a design equation incorporating these variables and overflow rate was done by Yao in 1970 (75).

He attempted to establish the significance of overflow rate in the design of tube settlers. Like Hazen he made a few simplifying assumptions -

a) Flow pattern in tubes are laminar with no lateral mixing;
b) particles are discrete and do not aggregate, and
c) once a particle strikes the lower surface of the tube settler it is considered removed.

In a methodical analysis using Stokes law, Yao (75) derived the following equation to describe the various trajectories of the suspended particles in laminar flow through circular tubes (see Figs.19 and 20).



θ = Angle of Inclination of High-Rate Settler.

Fig.19- Coordinate System for Theoretical Study of High-Rate Settlers.

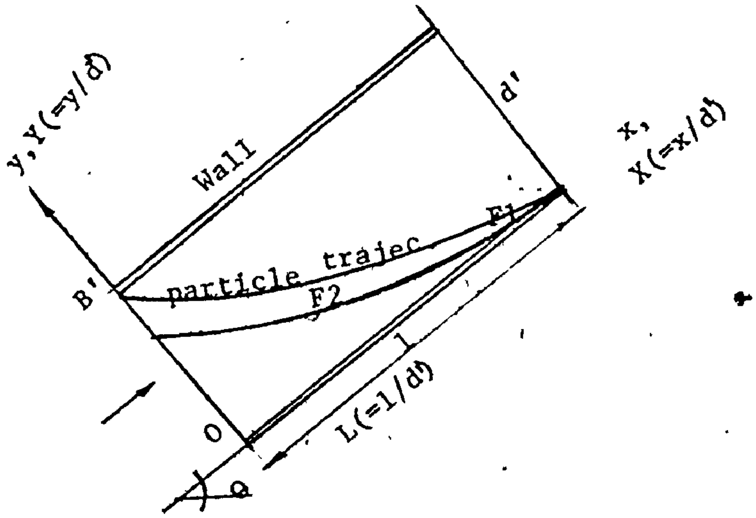


Fig.20- Sketch of Particle Trajectories in High-Rate Settler.

$$8\left[\frac{Y^2}{2} - \frac{Y^3}{3}\right] - \frac{V_s}{V_o} Y \sin\theta + \frac{V_s}{V_o} X \cos\theta = C \qquad ..\ 19)$$

Where the constant C can be evaluated if the co-ordinates at any point are known. Yao (75) obtained a useful relationship by substituting the co-ordinates for point B in dimensionless forms.

i.e. $X = \frac{l}{d'} = \frac{o}{d'} = 0$

and $Y = {}^{y}/_{d} = 1$

The result, $\frac{V_{sc}}{V_o} (\sin\theta + L\cos\theta) = \frac{4}{3}$ ...(20)

Where V_{sc} is the critical particle fall velocity and V_o is the average flow velocity.

Let $S = \frac{V_s}{V_o} (\sin\theta + L\cos\theta)$ ...(21)

The critical value of S, i.e. S_c for circular tubes is $\frac{4}{3}$ as determined by the deviation of equation (20) Yao (75) (76) proceeded to establish values of S_c for square conduits ${}^{11}/_{8}$, shallow open trays 1, and flow between parallel plates 1.

Theoretically therefore any suspended particle having a fall velocity greater than or equal to V_{sc} would be completely removed. Based on this logic, Yao (75) (76) rearranged a general form of equation (20) to relate to overflow rate inserting an unit adjustment constant.

$$\text{Overflow rate} = V_{sc} = C\,S_c\,\frac{V_o}{\sin\theta + \cos\theta} \quad \ldots (22)$$

Depending on the system of units used the proposed values of C:

$$C = 1.08 \times 10^4 \quad \text{(f.p.s.system)}$$

$$\text{and} \quad C = 14.4 \quad \text{(c.g.s.system)}$$

Equation (22) suggests that the efficiency of high rate sedimentation is a function of the angle of inclination, length of tube and the average liquid velocity. Yao (75) proceeded to show that fractional removal efficiency is equal to the S value of the suspended particles as defined by equation (21).

In 1970, Hernandez and Wright (42) attempted various combinations of variables and curve fitting devices to develop a generalised design criteria for tube settler applications. The best fit was obtained when turbidity removal was plotted against V^2R/L. Where R is the hydraulic radius, V the velocity of flow and L length of the tube settler. This means that there is some trade off between hydraulic radius, flow rate and tube length. (See Table III for recommended values of flow).

For the removal of activated sludge using tube settlers they plotted V^2R/L against an efficiency factor which they defined as :

$$EF = \frac{\text{MLSS-E.SS}}{\text{MLSS}} \quad \frac{100}{\log \text{SVI}}$$

TABLE III

Effect of tube length and diameter on tube rate

	Maxm.Tube Rates in US gpm/sq.ft* ($\theta = 60^{\circ}$)			
Diameter (inches)	Tube Length (feet)			
	2	4	6	8
1	0.17	0.25	0.30	0.33
2	0.12	0.12	0.21	0.25
3	0.10	0.14	0.17	0.20
4	0.08	0.12	0.15	0.17

Ref. (42)

* Although the type of tubes used was not reported, it is assumed to be MicroFloc tube_settlers, which provide approximately 12 sq.ft. of settling area per sq.ft. of plan area (see Appendix A.4).

Where : MLSS is the concentration of suspended solids in the mixed liquor (mg/l),

E.SS is the suspended solids content of the settler effluent, and

SVI is the sludge volume index.

Their efforts to obtain a satisfactory plot without taking into account the variation of settleability was unsuccessful. The data scatter cast some doubt on the validity of the equation. They recommend the value of V^2R/L to be maintained less than 2×10^{-7}. It is interesting to note that for a tube 2 inch square and 2 feet long, this would mean an overflow rate of 0.08 gpm/sq.ft. of tube settling area.

In 1972 Beach (6) reported a correlation to exist between relative length (tube length/tube equivalent diameter) and maximum tube rate.

$$\text{Maximum tube rate (US gpm/sq.ft)} = \left[\frac{\text{Tube length}}{\text{Tube equiv.diam}}\right]^{0.8}$$

This means if the ratio between the brackets is doubledthe flow rate can be increased by about 75%.Since the tube equivalent diameter is a function of the wetted perimeter the maximum tube rate would also be affected by shape.

None of these correlations nor theories have been re-tested since the initial presentations.

In summary, Hazen (40) Camp (8) pointed out the advantages that could be accrued by increasing the settling area and reducing the depth of settling. The early work by Camp (8) Frei (29) Dresser (24) and Schmitt and Vorgt (61) established (a) that tubes with relatively small diameters having a large wetted perimeter will overcome the hydraulic short-coming which exist in large conventional basins, and (b) sludge withdrawal problems could be overcome by inclining the tubes. Yao(75),Hernandez & Wright(42) and Beach(6) provided for the theoretical arguments to elucidate Hazen's idea. These developments paved the way for pilot plant study.

2.4.2 Pilot Plant Studies

In 1964 tests were begun at Neptune MicroFloc,Inc. Laboratories to evaluate the efficiency of settling tube configurations and various methods of sludge removal (37) (38). The tests were conducted on flocculated raw wastewater and employed turbidity as the measure of efficiency. Preliminary studies conducted with single tubes of different sizes demonstrated a high efficiency of sedimentation. At the same time, it was discovered that if the tubes were inclined slightly (5°), sludge deposits could be removed by periodically allowing the tubes to drain. The performance of the tube settlers was dependent upon tube length, tube diameter, flow rates, the nature of the incoming settleable matter, and chemical addition.

Relatively small diameter tubes (1 inch to 4 inches) which were 2 to 4 feet long provided best sedimentation, with settling times of 6 minutes or less.

Pilot scale tests were subsequently conducted on a multi-tube unit. Both coagulated wastewater and activated sludge were examined using a variety of tube diameters and lengths. The activated sludge results are of great interest. The mixed liquor from an extended aeration package plant as well as well as conventional activated sludge were tested.

For the package plant mixed liquor, the tubes performed satisfactorily but fouled quickly with sludge at all tube rates greater than 0.07 gpm/sq ft.* resulting ultimately in poor effluent quality. The sludge storage times for all tube diameters and lengths were too short to permit feasible tube drainage cycles. Under continuous application of the mixed liquor, which averaged 4,900 mg/l suspended solids, the tubes were not practical. It appeared to the investigators, however, that horizontal tubes might be useful in handling the short-term high solids flows which can occur at package plants and conventional plants during biological upset conditions.

To further evaluate this concept, tests were conducted on activated sludge mixed liquor averaging 5,500

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* Unless otherwise stated tube rates will be based on the extended settling area provided. (Please refer to Appendix A.4).

mg/l suspended solids. A tube settler hydraulic loading rate of 0.08 gpm/sq. ft. was used for all tests. Excellent solid/liquid separation was reported. The effluent averaged 20 to 25 mg/l suspended solids with sludge holding time (i.e. period between sludge clean-out cycles) of 1.5 and 1.75 hours. These times corresponded closely with those experienced in earlier single tube experiments. The effluent quality from the multiple tube unit was far superior to that obtained with single tubes. It was felt that differences in sludge settling characteristics and improved inlet and outlet hydraulics of the multi-tube unit were responsible for the better performance.

From these studies it became apparent that a tube settler might function as a supplemental solids separator prior to tertiary treatment with a mixed media filter (see Fig. 21). The tube settler would assist the system by absorbing abnormal fluctuations in solids concentration which would ordinarily cause filter binding. A long-term demonstration study was, in fact, carried out with an effluent polishing system integrating tube settling and mixed-media filtration. The results have been reported in detail (14) (15).

During the initial studies of horizontal tube settling, the effects of the angle of tube inclination were also studied (38). It was found that if the tubes were inclined steeply, the solids settling to the lower tube

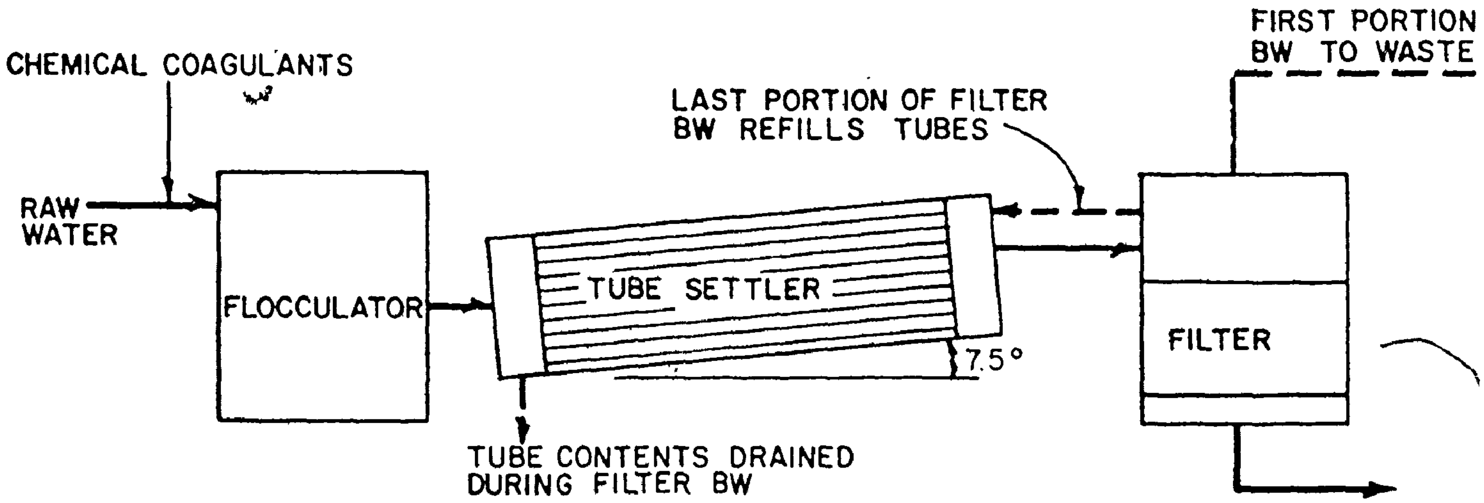

Fig.21- Essentially Horizontal Tube Settler

A. The flow schematic illustrates a treatment process utilizing essentially horizontal tube settlers in combination with a mixed-media filter.

B. Sludge is removed from the tubes by draining. A slight inclination (7-1/2°) in the direction of flow facilitates cleaning. This operation is carried out automatically, initiated on high filter headloss. The first portion of the backwash water is wasted together with the contents of the tube chamber. The last portion of the backwash water is used to refill the tube chamber, minimizing down time and wastage of water.

C. This automatic sludge removal system eliminates the need for operator judgment on when and how much sludge to waste.

D. The process is ideal for smaller treatment systems (10 gpm to 3 MGD).

surface would slide downward along the surface until finally exiting at the tube bottom. At angles greater than 45° a natural and continuous evacuation of solids could be achieved. In light of these results and the predetermined ability of horizontal tubes to clarify mixed liquor at high hydraulic loading rates, a study was begun to determine the feasibility of steeply inclined tube settling for activated sludge mixed liquor (see Fig.22).

A 3,000 gpd pilot unit was developed for testing in conjunction with an existing treatment plant. The unit employed 2 inch by 4 feet long hexagonal settling tubes which were incorporated into a package plant consisting of mixed liquor aeration and mixed-media filtration. The tubes, constructed of PVC plastic and inclined at 60°, provided continual solids separation within the aeration basin without need for supplemental sludge collection and return equipment. The effluent solids concentration from the tubes averaged 60 mg/l at a detention time of only 15 minutes. The settler also showed stable performance when surged with flows 50 to 100 per cent above the design flow of 2 US gpm.

The first pilot plant study on primary clarification of raw wastewaters with tube settlers were performed using single tubes (67). The tubes were 2 feet and 4 feet long, with a 2 inch settling depth and incline at 60°. The tube clarifiers were considerably more efficient in removing settleable solids as compared to a similar unit without tubes.

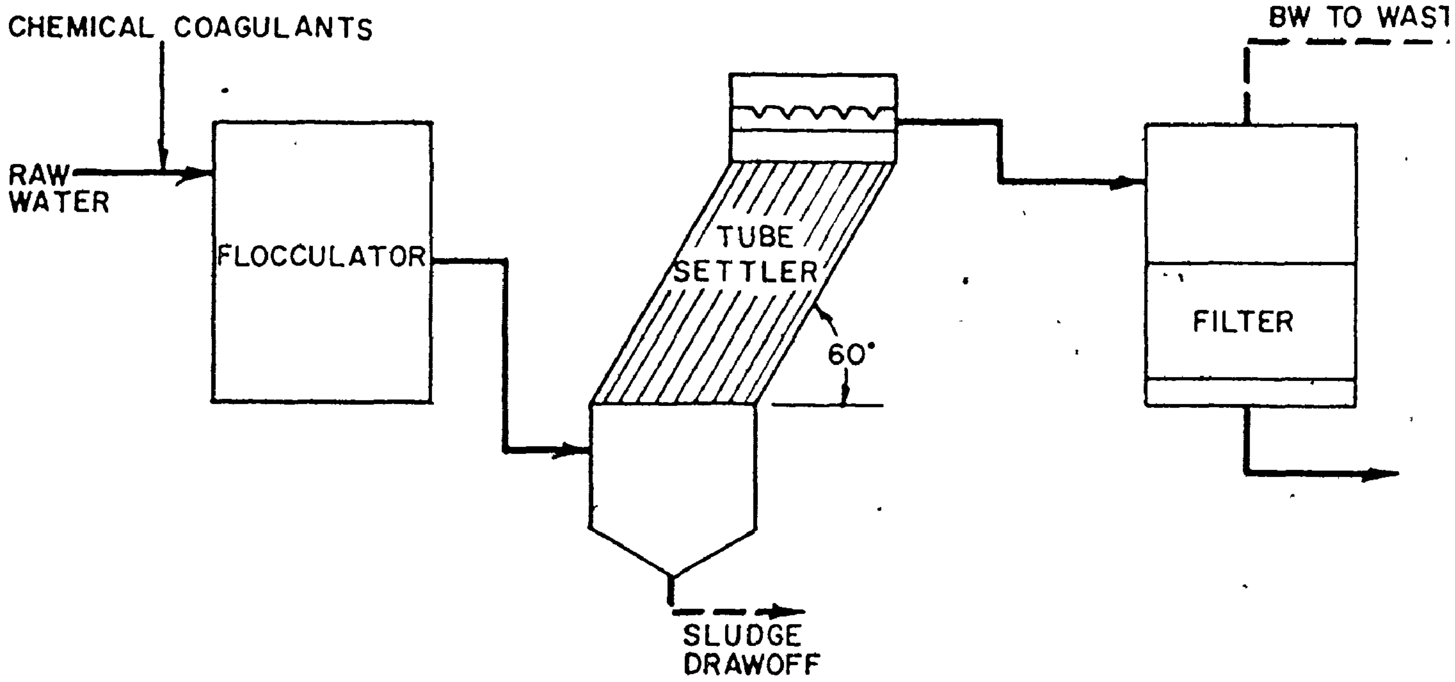

Fig.22- Steeply Inlcined Tube Settler

A. Research on tube settlers revealed that at a 60° angle the sludge which accumulates will slide down along the tube bottom and be removed continuously. This phenomenon eliminates the need for draining the tubes for periodic sludge removal. The filter backwash does not need to be coordinated with the sludge drawoff.

B. The steeply inclined tubes are applicable to plants of any capacity. Normal sludge removal methods - i.e., manual, scraper mechanisms, etc., - can be used.

C. Individual tube modules are constructed in a crisscross configuration. This design results in modules which are rigid and self-supporting, using 2 inch by 2 inch channels inclined at 60° angles. The typical module is 30 inches wide by 20 inches deep and 10 feet in length and weighs 65 pounds. Such a module provides 300 square feet of settling area and covers a water surface area of 25 square feet.

However, only a slight degree of improvement was possible with the 4 foot long tubes as compared to the 2 foot long tubes. The suspended solids removal was unaffected up to 0.5 US gpm/sq.ft. These experiences demonstrated the feasibility of high rate, shallow depth clarification using steeply inclined, small diameter tubes. At this point in time, Neptune MicroFloc has installed numerous tube settlers for water purification and industrial and municipal wastewater treatment. Units have been designed into new plants, incorporated into proprietary package treatment plants, and applied to improve the performance of existing clarifiers. Figures 23 and 24 illustrate the manner in which tube modules may be installed in existing rectangular and circular clarifiers.

In addition to tube settlers, improvement in conventional tray settling has also been achieved by the Lamella Separator in Sweden (30). This settler consists of a series of narrowly spaced inclined plates arranged in a plate pack as shown in Fig. 25. The suspension to be settled is fed into the top of the unit. Flow is downward through the plates as opposed to the upflow situation in tube settlers. Sludge and liquid thus flow in the same direction, sludge being deposited in a sludge collector, while the clarified liquid is drawn off through separate discharge pipes. The Lamella Separator has been applied on only chemically coagulated wastewaters at this time. There have been no reports of its use in activated sludge clarification.

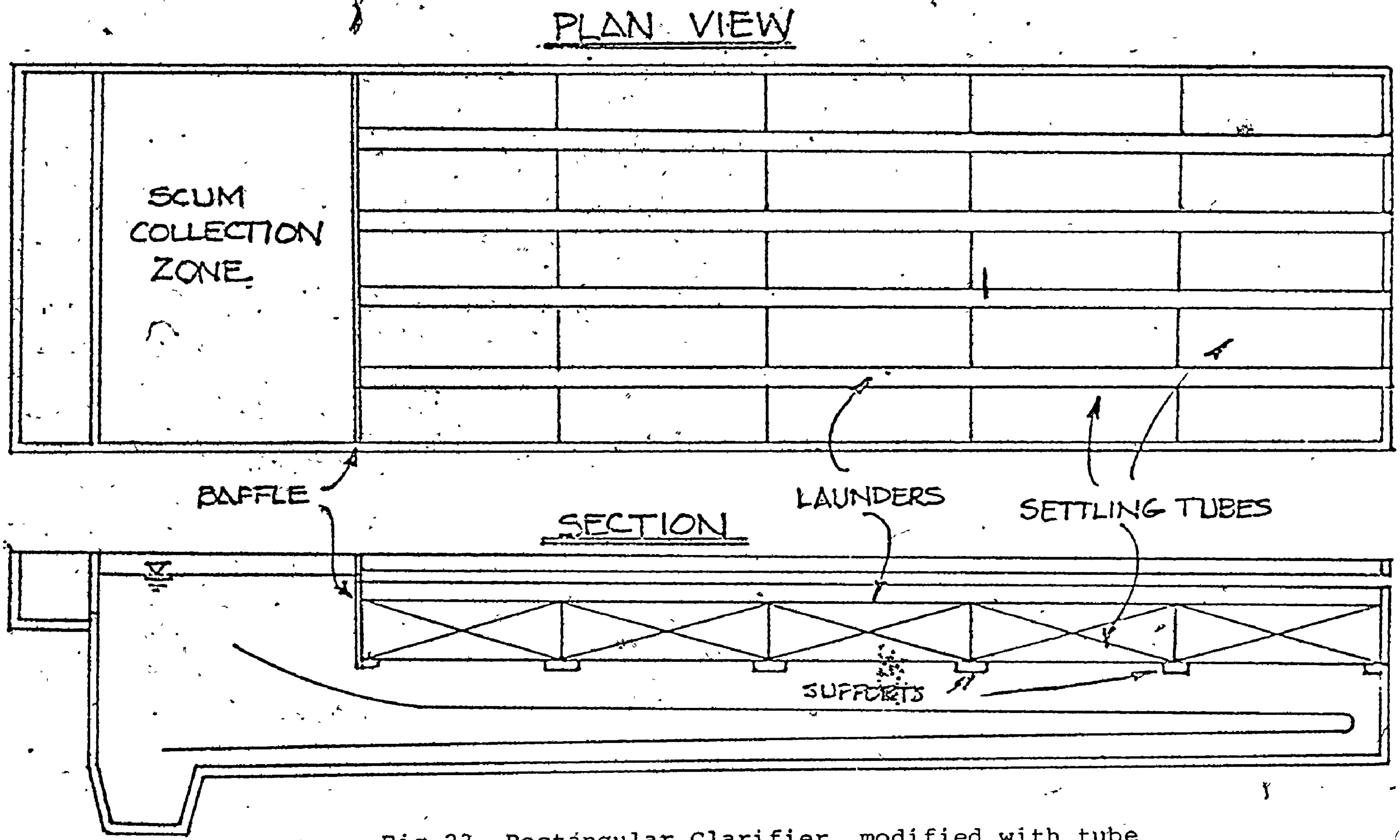

Fig.23- Rectangular Clarifier modified with tube Modules (Neptune MicroFloc)

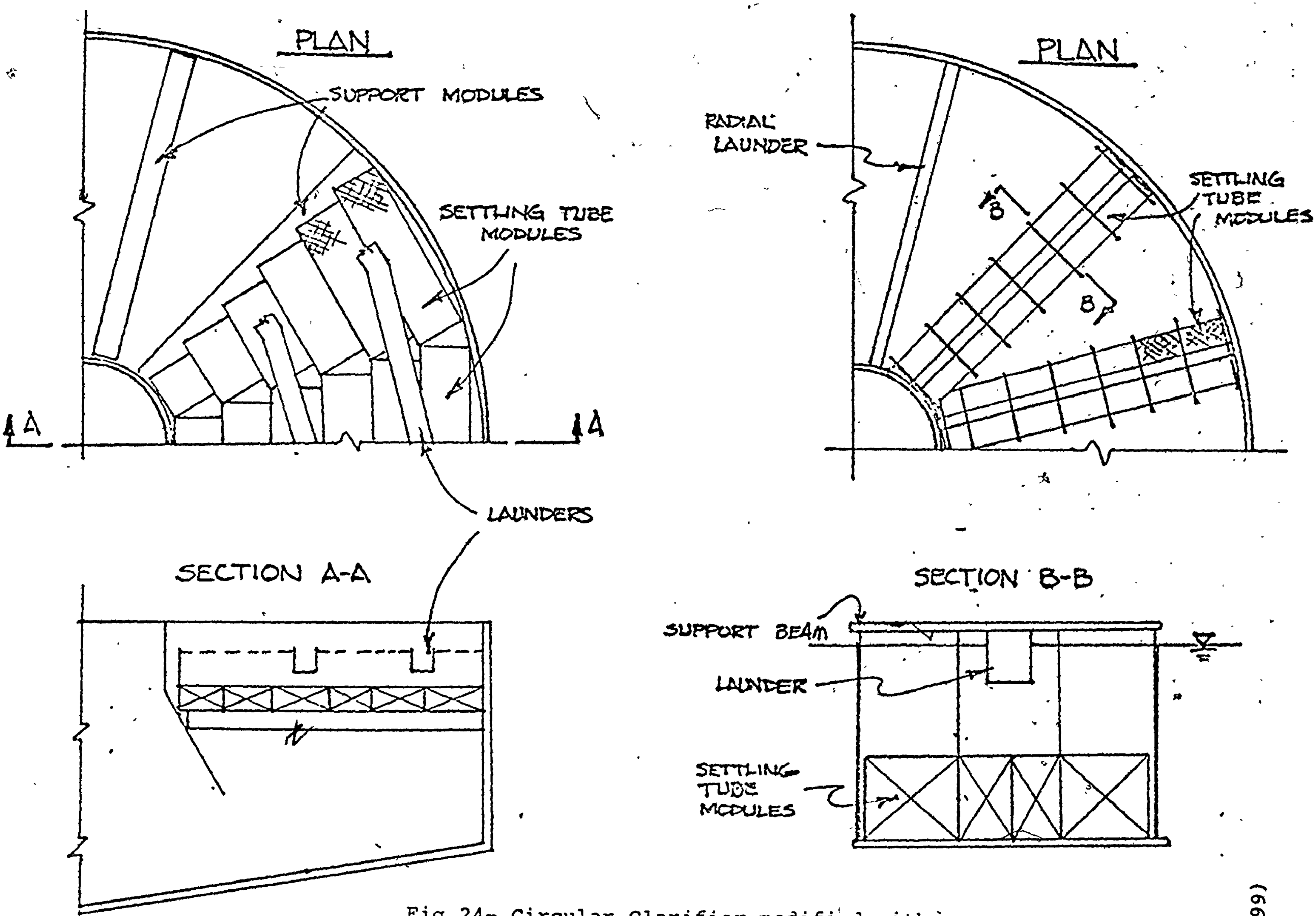

Fig.24- Circular Clarifier modified with Tube Modules (Neptune MicroFloc)

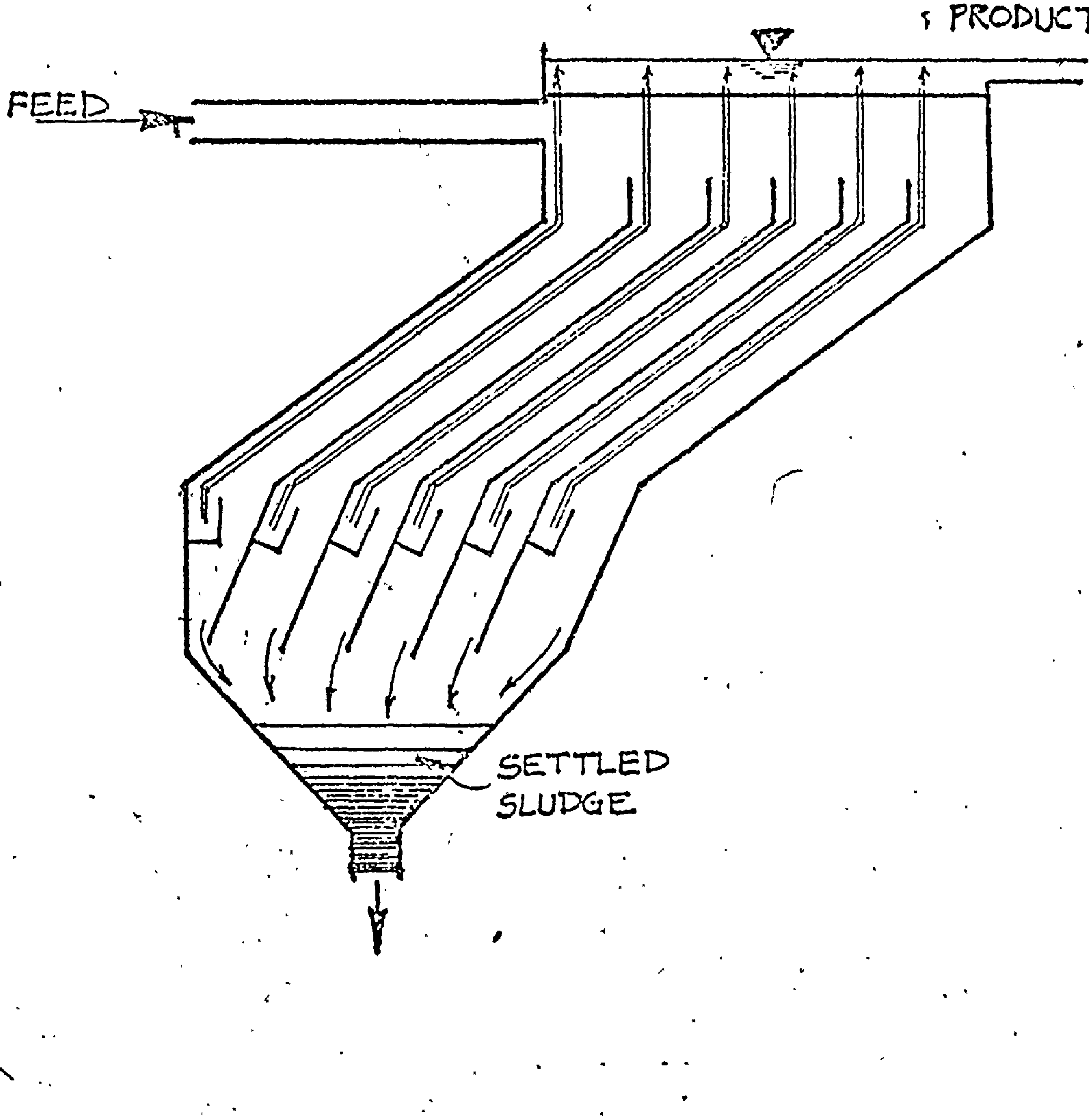

Fig.25- Lamella Settler

2.4.3 Full-Scale Evaluation of the Tube Settling Concept

A. Plant-Scale Experiences with Tube Settlers

As a direct result of encouraging pilot-scale tube settling results, commercially designed tube settlers have been applied at plant-scale in nearly all phases of wastewater clarification. Tube assemblies have been used in primary clarifiers, in aeration tanks, final clarifiers in the activated sludge and trickling filter processes, and in tertiary systems employing mixed-media filtration. As reported previously an even greater application has been experienced in the water treatment field. The performance of full-scale units has generally confirmed the results of pilot-scale investigations. Settling tubes used in the clarification of raw or trickling filter solids appear to be capable of either increasing the capacity of existing clarifiers or improving the performance of poorly operating units (see Table IV).

Primary Clarification: Slechta and Conley (67) confirmed the pilot study plant observation that tube length was not a significant factor in the removal of settleable matter from raw sewage over settling tube rates of 0.18 to 0.28 US gpm/sq.foot (see Table V).

TABLE IV

Partial List of Tube Settler Installations in Secondary Treatment Plants*

Name	Capacity U S MGD	Location
Wickham S.T.P.	0.30	Aliquippa,PA
Berryville S.T.P.	0.21	Berryville,VA
Oglesby S.T.P.	1.50	Oglesby, IL
East Windsor S.T.P.	1.00	East Windsor,NJ
Beaver S.T.P.	1.35	Beaver, PA
Beaverton S.T.P.	1.00	Beaverton, OR
Azalea Park S.T.P.	1.00	Orange County,FL
Upermoreland	6.00	Hatboro, PA
Watonga S.T.P.	0.50	Watonga, OK
Trenton S.T.P.	4.0	Trenton, MICH
Lebanon S.T.P.	1.1	Lebanon, Ohio
Winnipeg NEWPCC	72	Winnipeg, Canada

* Some of these plants may not be operational at present.

TABLE V

Effect of Tube Length on Suspended Solids Removal from Raw Wastewater*

Tube Length feet	Removal %
No Tubes	35
2	45
4	55

* Tube Rate = 0.25 US gpm/sq.ft (4320 gpd/sq.ft. of plan area).
Influent SS = 210 mg/l.

The initial problems encountered such as blockage of tubes with solids, were solved by weekly agitation of tubes and surface jet tube cleaning. Analysis of the type of material that clings on to the top edges of the tubes indicated 85% to be fibrous material such as paper. It seems therefore that different types of materials will affect performance of tube clarifiers somewhat differently. This was borne out by studies (plant scale) conducted by White and Baskerville (74). The data they obtained for the first six months was so random that no interpretation was possible. In fact there was no clear indication that the performance of the tube basin was any better than that of the control tank.They reported similar results [illegible] by other researchers in the U.K.

Unlike Slechta and Conley (67) who did not report temperature effects, White and Baskerville (74) using tracers showed that even a 0.2°C temperature difference between the influent and the bulk liquid in the tank tends to effect the performance of the tube basin. The raw sewage(or influent) would preferentially flow up the first few tubes, thus drastically reducing area and increasing the overflow rate. They make reference to tracer studies conducted by El-Baroudi and Fuller (25) who claimed that the total tube volume was never fully utilized under the conditions investigated due to the difficulty of distributing the flow uniformly under the tubes. Unfortunately White and Baskerville (74) did not quantify these differences.

In order to substantiate the results obtained, White and Baskerville (74) analysed the nature of the suspension with respect to settling velocity. The distribution of the settling velocities were so wide that even a large change in tube surface area would result in only a small change in the percentage of particles removed.

60% of the particles had settling velocities greater than 0.32 feet per hour. Theoretically, if the surface area was doubled, particles with settling velocities greater than 0.16 feet per hour should be captured. In the suspension under consideration only 69 per cent of the particles had settling velocities greater than 0.16 feet per hour. An insignificant increase in solids captured.

In summary, settling tubes in primary clarifiers can be used to improve quality or increase capacity. However, the actual value would depend on the tank hydraulics and the nature and type of suspension. A permanent solution to clean the tubes must be installed.

<u>Secondary Clarification</u> : A logical extension to the successful preliminary tests in separating biological solid from wastewaters was the evaluation of achieving this separation in an aeration basin directly, rather than in a separate secondary clarifier. Culp (12) summarised a study conducted by MicroFloc Research personnel. The turbulence and air bubbles in the aeration tank affected the performance of the tube settlers significantly. Therefore, the location and baffling to release the air bubbles prior to passing through the tubes, were found to be critical for this type of application.

Several observations showed that effluent from the conventional clarifier was superior to the supernatant of the SVI test, indicating the lengthy detention time (7 to 8 hrs.) in the clarifier was providing additional flocculation. On several occasions the tube effluent concentrations (detention time 40 mins) were comparable to the SVI supernatant concentrations (detention time 30 mins). The effluent concentrations at 40-100 mg/l, however, were considerably above those of the conventional clarifier (approximately 30 mg/l or less). Culp (12) concluded that (a) at reasonable SVI's (less than 150) the tube effluent (up to 100 mg/l) does not meet secondary requirements. However, this should not present any major problems for filtration indicating that the secondary clarifier could be eliminated; (b) Location of the tubes in the aerator affects performance significantly; and (c) Tube hydraulic loading should be less than 1.8 US gpm/sq.ft. of plan area. Above this, the system became less stable and gross carry-over of solids occurred more often.

The next step was to evaluate the capability of the tubes to separate biological solids in a conventional secondary clarifier. Slechta and Conley (67) reported the results of three plant scale evaluations. The plants were located in Wickham, Pennsylvania; Miami, Florida and Springfield, Illinois. In all three instances, tube assemblies (60°) were installed in existing rectangular clarifiers. The data from the Wickham installation indicate that the tubes produced a high quality effluent (less than 30 mg/l) over a wide range of operating conditions, i.e. SVI 38 to 103 and solids loadings 40 to 85 nd

0.19 US gpm/sq. ft. Initially, the plant experienced a periodic discharge of solids with an outside shape same as the tubes. The formation of a surface mat of activated sludge was overcome by use of a surface jet header. The differences in the mode of operation, possible differences in the hydraulic characteristics and biological conditioning between the three plants do not permit collation of data in one table. Miami, for instance, was a conventional activated sludge plant initially. The biological solids conditioning was excellent. This is apparent from the fact that, although the SVI was high (97 to 133), a low SS concentration in the supernatant of the SVI test samples occurred consistently (see Table VI). During this period, tube rates up to 0.17 US gpm/sq. ft. produced an excellent effluent quality. After three months of operation the mode of operation was changed to contact stabilization. A significant deterioration of the biological conditioning raised the SVI in excess of 150 consistently.

In addition, there was a significant quantity of dispersed solids (100 to 150 mg/l) in the supernatant. The overflow rate had to be reduced to 0.1 US gpm/sq.ft. When the flow was increased to 0.17 US MGD the effluent SS

TABLE VI

Operational Data from the Miami, STP
Slechta & Conley (67)

Total Plant Flow (mgd)	Activated Sludge Characteristics			Common Rate of	Control Clarifier Conditions			Clarifier with Tube Settler	
	MLSS (mg/l)	SVI	Super-natant SS (mg/l)	Return Sludge (%)	Overflow Rate (gpm/sq.ft)	Solids Loading (psf/day)	Effluent SS (mg/l)	Effluent SS (mg/l)	Tube Rate (gpm/ sq.ft.)
2.35	2,335	97	60	55	1.58	68	48	20	0.16
0.90	2,440	127	28	70	0.61	43	14	14	0.06
2.50	2,700	111	10	52	1.68	82	1,020	28	0.17
2.25	2,595	108	26	58	1.52	74	98	18	0.16
0.75	2,935	133	12	85	0.50	48	8	4	0.05
2.50	2,805	121	-	52	1.68	85	1,480	24	0.17
2.10	2,415	126	8	62	1.41	66	470	8	0.15
2.40	2,410	125	16	54	1.62	71	980	156	0.16
2.00	1,995	120	18	65	1.35	53	390	22	0.4

increased from 200 to 300 mg/l. Usually this significant increase in loss of solids takes place when the thickening limitation is exceeded. Slechta and Conley (67) reported of numerous hydraulic anomalies at the plant, in one area the tube rate was abonormally high (0.33 US gpm/sq.ft.), but they did not perform any tracer studies nor did they relate the data to thickening theory.

The Springfield clarifier (34 feet wide x 60 feet long) had two sets of tubes occupying 42 per cent of the basin. One stack of tubes consisted of 2 foot long and, the other, 4 foot long tubes. Andrew (1) reports that only during periods of very low SVI's (less than 75) or tube rates similar to the overflow rate of the unmodified basin (980 US gpd/sq.ft) was the effluent quality acceptable (less than 30 mg/l). At high SVI's $>$ 100 the tube clarifier produced a poorer effluent quality.

The report indicates that "tube" shaped blocks of sludge appeared on the surface occasionally. An interesting observation noted here is that when the flow to the clarifier was reduced to control the loss of solids in the effluent, the tank without tubes returned to normal much sooner than the one with tubes.

The recurrence of failure at high SVI values, (generally greater than 100) tends to substantiate two arguments. SVI is a good indicator of gross changes in settling characteristics and (2) the wide fluctuation in effluent quality, for instance 30 mg/l to greater than 200 mg/l (1) is typical of a shift from clarification to thickening dependency. In such situations an increase in clarification area alone will not suffice. A larger tank area must be provided for solids transport. Since this is not possible in the case of existing tanks, either the solids loading must be reduced to less than the limiting flux or chemicals be added to improve the settling characteristics.

Two of the more recent applications are at Trenton, Michigan and Lebanon, Ohio. The Trenton application is of special significance to this study due to the tank similarities. The tanks are circular, 65 feet in diameter and 10 feet SWD. One of the peculiarities at Trenton is the use of stacked 2 foot long tubes, with a plenum of nearly 1 foot between them, leaving no more than 2 to 3 feet clearance at the bottom. This is quite small for a tank which receives MLSS in excess of 5000 mg/l. The results shown in Table VII cover a period of 12 months (41). Tube rates of up to 10.7 US gpm per square foot of plan area produced an effluent with less than 30 mg/l SS. This is significant considering that the SVI's were nearly always greater than 75. The important difference here is that pickle liquor was added to the system just ahead of the aeration tank and polyelectrolyte (approx. 0.5 to 1.0 mg/l) ahead of the clarifier. This maintains more or less a

TABLE VII

(June 1975-May 1976)
Operational Data, From Trenton, Mich. Hennessey(41)

Period	Aeration Basin		Final Clarifier			
	MLSS (mg/l)	SVI	Overflow Rate** (gpm/ft^2)	Overflow Rate* (gpm/sq.ft)	Solids Loading** lbs/d/ft^2	Effl. SS (mg/l)
6/1(75)-6/30(75)	8054	111	0.26	0.020	60.05	17
7/1- - 7/30	7480	83	0.24	0.018	55.22	7
8/1 - 8/31	5161	71	0.27	0.021	40.60	4
9/1 - 9/30	5269	48	0.26	0.021	38.0	6
10/1-10/31	6070	91	0.19	0.015	33.9	5
11/1-11/30	6280	125	0.21	0.017	38.9	9
12/1-12/31	6294	100	0.27	0.021	44.1	11
1/1(76)-1/31(76)	6970	125	0.24	0.019	59.3	20
2/1 - 2/29	5890	71	0.39	0.031	63.8	35
3/1 - 3/31	5570	100	0.36	0.028	57.2	24
4/1 - 4/30	6849	91	0.28	0.022	51.0	15
5/1 - 5/31	5483	71	0.28	0.022	38.9	23

* was calculated based on a tube settling area of 41,472 sq.ft per tank.
** was calculated based on the tank settling area of 3248 sq.ft per tank.

constant rate of settling. This, coupled with a low tube rate may be the major reasons for the excellent results (see Table VIII).

The tube settlers in Lebanon were placed by the US EPA to evaluate their usefulness as a measure of upgrading the performance of hydraulically overloaded treatment plants. Even though the hydraulic and solids loadings are comparable with non-tubed conventional clarifiers, the effluent quality was far superior. This observation was borne out during the second phase of testing in Lebanon (54) (see Tables VIII and IX). More recent data (see Table X) from Lebanon indicate that up to 50 per cent more hydraulic loading (overflow rate) could be exerted on the clarifier with tubes and still produce an acceptable effluent quality.

/

Several operational problems became apparent during these studies. First of all, over a period of time solids tended to collect on the upper surface edges of the tube modules. A mat of solids several inches deep eventually formed, blocking off the tubes. This usually resulted in the appearance of masses of floating solids in

TABLE VIII

Operational Data from Lebanon, Ohio
(Feb.-May, 1973) - Oppelt (54).

Period	Aeration Basin		Clarifier with Tubes				Conventional Clarifier			Conv.Clarifier SS / Clarifier with Tubes SS
	MLSS	SVI	Overflow Rate* gpm/ft^2	Overflow Rate** gpm/ft^2	Solids Loading Rate** $lbs/ft^2/d$	Effl. SS mg/l	Overflow Rate ** gpm/ft^2	Solids Loading Rate** $lbs/ft^2/d$	Effluent SS mg/l	
2/27-3/8	2850	31.9	0.050	0.605	25.6	34.5	0.746	34.5	157.0	4.55
3/9-3/19	3560	31.1	0.062	0.744	36.8	18.6	0.751	38.9	98.6	5.30
3/20-3/26	3320	26.6	0.038	0.458	20.8	14.8	0.624	28.4	64.3	4.34
3/27-4/3	2830	32.9	0.041	0.488	19.6	33.5	0.539	24.6	43.3	1.30
4/4-4/11	3210	43.6	0.052	0.618	28.1	34.7	0.658	29.6	69.5	1.94
4/12-4/19	3410	32.5	0.052	0.621	29.9	44.0	0.686	31.9	83.9	1.90
4/20-2/27	2880	36.5	0.049	0.586	24.4	40.1	0.657	26.5	52.0	1.30
4/28-5/7	1910	31.2	0.050	0.602	16.8	34.9	0.612	17.3	51.5	1.45

* Based on tube settling area (see Table IX)
** Based on water surface area (see Table IX)

TABLE IX

Comparative Data
Lebanon STP Final Settling Basins
(Feb.1973 - May 1973)
Oppelt (54)

	East (Conventional Clarifier)	West (Clarifier With Tubes)
Flow Rate		
Nominal	(0.375 MGD)	(0.375 MGD)
Peak	(0.575 MGD)	(0.575 MGD)
Water Surface Area (less weir area)	(646.5 ft^2)	(455.5 ft^2)
Settling Area	(646.5 ft^2)	(5466 ft^2)
Surface Overflow Rate		
Nominal	580 gpd/ft^2	68.6 gpd/ft^2
Peak	890 gpd/ft^2	105.2 gpd/ft^2
Total Weir Length	(49 feet 8 in.)	(312ft. 0 in.)
Weir Loading Rate		
Nominal	(7,550 gpd/ft)	(1,202 gpd/ft)
Peak	(11,575 gpd/ft)	(1,843 gpd/ft)

TABLE X

Operational Data from Lebanon, Ohio (April-May 1976)
- Oppelt (54)

Period	Aeration Basin		Clarifier with Tubes				Conventional Clarifier			Tube Clarifier O.R / Conv.Clarifier O.R.***
	MLSS	SVI	Overflow Rate* gpm/ft^2	Overflow Rate** gpm/ft^2	Solids Loading Rate ** lbs/ft^2/d	Effl. SS mg/1	Overflow Rate ** gpm/ft^2	Solids Loading Rate ** lbs/ft^2/d	Effluent SS mg/1	
4/4-4/10	3036	177	0.057	0.679	36.5	15.0	0.479	27.55	20.0	1.42
4/11-4/17	3179	172	0.057	0.679	35.53	6.5	0.509	30.42	16.4	1.33
4/18-4/24	2761	187	0.045	0.540	25.55	19.4	0.340	17.14	23.1	1.59
4/25-5/1	2303	176	0.051	0.609	21.99	17.5	0.450	17.07	12.2	1.36
5/2-5/8	2400	182	0.048	0.570	28.48	32.8	0.479	25.31	23.5	1.18
5/9-5/15	2732	234	0.047	0.559	30.35	8.5	0.379	21.67	19.6	1.47
5/16-5/22	2722	252	0.031	0.371	25.11	9.0	0.359	21.2	21.3	1.03
5/23-5/29	2817	183	0.036	0.429	21.69	10.0	0.352	18.51	15.9	1.22

* Based on tube settling area (see Table IX)

** Based on water surface area (see Table IX)

*** O.R. = Overflow Rate

the settler, possible tube fouling, and eventually the discharge of sludge in the shape of the settling tubes, called "sausages." It was found that regular cleaning of the modules was required as frequently as once per week in some instances. Subsurface water jets or air diffusers, gentle module agitation, or periodic draining of the basin were found to be effective cleaning methods.

It was also found that anomalies in basin hydraulics can significantly affect basin performance. Because tube settlers operate on an upflow basis, it was necessary that the tube modules be preceded by a deep baffle to force normal basin flow first down from the influent zone and then up through the modules.

The preceding full scale experiences could be summarised in the following general terms :

a) Tube settlers improve the effluent quality quality and/or increase hydraulic capacity. (1) (41) (12) (54) (67).

b) The latter improvement is more noticeable during periods of good biological conditioning represented by low SVI values (less than 100) (1) (41) (54) (12) (67).

c) Hydraulic anomalies cause a non-uniform distribution of flow. This increases the tube rates grossly in some areas thus reducing the effective extended area. (67) (12) (1).

d) Solids tend to cling on to the surface of the tube modules and block off entire tubes. All reported instances either have installed or recommended the installation of some kind of tube cleaning device. This should be operated as frequently as required. This varies from once a week to once a month or so (67) (1) (12) (41) (54).

e) The biological conditioning in all of the plants show a wide fluctuation indicated by a variation in SVI (35 to greater than 150) (67). Although not reported, this may cause a change from clarification to thickening.

CHAPTER 3

3.0 EXPERIMENTAL FACILITIES, ANALYTICAL PROCEDURES AND DESIGN.

3.1 Experimental Facilities :

Settled sewage from the primary sedimentation tanks flow via a common conduit into four aeration tanks. Each aeration tank has four passes and the feed enters to passes 2 and 3. Pass number 1 is used only for re-aeration of the recycle sludge. The total detention time at 55 MGD is approximately 3 hrs. Coarse bubble air spargers introduce 1.5 cu. ft. of air per gallon of sewage.

The aerated sewage flows via an aerated conduit into the final clarifier. There are ten final clarifiers, each 80 feet square with centre feed and peripheral collection. The recycled sludge is withdrawn directly below the influent. The central baffle (18 foot square) reduces the downward flow to less than 0.1 fps at 14 MGD. The total surface area available for clarification is 5811 sq. ft., (see Appendix B.1 for further details).

A study conducted by Girling (34) in 1972, indicated that the flow split (at the same gate opening) to the Control Clarifier was 1 to 5 per cent greater than the Tube Clarifier, and the solids carry through to the Control Clarifier was significantly higher. This hydraulic anomaly was reduced by baffling. Due to the difficulty in monitoring the remaining potential difference, the Control Clarifier was kept out of service for the period of testing.

For the purpose of comparison between the tube clarifier and non-tube clarifier, historical plant data and performance of another clarifier, No.10 will be used.

The following table (Table XI) is a summary of the measured variables, frequency of sampling and the method of analysis.

3.2 Experimental Procedures :

TABLE XI
Summary of Measured Variables

Parameter	Frequency of Sampling	Method of Analysis
(A) Biological		
Solids retention time (θc)	Constant @ 10 days	Determined using ratio total mass of solids in the system/ rate of solids wasting. MLSS in each pass is determined by centrifugation, totalized, and 5% added for solids accumulated in the clarifier.
Food to Micro-organisms. Ratio (F/M)	Once a day, off a 24 hr. composite sample.	Determined daily using TOC and MLVSS values. TOC is determined using a Beckman Model 915. MLVSS was determined as per standard methods.

TABLE XI - continued

Parameter	Frequency of Sampling	Method of Analysis
D.O.	At two hourly intervals. The D.O. of each pass of the aeration tank is measured	Measured using a periodically calibrated D.O.meter. Measurements are usually taken in duplicate.
T.O.C.	A 24 hr. composite sample from the influent to the aeration tank, and four six hourly composite samples of the final effluent are collected for T.O.C. analysis	T.O.C. was measured using a Beckman 915 furnace coupled with a 215A infrared analyser. Each sample was analysed in duplicate.
B.O.D$_5$.	24 hr.composite samples are collected and prepared for analysis the following day	Analysed as per Standard Methods. Only the raw sewage and final effluent were monitored for BOD.
MLSS	One grab sample from each pass collected once every two hours	The operators centrifuge 10 mls. of each sample for a fixed period of time and speed.The laboratory provides a weekly calibration chart for the centrifuge. This encompases the differences which could occur in biological conditioning.

TABLE XI - continued

Parameter	Frequency of Sampling	Method of Analysis
SVI	2 Litre grab samples from pass No.4 were evaluated every 2 hours	A 2 litre sample from the No.4 pass was placed in a graduated, 5 inch diameter settleometer, stirred and permitted to settle in quiescence for 30 mins. The volume occupied per gram applied solids was then computed.
Zone Settling Velocity. (ZSV)	Measured weekly	To be determined for concentrations between influent to the clarifier and the underflow from the clarifier. The column dimensions are 5½ inch I.D., 5 ft. high. Initial mixing was done manually. The liquid solid interface was monitored until no further compaction occurred. Prior to taking readings the MLSS of each sample was determined by centrifugation, in duplicate.
SS	Four six hourly composites of the tube clarifier effluent were collected daily. Each sample was approx. 1 to 1½ litres in volume, and consisted of samples taken every 5 to 7 mins.	The samples were analysed as per Standard Methods. The samples were refrigerated at 4°C until analysis. Prior to analysis each sample was brought to room temperature. Periodical duplication of tests were carried out. The same balance was used throughout the test program to determine the weight of solids. It was calibrated at least once a month.

TABLE XI - continued

Parameter	Frequency of Sampling	Method of Analysis
Sludge blanket height	Monitored once every two hours	This was measured using a light source/receiver instrument which reads out the % of light absorbed as a millivolt signal. The unit was zeroed each time by dipping the probe into the clarified liquid on the surface.
(B) Hydraulic Influent to Aerator. ($\dot{Q}$)	Continuously measured and integrated.	Measured by a venturi and both indicated and integrated. The control valve set point is used to maintain the flow at any selected value between the upper and lower limits.
Return sludge rates. (Q_R)	Continuously measured and indicated	Measured by a magmeter and indicated. A flow control loop enables one to vary the percentage of return.
Sludge wasting. (Q_W)	The amount of sludge to be wasted was calculated daily based on a 10 day SRT. Wasting was more or less continuous.	Flow was measured by a magmeter and the solids concentration was measured by a sludge density meter.

TABLE XI - continued

Parameter	Frequency of Sampling	Method of Analysis
Tracer Studies	A tracer run was conducted for each set of operating conditions i.e. Q and Q_r. The duration of each run was approx. 120-180 mins. The dye was introduced in less than 1/50th of the theoretical detention time.	For any one particular run 20 mls of 20% Rhodamine WT was added at the centre of the tank where the turbulence was maximum. The overflow was continuously monitored using a Turner-111 fluorometer and a suitable recorder with a variable speed chart drive. The underflow was monitored by collecting periodical samples for analysis on a batch basis. The standard curve (peak height vs concentration was established on a batch basis. A uniform procedure was maintained in measuring, adding, mixing and analysing (see Appendix C for further details).

3.3 Experimental Design :

Two basic parameters responsible for a successful separation of biological solids are, (a) biological conditioning within the aeration tank, and (b) hydraulic rates and their effect on the clarifier performance. Biological conditioning, as previously reported, (7) (58) (56) is governed by many factors such as oxygen tension, food to microorganism ratio, concentration of polysaccharides, etc. Bisogni and Lawrence (7) related most of these variables to solids retention time θ_c. In order to design the experimental program, three variables were selected: (a) Solids retention time, (θ_c), (b) Overflow rate (Q), and (c) Return rate (Q_R).

Having elected that each run must encompass weekly variations in sewage characteristics a grid with three variables would have meant an extensive time frame. In order to limit this, it was decided to maintain θ_c constant. Previous studies (35) (58) indicate 10 days to be a reasonable choice for θ_c. The reduced grid included flows (Q), from 5.5 MGD to 11.5 MGD and return rate (Q_R) was varied between 10 and 80 per cent.

CHAPTER IV

4.0 RESULTS AND DISCUSSION

4.1 Introduction

Table XII is a summary of the averaged results obtained during the four month period of testing. Sixteen runs were concluded ranging from overflow rates normally experienced by the unmodified basins i.e. less than 1000 gpd/sq.ft. to nearly 2000 gpd/sq.ft. of clarifier surface area.

Run No. 1 was terminated after it was found that the central baffle did not extend below the bottom of the tubes. In order to ensure an initial downward flow, a two foot downward extension was added to the baffle. Run No.2 commenced after only two days of plant re-starting. In part these may be the reasons for the gross loss of solids in the effluent. Even though the SVI's were greater than 100 during these periods, similar conditions were met in December 1975 and flows up to 1495 gpd/sq.ft. were treated successfully (see Table I).

In general the results indicate that below an SVI of 75, the tubes are capable of treating flow up to 2000 gpd/sq. ft. of clarifier area, or 0.113 gpm/sq.ft. of tube

TABLE XII

SUMMARY OF "AVERAGED" RESULTS

Period		Clarifier Overflow Rate**		Tube Loading*	SVI	Return Rate**		Applied Solids Loading**	Results
Run No:	No: of Days	GPD/ sq.ft.	MGD.	gpm/sq.ft		GPD/ sq.ft.	(% of Q)	lbs/sq.ft/day	Effluent SS (mg/l)
1	1	1549	9.0	0.089	115	568	(37)	33.8	102
2	2	1540	9.0	0.089	108	1230	(80)	78.9	460
3	4	1325	7.7	0.077	100	1006	(76)	64.7	17.8
4	6	1300	7.6	0.075	83	619	(48)	41.9	10.1
5	4	1505	8.7	0.087	76	688	(46)	44.9	11.8
6	11	1779	10.3	0.102	46	688	(39)	54.5	9.8
7	5	1841	10.7	0.107	49	430	(23)	54.3	7.5
8	6	1829	10.6	0.106	46	172	(9.5)	44.7	16.6
9	9	1949	11.3	0.113	41	189	(9.7)	46.5	16.5

* Based on tube settling area (see Appendix A.4)
** Based on clarifier surface area

TABLE XII - Continued

SUMMARY OF "AVERAGED" RESULTS

Period		Clarifier Overflow Rate**		Tube Loading*	SVI	Return Rate**		Applied Solids Loading**	Results Effluent SS
Run No:	No: of Days	GPD/ sq.ft.	MGD.	gpm/sq.ft		GPD/ sq.ft.	(% of Q)	lbs/sq.ft/day	(mg/l)
10	4	1853	10.8	0.107	53	473	(25)	63.9	20.6
11	7	1352	7.9	0.078	58	473	(35)	44.3	25.9
12	7	1525	8.9	0.088	57	473	(31)	39.9	20.6
13	5	1480	8.6	0.086	57	473	(32)	41.0	14.9
14	2	964	5.6	0.055	74	473	(49)	28.7	34.3
15	5	1287	7.5	0.074	88	387	(30)	44.5	298
16	7	1222	7.1	0.070	87	305	(25)	35.0	94.7

* Based on tube settling area (see Appendix A.4)
** Based on clarifier surface area

settling area. Under these conditions, the effluent quality was excellent, less than 25 mg/l. When the SVI increases beyond 75 (upsets in biological conditioning) the overflow rates had to be reduced to those of the conventional clarifiers i.e. less than 1000 gpd/sq. ft. of clarifier area or less than 0.06 gpm/sq. ft. of tube settling area to avoid gross loss of solids. In plotting the measured (SVI) and computed (overflow rate and solids loading) variables against clarifier performance (effluent SS), a wide scatter of data points were obtained indicating no apparent correlation to exist. This is discussed in detail in the following section.

Settling column tests were performed only during runs 6, 8, 11 and 15. Based on observations made during SVI tests and from the results obtained, the limiting flux for each of these tests were grouped with other runs (see Table XIII). This assumption is considered reasonable on the basis that the limiting flux varied very little, 68 to 86 psf/day, during the period (runs 4 to 14) classified as clarification dependent.

4.2 Correlations

4.2.1 Effect of Overflow Rate (see Fig. 26).

The data points in Fig. 26 are classified into three

TABLE XIII

Summary of Calculations from Flux Plots

Run Nos.	SVI Range	Applied Solids Flux(Avg)	Limiting Flux	Maxm.Permissible Hydraulic Loading $(Q+Q_R)$		Max. Possible Overflow Rate(Q)		Applied Hydraulic Loading(Average)	
				Calc.from flux plots		Calc. from ZSV		Q	$Q+Q_R$
		psf/d	psf/d	MGD	MGD	MGD	MGD	MGD	MGD
4,5&6	45-80	48.9	67.7	19.7 @ 2000mg/1	15.8 @ 2500mg/1	16.2 @ 2000mg/1	13.2 @ 2500mg/1	8.9	12.90
7,8&9	20-50	46.6	79.2	23.0 @ 2000mg/1	18.4 @ 2500mg/1	22.4 @ 2000mg/1	18.7 @ 2500mg/1	10.9	12.40
10,11, 12&13	45-60	45.7	86.1	17.3 @ 2900mg/1	16.2 @ 3100mg/1	14.6 @ 2900mg/1	13.0 @ 3100mg/1	9.0	11.75
14,15 &16	70-95	37.8	34.3	10.0 @ 2000mg/1	8.0 @ 2500mg/1	12.8 @ 2000mg/1	10.7 @ 2500mg/1	6.7	8.95

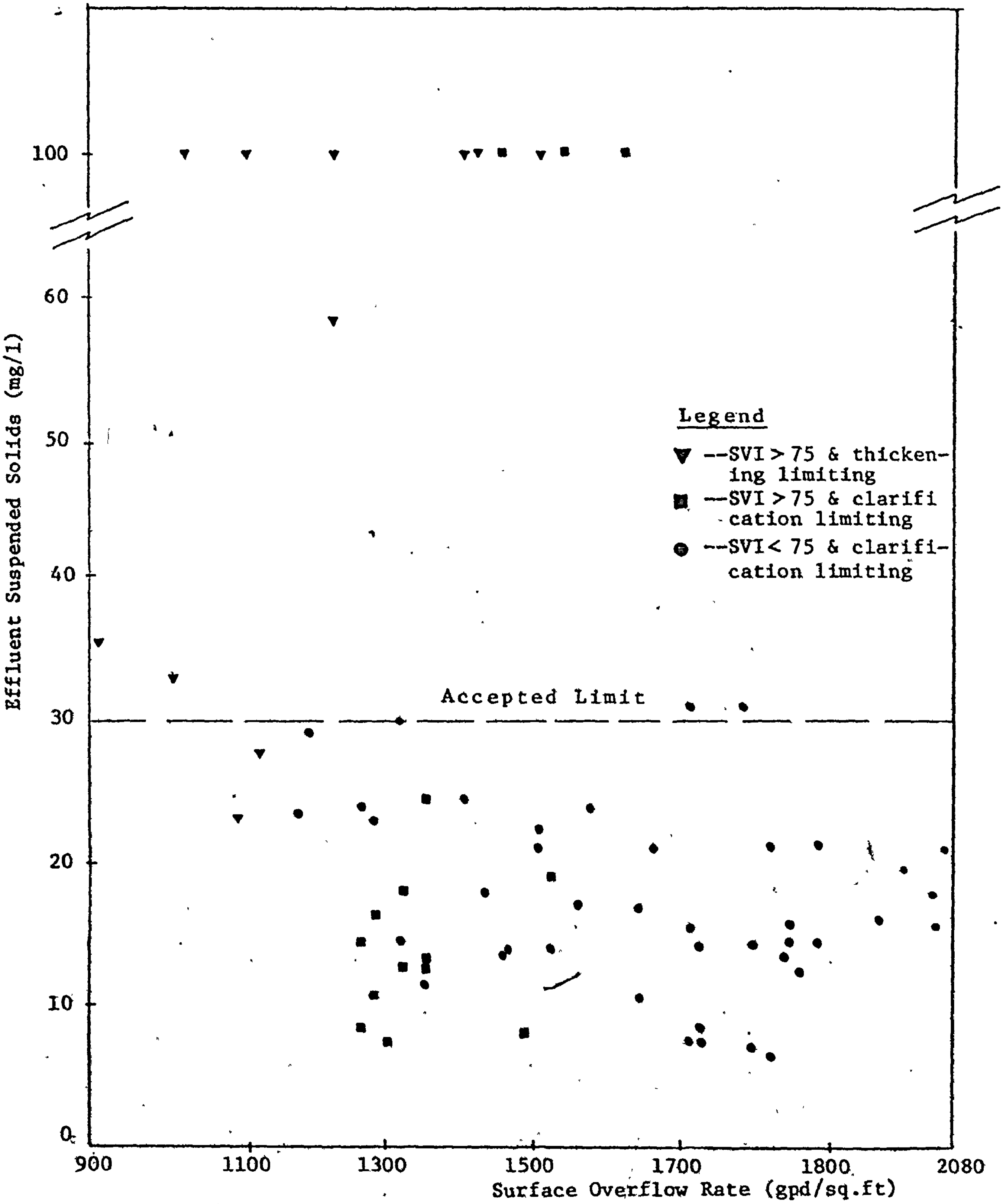

Fig.26- Effect of Overflow Rate on Clarification

broad categories -

a) <u>SVI < 75 with clarification limiting :</u> During this period the plant performance was unaffected by overflow rates up to 2000 gpd/sq.ft. of clarifier surface area or 0.116 gpm?sq.ft. of tube settling area. A similar conclusion was reported by Slechta and Conley (67).

At SVI's greater than 75 the performance of the clarifier can be divided into two categories -

b) <u>SVI > 75 with clarification limiting :</u> During runs 4 and 5 the zone settling velocity was 13 ft/hr (see Fig. 29) and the limiting solids flux 2.83 lbs/hr/sq.ft.* (see Fig.30). Computation indicates that clarification was the limiting factor during this period (see Table XIII). Overflow rates in excess of 1500 gpd/sq.ft. of clarifier surface area or 0.087 gpm/sq.ft. of tube settling area produced unacceptable effluent quality under these conditions.

c) <u>SVI > 75 with Thickening Limiting :</u> Runs 14, 15 and 16 were definitely in the thickening limiting regime (see Table XIII). The limiting solids flux had dropped to 1.83 lbs/hr/sq.ft. from 3.60 lbs/hr/sq.ft. for runs 10, 11, 12 and 13 (see Fig.30). Under these conditions overflow rates in excess of 1000 gpd/sq.ft. of clarifier surface

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* Solids loadings are based only on clarifier surface area.

area or 0.06 produced an unacceptable effluent quality.

No approved correlation exists between overflow rate and effluent SS. However, most failures, particularly at low overflow rates, result under thickening limitations.

4.2.2 Effects of Solids Loading (see Fig.27):

Once again no apparent correlation exists between solids loading and effluent suspended solids. During periods when SVI was less than 75, plant performance was independent of solids loading up to 60 lbs/day/sq.ft. Slechta and Conley (67) reported instances where solids loading up to 85 lbs/day/sq.ft. provided acceptable effluent quality.

Although this is nearly twice the recommended guideline for solids loadings in conventional clarifiers (63), the ability to handle the excess solids should not be related to the presence of tube settlers. Table XIII would indicate that applied solids loading was much smaller than the limiting solids loadings. Therefore, even a non-tubed clarifier would have handled this solids loading.

Nevertheless, the excellent effluent quality during this period can be related to the tube settler. In other words, the extended clarification area provided by the tubes

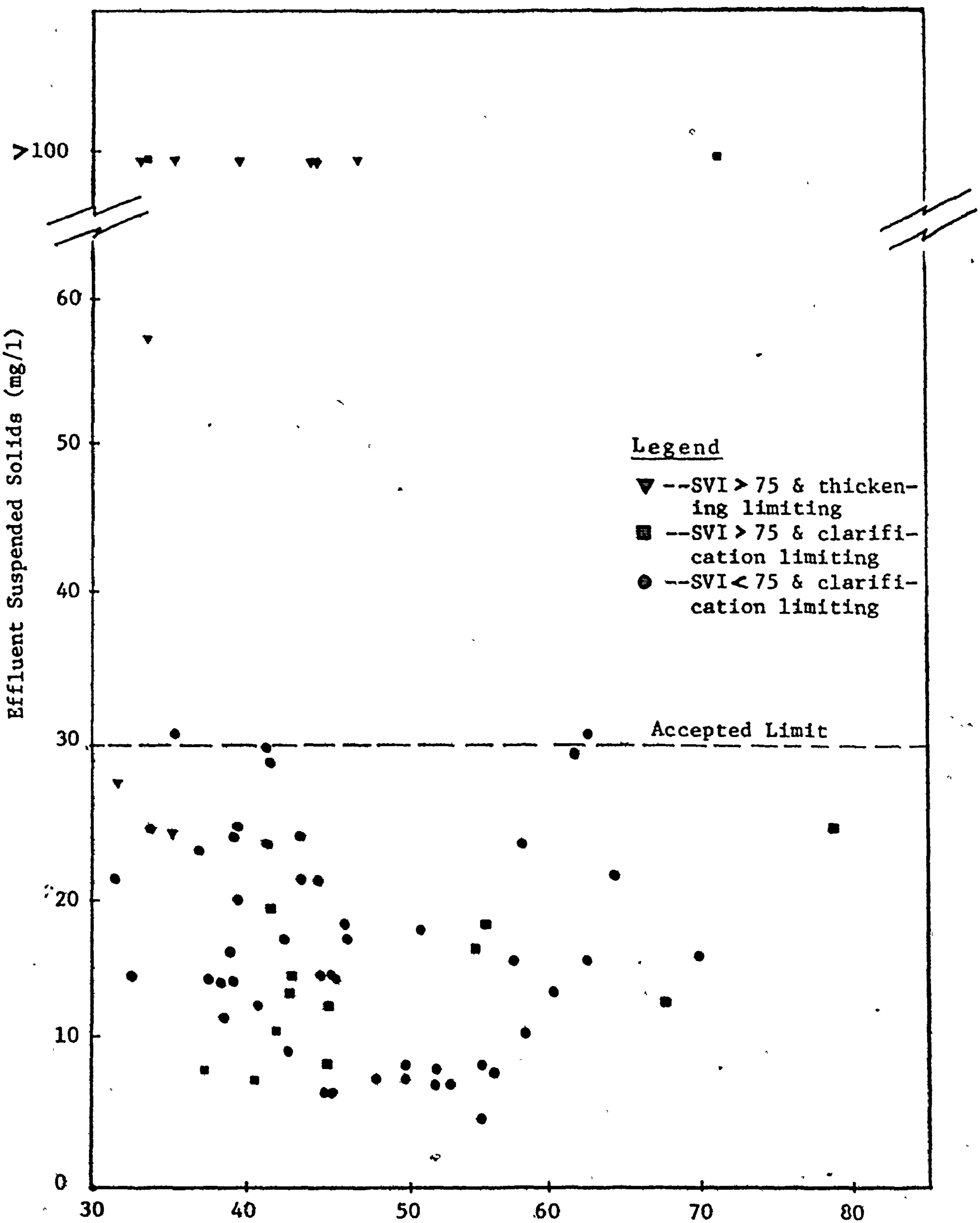


Fig.27 Effect of Solids Loading on Clarification.

(see Appendix A.4) further removed solids from the overflow. During runs 7, 8 and 9, the tube clarifier consistently produced an effluent quality of less than 15 mg/l. At the same period the average effluent from the non-tube clarifier was 28 mg/l, at an overflow rate of 1100 gpd/sq. ft. of clarifier surface area.

Similar results were reported by Oppelt (54) and Hennessey (41) where the tube clarifier consistently produced a better effluent quality when operating under the same conditions as the control clarifier (see Table VIII).

Previous reports (67) (54) (41) did not discriminate between clarification and thickening.

This is an important differentiation as the tube settlers do not provide area for solids transport. During thickening the limiting layer would act as a barrier below the tubes. Solids in excess of the limiting flux would build up on this layer, eventually pass through the tubes to overflow the thickener. Whatever solids settle in the tubes at this time would only worsen the situation and could even block off the tube.

Therefore when the process of solids separation is thickening limited solids loading to the clarifier must be reduced below the limiting flux. The tube clarifier and non-tube clarifier will then be equally loaded. As previously noted (54) (67) the tube clarifier would produce a superior effluent quality under these conditions.

4.2.3 Effect of SVI (See Fig. 28)

The noticeable scatter of the data points indicate that no firm relationship exists between SVI and effluent suspended solids. There is however, a distinct difference between plant performance for SVI less than 75 as against SVI greater than 75. Almost all instances when effluent SS exceeded 30 mg/l, SVI was greater than 75.

This does not mean that whenever SVI was greater than 75 the tube clarifier failed to produce acceptable effluent quality. Fig. 28 indicates many instances when the tube clarifier produced excellent effluent quality at SVI greater than 75. Slechta and Conley (67) report that on SVI as high as 135 produced an acceptable effluent. The recent data from Lebanon (see Table X) indicate that SVI's as high as 250 produced an effluent with less than 30 mg/l suspended solids. This was achieved at low overflow rates around 0.045 gpm/sq.ft. of tube settling area.

Andrew (1) reported that tube clarifiers are more easily affected by changes in SVI than conventional tanks.

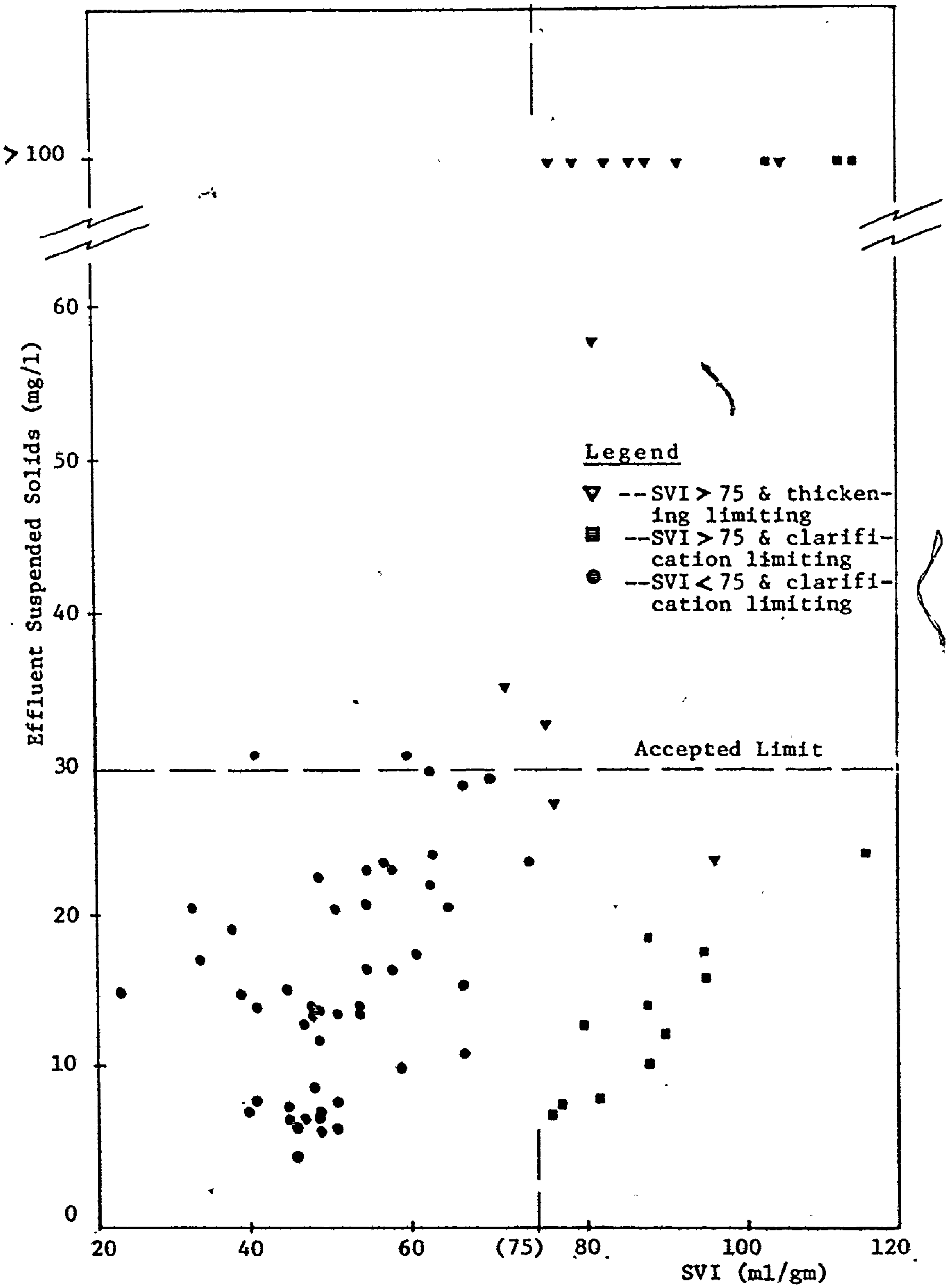


Fig. 28-Effect of SVI on Clarification

Therefore, it can be concluded that although SVI is not an accurate indicator of solids settling characteristics it does indicate gross changes in biological conditioning.

4.3 Variation in Plant Performance due to changes in Biological Conditioning.

The successful runs occurred during periods when the limiting component was clarification. The following data were extracted from Figs. 29, 30, and Table XIII.

Run Nos:	ZSV @ 3000 mg/l	Limiting Clarification Rate / Applied Hydraulic Rate
4,5 & 6	13.1 ft/hr	1.48
7,8 & 9	18.6 ft/hr	1.85
10,11,12 & 13	16.2 ft/hr	1.44

With reference to the above, if we extend a "Hydraulic factor of safety" of 1.5 to the limiting value, the tubes have in effect operated at the maximum permissible rate. This however, is based on the tank surface area. The tube settlers provide approximately 12 sq. feet of settling area per square foot of plan area (see Appendix A.4). As long as the settling characteristics maintain clarification as the limiting component, the tubes will permit a higher flow rate whilst still producing an acceptable effluent quality. After all, the tube settlers were designed to

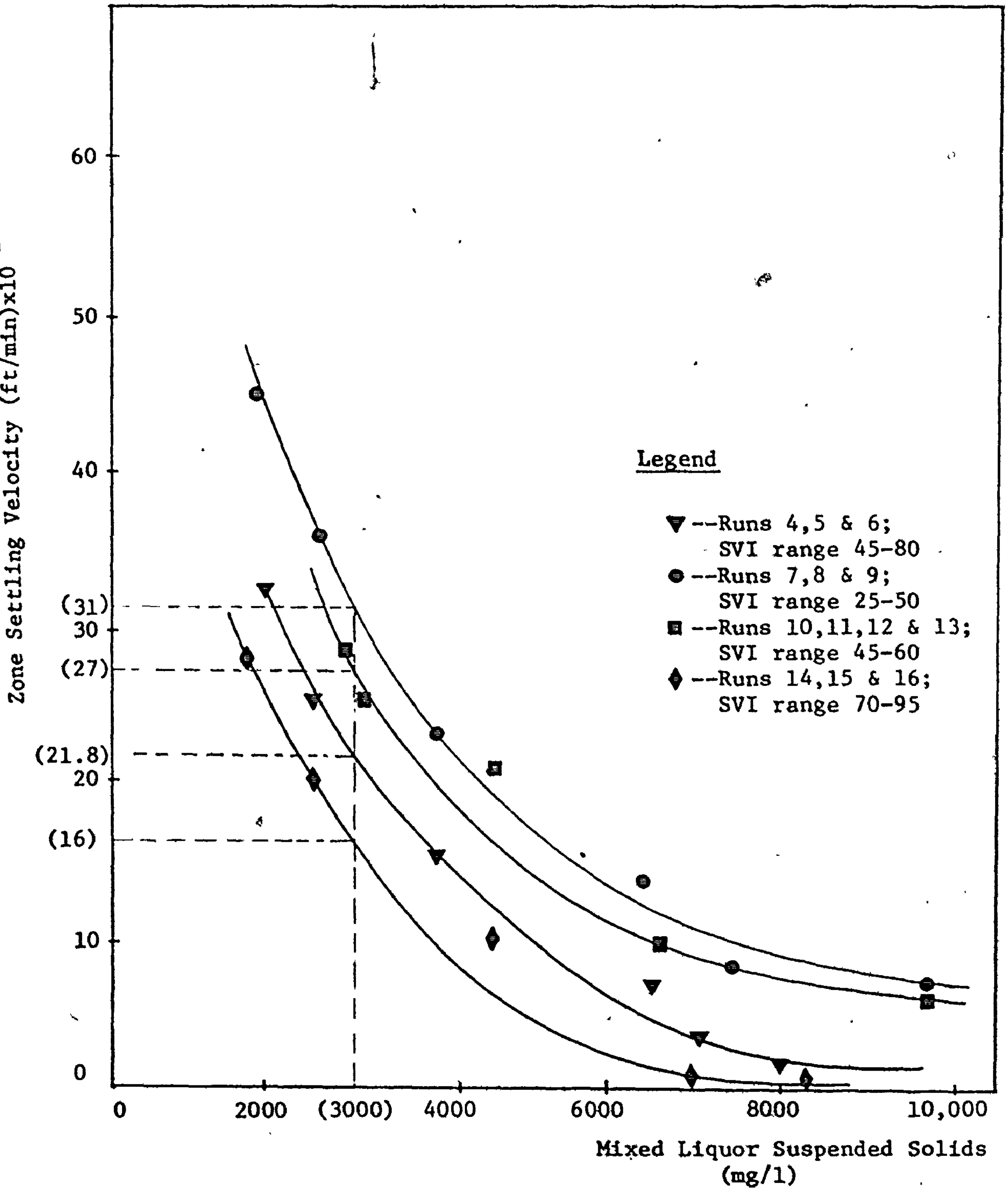


Fig.29 Zone Settling Velocity Vs Initial Suspended Solids Concentration.

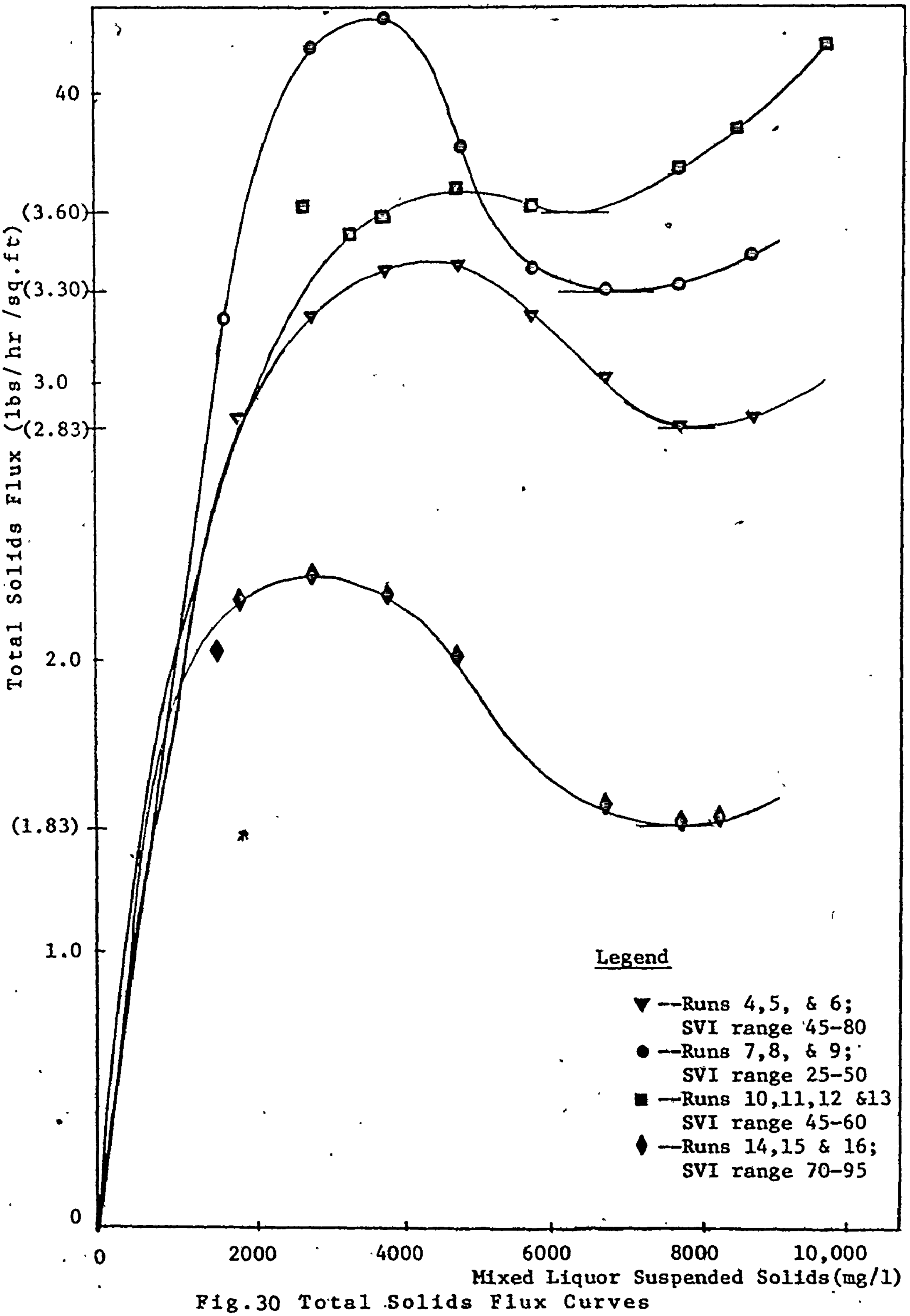


Fig.30 Total Solids Flux Curves

provide an extended clarification area per sq. foot of plan area.

Although the aeration tank and plant control conditions of D.O., F/M, SRT, etc. were not changed, a drastic change in the settling characteristics shifted the limiting component from clarification to thickening during runs 14, 15 and 16. This change caused an immediate deterioration in effluent quality.

Run No:	ZSV @ 3000 mg/l	Limiting Solids Transporting Rate / Applied Hydraulic Rate
14,15 & 16	9.6	0.9

In order to treat 10.0 MGD @ 25% return and 2500 mg/l MLSS under these conditions, an extra 3000 sq. feet of thickening area has to be provided. Previous workers (13) (14) (15) (31) (33) (54) (73) (78) failed to provide this analysis directly. However, they did realise that during "biologically upset periods" the tube rate must be reduced drastically. MicroFloc, for instance, in the design guidelines (69) state that "Chemicals should be fed to the influent of the secondary clarifier during those periods when the biological process is upset, thereby making the clarification process reliable nearly 100 per cent of the time." Slechta and Conley (67) reported that failure occurred at values of SVI's greater than 150. Although the zone settling velocities were not reported, the successful operation of the tube clarifier at Trenton, Michigan, over a wide SVI range could be due to the addition of chemicals (see Table VII).

At the Miami installation, it was reported (67) that when the plant operated in the activated sludge mode the settling characteristics were excellent and high tube loadings were possible. However, when the process was changed to contact stabilization, the biological conditioning deteriorated and loss of solids in the effluent caused a reduction in the applied tube loading. During initial periods of yet another experimental unit, the MLSS varied between 2800 and 3000 mg/l having an SVI of approximately 40. The solids loading on the tube basin was approximately 55 lbs/sq.ft/day whereas the control unit loading was 40 lbs/sq.ft/day. Both units produced essentially the same effluent quality, approximately 15 mg/l. After 3 weeks of operation there was severe loss of solids from the tube clarifier. Investigation of the settling characteristics indicated that although the SVI remained relatively low, no settling occurred during the first five minutes of settling as compared to a settled volume of 175 ml.

Therefore all these reported instances merely state that a greater understanding of the factors affecting biological conditioning would enhance the possibility of maintaining a fast settling floc. This would enable the plant to remain within the clarification limited regime and hence be in a position to effectively use the advantages offerred by the tube settlers.

4.4 Hydraulic Effects (Tracer Study Results)

Table XIV is a summary of the tracer study results. The traditional approach of trying to establish some relationship between tracer study results and solids removal efficiencies was not attempted here as the separation of biological solids in the secondary clarifier is a function of both aeration tank performance and clarifier hydraulics. They both tend to vary considerable as indicated by the wide variation in settling velocity and the serious underflow of short circuiting as shown in Fig. 31.

According to Wallace (72) close agreement between the results obtained from the same basin at different operating conditions is a requirement for a test to have any significance. Reasonably consistent results were obtained throughout the study (see Table XIV). The only shortcoming was the percentage of tracer recovered, which ranged from 80 to 150 per cent. Wallace (72) reports

TABLE XIV

Summary of Tracer Study Results

Overflow Rate(Q) GPD/sq.ft.	Return Rate(Q_R) (MGD)		t1/T	tp/T	tg/T	d Dispersion Coefficient	Is Index of Short Circuiting	%Tracer Recovery
	(MGD)	%of Q						
1377	4	50	0.31	0.51	0.83	0.12	0.38	80.3
1720	1	10	0.20	0.42	0.71	0.13	0.40	152
1720	1.5	15	0.21	0.50	0.79	0.23	0.37	140
1720	2.5	25	0.17	0.43	0.75	0.14	0.43	96.5
1720	5	50	0.19	0.45	0.88	0.18	0.48	98.3
1893	1.1	10	0.24	0.43	0.69	0.11	0.37	109
1893	2.75	25	0.26	0.44	0.75	0.11	0.41	95.6
1893	4	36	0.19	0.45	0.92	0.13	0.50	126

acceptable recoveries to be within 70 to 110 per cent.

The high recovery obtained may be attributed to any one of the following reasons :

a) The effluent tracer concentration was monitored using a fluorometer door that permitted continuous flow through the unit, whereas the standard curve was plotted using a different door that allowed batch samples to be introduced.

b) Wallace (72) recommended that the flow cell be maintained free of moisture. This particular precaution was not maintained during this study.

c) The fluorometer may not have a perfectly linear relationship for the entire range of concentrations tested. The standard curve drawn assuming a straight line relationship (see Appendix A).

The following table compares the average values of Heinke and Qazi (43) and the experimental results of Table XIV.

	Heinke & Qazi(43)		Exptl. Results
	Circ.Tank*	Rect.Tank**	Circular Tank
ti/T	0.10	0.32	0.22
tp/T	0.26	0.52	0.45
tg/T	0.36	0.74	0.79
I_s	0.30	0.27	0.42

* The tank dimensions are 120 ft.in diameter with side wall depth of 11 ft. The weir loading was 16,800 gpd/lin.ft.

** The tank dimensions are 135 ft.long by 30 ft.wide and 9 ft. deep.The weir loading was 12,300 gpd/lin.ft.

The ratios t_i/T and t_p/T for the clarifier at Winnipeg, seem to fall between the rectangular and circular tanks reported by Heinke and Qazi.

The meantime ratio t_g/T represents the best measure of effective basin volume (43) (53) (71). Therefore the value 0.79 obtained at Winnipeg indicates that nearly 80 per cent of the tank volume to be effective. Whereas the results reported by Heinke and Qazi (43) for the circular tank had an effective volume of only 36 per cent. A value less than unity indicates the existence of stagnant areas.

Murphy (53) introduced the index of short circuiting to measure the interaction between short circuiting, and dispersion together with their relationship to the skewness of the dispersion curve. The high value (0.42) obtained for the basin at Winnipeg, indicates an interaction between short circuiting and dispersion. Murphy (53) reported for basins with peripheral collection weirs the peak tracer concentration was attained when the first dye front reached the surface. He stated that a considerable portion of the tracer did not leave the basin but flowed radially across the surface to the centre, where it mixed with the incoming flow and formed a circular current.

Two striking features emered on analysis of the underflow dispersion curves (see Fig.31) :

1) There is a definite indication of severe short circuiting, probably due to the position of the outlet and the method of sludge withdrawal, and

2) The short circuiting gets progressively severe as the return rate is increased. This corresponds to an absolute value rather than a percentage of the influent.

The bulk of the tracer studies were conducted at the end of the programme, when the settling characteristics were poor. In addition, each set of conditions were maintained only for a part of the day, i.e. morning or afternoon. Unlike the previous observations as shown in Table XII, return rates in excess of 2.5 MGD resulted in a deterioration of the effluent quality. Conventionally, short circuiting in the overflow has been held responsible for clarifier failure. Recently however, researchers (46) (57) have devoted their efforts to trace the behaviour

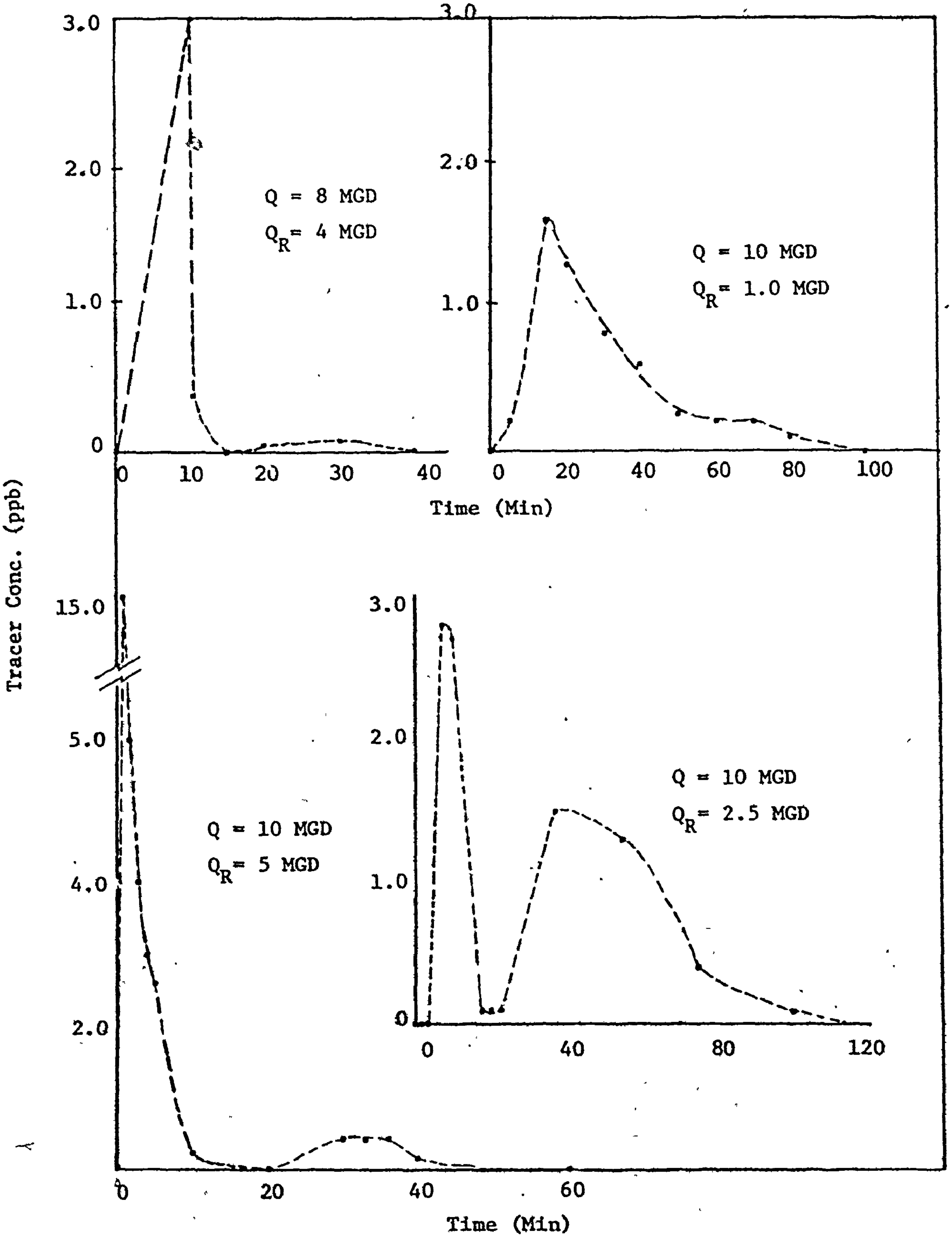


Fig.31-Underflow Tracer Dispersion Curves

pattern of the solids, with particular interest being paid to the underflow. The observation that clarifier failure could also occur due to short circuiting in the underflow is not entirely new. Katz (46) reported that return rates greater than 25 per cent significantly affect the solid settling pattern and eventually lead to loss of solids in the overflow.

This then poses a contradictory argument to equation 5, which states that an increase in Q_R will increase U and hence increase the solids handling capacity. This may well be true if the downward force exerted by sludge withdrawal is uniformly distributed across the tank.

It must be noted, however, that during runs 4, 5 and 6 the return rate was greater than 2.5 MGD and still an excellent effluent quality was attained. The applied solids flux, however, was considerably lower than the limiting flux (see Table XII).

CHAPTER V

5.0 CONCLUSIONS

a) When the solids separation process is clarification limiting, the tube settlers permit overflow rates up to 2000 gpd/sq.ft. of tank surface area or 0.113 gpm/sq.ft. of tube settling area at solids loadings up to 60 lbs/day/sq.ft. Based on historical data, this is 50 to 100 per cent greater than the capability of a non-tube basin.

b) When the solid separation process is thickening dependent, the tubes will not provide additional capacity but only improve the quality of effluent. Overflow rates in excess of 0.06 gpm/sq.ft. of tube settling area caused the effluent quality to deteriorate.

c) SVI is not an accurate indicator of settling characteristics but is a good indicator of gross changes in biological conditioning.

d) No apparent correlations seem to exist between any of the measured variables (i.e. overflow rate, solids loading and SVI) and effluent SS.

e) The effluent suspended solids begins to deteriorate as the return rate exceeds 2.5 MGD (430 gpd/sq.ft.) This is believed to be caused by serious short circuiting in the underflow.

CHAPTER VI

6.0 RECOMMENDATIONS

6.1 Application Guidelines

It is recommended that :

a) Prior to installation of the tube modules, a complete assessment of the secondary system must be completed in an attempt to determine whether clarification or thickening is limiting. The limiting flux and the maximum rate of underflow should be established.

Use historical data and column tests to evaluate sludge settling characteristics and potential thickening limitations.

b) On site studies with only partial coverage of the basin with tubes should be conducted first. However, the entire basin must be tested to the same limits with the tube section isolated and sampled separately. Local stress conditions will not provide an accurate evaluation for scale up. For instance, according to the results showing in Fig. 3, overflow rates in excess of 4000 gpd/sq.ft. of clarifier area, or 0.231 gpm/sq.ft. of tube settling area, provided acceptable effluent quality. Whereas, the capability of the full scale study fell far below these values, i.e. 0.113 gpm/sq.ft. of tube settling area when thickening is limiting.

c) The following overflow rates (based on tube settling area) could be used for the design of clarifiers with tube settlers :

Conditions	Recommended design overflow rates gpm/sq.ft. tube area
Conventional Activated Sludge with chemical addition.	0.15 **
Conventional Activated Sludge without chemical addition, & thickening limiting.	0.06 *
Conventional Activated Sludge without chemical addition, & clarification limiting.	0.12 **

* The solids loading should not exceed 40 lbs/day/sq.ft. of clarifier area.

** The solids loading should not exceed 60 lbs/day/sq.ft. of clarifier surface area.

APPENDIX A

A.1 EXPERIMENTAL DATA

TABLE A.1

PLANT OPERATIONAL DATA

RUN NO. 1

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp. $^{\circ}C$	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ ft^2	(ml/gm)	(mg/l)	days		(TOC/$_{MLVSS}$)	(ppm)
March 1 Monday	9.0	3.3	1549	1600	33.8	115	102	N/D	10	N/D	0.4 to 3
	A 2 FOOT DOWNWARD EXTENSION WAS ADDED TO THE CENTRAL BAFFLE (MARCH 2 - MARCH 6)										

TABLE A.1 - Contd.

RUN NO. 2

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp.	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days	^oC	(TOC/MLVSS)	(ppm)
March 8 Monday	9.4	7.2	1635	2500	71.2	104	480	N/D	10	0.3	N/D
March 9 Tuesday	8.5	7.2	1463	3200	86.2	113	440	N/D	11.5	0.2.	0.4 to 1.2
	REDUCED HYDRAULIC LOADING DUE TO EXCESSIVE LOSS OF SOLIDS										

TABLE A.1 - Contd

RUN NO. 3

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (Θ)	Temp.	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days	^oC	(TOC/MLVSS)	(ppm)
March 10 Wednesday	7.9	5.9	1360	3400	78.9	116	24.5	N/D	11.5	0.18	1.8 to 4.3
March 11 Thursday	7.7	5.9	1325	2900	67.7	91	12.75	N/D	11.5	0.17	1.8 to 4.0
March 12 Friday	7.5	5.9	1290	2400	55.1	96	16.25	N/D	11.5	0.2	0.9 tp 5.6
March 13 Saturday	7.7	5.9	1325	2400	55.9	99	18.00	N/D	12.0	0.32	1.0 to 3.0

RUN NO. 4

TABLE A.1 - Contd

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp. ^oC	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days		($^{TOC}/_{MLVSS}$)	(ppm)
March 15 Monday	7.9	3.6	1360	2300	45.5	83	12.30	N/D	10.0	0.32	2.3 to 6.4
March 16 Tuesday	7.5	3.6	1290	2200	42.0	89	10.74	8.6	11.5	0.35	N/D
March 17 Wednesday	7.4	3.6	1273	2400	45.4	83	8.25	13.0	11.0	0.33	0.5 to 4
March 18 Thursday	7.4	3.6	1273	2300	43.0	89	14.5	9.6	10.0	0.33	0.7 to 3.8
March 19 Friday	7.6	3.6	1308	2100	40.5	77	7.25	N/D	9.0	0.31	0.8 to 4.8
March 20 Saturday	7.5	3.6	1290	1800	34.4	78	N/D	10.8	6.5	0.22	1.2 to 7

TABLE A.1 - Contd

RUN NO. 5

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ) days	Temp. ^oC	F/M	D.O. in Pass No. 4 (ppm)
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)			(TOC/MLVSS)	
March 22 Monday	8.8	4	1514	1900	41.8	89	19.0	13.2	9.0	0.41	0.7 to 3.1
March 23 Tuesday	7.9	4	1360	2100	43.0	81	13.25	5.7	9.0	0.3	0.8 to 4.8
March 24 Wednesday	8.7	4	1497	1700	37.1	78	8.00	N/D	9.0	0.3	1.7 to 5.0
March 25 Thursday	9.6	4	1652	2500	58.5	58	10.5	N/D	N/D	0.28	0.5 to 5.2

RUN NO. 6

TABLE A.1 - Contd

Date	Q MGD	Q_R MGD	Overflow Rate GPD/ft^2	Solids Loading (mg/l)	 lbs/day/ft^2	SVI (ml/gm)	Effluent SS (mg/l)	S.R.T. (θ) days	Temp. ^oC	F/M (TOC/MLVSS)	D.O. in Pass No. 4 (ppm)
Mar.26 Friday	10.0	4	1720	2600	62.6	59	31.0	7.6	10.5	0.29	0.7 to 4.0
Mar.27 Sat.	10.1	4	1738	2300	55.8	40	8.25	13.7	9.5	0.29	1.6 to 5.3
Mar.28 Sunday	10.0	4	1720	2400	57.8	38	15.5	3.3	6.0	0.31	4.3 to 6.3
Mar.29 Monday	10.0	4	1720	2100	50.6	49	7.3	6.0	6.0	0.4	5.0 to 7.2
Mar.30 Tues.	10.5	4.0	1807	2100	52.4	44	7.0	8.5	5.0	0.39	3.0 to 7.5
Mar.31 Wed.	10.1	4.0	1738	2000	48.7	48	7.25	5.7	5.0	0.31	1.0 to 5.8
Apr.1 Thurs.	10.6	4	1824	2200	55.3	45	6.5	6.1	5.0	0.29	1.3 to 4.1
Apr.2 Friday	10.7	4.0	1831	2200	55.6	45	4.75	7.8	6.0	0.28	2.2 to 6.9
Apr.3 Sat.	10.8	4.0	1858	2100	53.5	46	7.0	4.6	6.0	0.26	2.9 to 4.9
Apr.4 Sunday	10.6	4	1824	2100	52.8	44	8.0	26.5	6.5	0.26	4.0 to 6.1
Apr.5 Monday	10.7	4	1841	2000	50.6	50	8.25	30.0	6.0	0.26	0.8 to 7.2

RUN NO. 7

TABLE A.1 - Contd.

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp.	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days	^oC	$(TOC/_{MLVSS})$	(ppm)
April 6 Tuesday	10.8	2.5	1858	2000	45.8	48	6.5	27.0	7.5	0.25	1.4 to 5.0
April 7 Wed.	10.6	2.5	1824	1900	42.8	47	9.25	44	8.0	0.26	0.8 to 4.2
April 8 Thursday	10.6	2.5	1824	2000	45.1	50	6.5	31.4	8.5	0.23	1.5 to 6.0
April 9 Friday	10.6	2.5	1824	2500	56.3	48	7.75	30	10.0	0.19	0.6 to 6.3
April 10 Saturday	10.7	2.5	1841	2500	56.8	53	N/D	7.7	8.5	0.2	1.5 to 5.8

RUN NO. 8

TABLE A.1 - Contd

Date	Q MGD	Q_R MGD	Overflow Rate GPD/ft^2	Solids Loading (mg/l)	Solids Loading lbs/day/ft^2	SVI (ml/gm)	Effluent SS (mg/l)	S.R.T. (θ) days	Temp. °C	F/M (TOC/MLVSS)	D.O. in Pass No. 4 (ppm)
April 11 Sunday	10.5	1.0	1807	2500	49.5	44	N/D	5.5	8.0	0.26	4.7 to 8.2
April 12 Monday	10.4	1.0	1790	1800	35.3	40	31.0	10.6	8.5	0.28	0.7 to 9.2
April 13 Tuesday	10.5	1	1807	1900	37.6	47	14.25	N/D	9.5	0.25	0.4 to 3.5
April 14 Wed.	10.9	1	1875	2000	40.9	48	12.25	N/D	10.0	0.25	1.5 to 5.6
April 15 Thursday	10.9	1	1875	2600	53.2	50	N/D	N/D	11.5	0.18	0.8 to 3.8
April 16 Friday	10.6	1	1824	2600	51.9	47	N/D	N/D	11.5	0.28	1.8 to 6.2

RUN NO. 9

TABLE A.1 - Contd

Date	Q MGD	Q_R MGD	Overflow Rate GPD/ft^2	Solids Loading (mg/l)	 lbs/day/ ft^2	SVI (ml/gm)	Effluent SS (mg/l)	S.R.T. (Θ) days	Temp. ^oC	F/M (TOC/MLVSS)	D.O. in Pass No. 4 (ppm)
April 17 Saturday	11.1	1.1	1910	2700	56.7	43	N/D	N/D	8.5	0.27	3.3 to 8.1
April 18 Sunday	11.5	1.1	1979	1800	39.0	66	16.0	6.1	N/D	0.41	3.1 to 8.0
April 19 Monday	11.7	1.1	2013	1800	39.6	37	19.75	4.5	7.5	0.49	1.4 to 6.8
April 20 Tuesday	12.0	1.1	2065	1400	31.6	32.0	21.0	N/D	9.5	0.63	2.3 to 6.0
April 21 Wed.	11.9	1.1	2048	2800	62.6	23	15.5	N/D	10.5	0.30	1.8 to 5.1
April 22 Thursday	11.9	1.1	2048	2300	51.4	33	17.75	19.6	10.0	0.34	1.6 to 5.8
April 23 Friday	10.8	1.1	1858	2200	45.0	40.0	14.5	25.4	10.0	0.33	0.8 to 6.2
April 24 Saturday	11.0	1.1	1893	2200	45.8	47	14.5	18.6	10.5	0.32	0.4 to 5.1
April 25 Sunday	10.1	1.1	1738	2400	45.9	44	14.25	34.9	10.5	0.28	0.7 to 4.8

RUN NO. 10

TABLE A.1 - Contd

Date	Q MGD	Q_R MGD	Overflow Rate GPD/ft^2	Solids Loading (mg/l)	 lbs/day/ft^2	SVI (ml/gm)	Effluent SS (mg/l)	S.R.T. (θ) days	Temp. ^{o}C	F/M ($TOC/_{MLVSS}$)	D.O. in Pass No. 4 (ppm)
April 26 Monday	10.7	2.75	1841	2600	60.2	46	13.25	29	10.0	0.3	0.6 to 5.8
April 27 Tuesday	10.8	2.75	1858	3000	69.9	44	15.75	15.2	10.5	0.25	0.7 to 6.4
April 28 Wed.	10.6	2.75	1824	2800	64.3	54	21.25	14.3	12.0	0.25	0.9 to 4.0
April 29 Thursday	11.0	2.75	1893	2600	61.5	69	29.75	9.6	12.0	0.26	N/D

RUN NO. 11

TABLE A.1 - Contd

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp. ^oC	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days		(TOC/MLVSS)	(ppm)
May 3 Monday	10.0	2.75	1720	2100	46.1	42	N/D	N/D	10.5	N/D	3.4
May 4 Tuesday	7.5	2.75	1290	2100	37.0	48.	23.0	N/D	12.0	0.26	1.9
May 5 Wed.	6.9	2.75	1187	2500	41.5	54	23.5	15.0	12.0	0.17	2.5
May 6 Thursday	7.0	2.75	1204	2500	41.9	66	29.0	10.2	12.0	0.19	1.6
May 7 Friday	7.4	2.75	1273	2500	43.7	74	24.0	7.5	12.0	0.2	1.9
May 8 Saturday	8.5	2.75	1462	3000	58.1	57	23.5	9.7	11.0	0.19	3.6
May 9 Sunday	7.7	2.75	1325	3000	41.4	62	30.0	25.0	11.0	0.23	5.2

RUN NO. 12

TABLE A.1 - Contd

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp. ^oC	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days		(TOC/MLVSS)	(ppm)
May 10 Monday	9.2	2.75	1583	1900	39.1	56	24.0	44.0	11.5	0.34	1.7
May 11 Tuesday	8.8	2.75	1514	1700	33.8	62	22.5	N/D	12.0	0.34	1.7
May 12 Wed.	8.2	2.75	1411	2100	39.6	62	24.5	12.0	13.0	0.29	1.7
May 13 Thursday	9.7	2.75	1669	2100	44.9	64	21.0	11.4	13.0	0.32	1.7
May 14 Friday	9.6	2.75	1652	2200	46.9	54	17.0	77	13.5	0.32	2.3
May 15 Saturday	8.8	2.75	1514	2200	43.7	50	21.0	N/D	13.0	0.33	5.0
May 16 Sunday	7.7	2.75	1325	1800	32.4	53	14.5	N/D	12.5	0.33	3.6

RUN NO. 13

TABLE A.1 - Contd

Date	Q MGD	Q_R MGD	Overflow Rate GPD/ft^2	Solids Loading (mg/l)	Solids Loading lbs/day/ft^2	SVI (ml/gm)	Effluent SS (mg/l)	S.R.T. (θ) days	Temp. °C	F/M (TOC/MLVSS)	D.O. in Pass No. 4 (ppm)
May 17 Monday	8.9	2.75	1531	1900	38.1	50	14.0	N/D	12.0	0.36	2.8
May 18 Tuesday	8.6	2.75	1479	2000	39.1	53	14.0	43.9	13.5	0.34	1.5
May 19 Wed.	9.1	2.75	1565	2100	42.8	57	17.0	N/D	14.5	0.22	1.3
May 20 Thursday	8.5	2.75	1462	2400	46.5	60	18.0	N/D	14.5	0.27	2.0
May 21 Friday	7.9	2.75	1359	2100	38.5	66	11.5	N/D	14.5	0.3	1.6

RUN NO. 14

TABLE A.1 - Contd

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T.	Temp.	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	(θ) days	°C	(TOC/MLVSS)	(ppm)
May 22 Saturday	5.3	2.75	912	2100	29.1	71	35.5	N/d	15.0	0.17	1.7
May 23 Sunday	5.9	2.75	1015	1900	28.3	76	33.0	N/d	14.0	N/D	3.2

RUN NO. 15

TABLE A.1 - Contd

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp.	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days	^oC	(TOC/MLVSS)	(ppm)
June 1 Tuesday	8.8	2.25	1514	2500	47.5	77	144	N/D	17.0	0.18	1.4
June 2 Wed.	8.2	2.25	1411	2500	44.9	80	462	14.4	17.0	0.16	1.1
June 3 Thursday	7.5	2.25	1290	2800	49.4	87	N/D	9.0	18.0	0.14	1.6
June 4 Friday	5.7	2.25		SAMPLER			MALFUNCTION				
June 5 Saturday	4.7	2.25		SAMPLER			MALFUNCTION				
June 6 Sunday	7.2	2.25	1239	2600	44.5	106	458	8.7	16.0	0.13	4.6

RUN NO. 16

TABLE A.1 - Contd

Date	Q	Q_R	Overflow Rate	Solids Loading		SVI	Effluent SS	S.R.T. (θ)	Temp.	F/M	D.O. in Pass No. 4
	MGD	MGD	GPD/ft^2	(mg/l)	lbs/day/ft^2	(ml/gm)	(mg/l)	days	^oC	(TOC/$_{MLVSS}$)	(ppm)
June 7 Monday	6.5	1.77	1118	2200	35.5	87	182	9.9	17.0	0.14	2.8
June 8 Tuesday	6.0	1.77	1032	2500	33.4	89	138	12.0	18.0	0.18	2.9
June 9 Wed.	8.3	1.77	1428	2300	39.8	93	125	10.0	18.0	0.29	3.6
June 10 Thursday	8.5	1.77	1462	1900	33.6	84	167	13.4	17.0	0.35	5.3
June 11 Friday	7.2	1.77	1239	2200	33.9	82	58	11.9	16.5	0.21	3.2
June 12 Saturday	6.4	1.77	1101	2500	35.1	97	24.21	12.5	17.0	0.19	4.0
June 13 Sunday	6.6	1.77	1135	2200	31.7	77	27.75	9.6	15.5	0.2	4.5

TABLE A II

TRACER STUDY RESULTS

Time	C1	C2	C3	C4	C5	C6	C7	C8
(mins)	(ppb)							
0	*	*	*	*	*	*	*	*
5	*	*	*	*	*	*	*	*
10	*	*	*	0.4	0.3	*	*	0.45
20	1.0	4.0	4.8	4.4	5.4	3.0	5.1	6.0
21.0	*	*	*	*	*	*	5.2	*
22.5	*	*	*	5.0	*	*	*	*
23.5	*	*	*	*	*	4.8	*	*
25	3.3	5.6	7.8	4.6	5.0	4.6	4.4	4.5
25.4	*	5.7	*	*	*	*	*	*
26.5	*	*	8.4	*	*	*	*	*
28	5.0	*	*	*	*	*	*	*
30	4.4	5.6	7.8	3.6	3.6	3.7	3.6	4.0
35	3.3	4.5	5.2	3.4	2.6	3.4	3.2	3.4
40	3.0	4.3	3.3	2.4	2.2	3.2	2.6	2.9
45	2.4	*	2.6	1.8	1.95	3.2	2.2	2.4
50	2.4	3.7	2.0	1.6	1.6	2.9	1.6	2.1
55	*	*	*	1.5	1.2	2.5	1.5	2.0
60	2.3	3.0	1.7	1.5	1.1	1.9	1.2	1.6
65	*	*	*	1.2	0.8	1.5	1.0	1.5
70	2.0	2.4	1.6	1.1	0.75	1.1	0.8	1.3
75	*	*	*	0.9	0.75	*	0.8	1.1
80	1.3	1.6	1.1	0.75	0.4	0.5	0.5	0.9
85	*	*	*	0.75	*	*	0.4	*
90	1.1	1.5	1.0	0.6	*	0.2	0.4	0.7

TABLE A II - Contd

Time	C1	C2	C3	C4	C5	C6	C7	C8
(mins)	(p p b)							
95	*	*	*	0.6	*	*	0.2	*
100	0.45	1.0	0.8	0.4	0.4	0.1	*	0.5
110	0.25	0.7	0.6	0.4	*	*	*	0.4
120	0.25	0.5	0.55	0.2	0.4	*	*	0.2
130	0.20	*	0.4	*	*	*	*	0.2
140	0.20	*	0.4	*	*	*	*	*
150	0.20	*	0.4	*	*	*	*	*
160	*	*	0.4	*	0.4	*	*	*
170	*	*	0.4	*	*	*	*	*
180	*	*	0.4	*	*	*	*	*
190	*	*	0.4	*	*	*	*	*
200	*	*	*	*	*	*	*	*

C1 – Q: 8 MGD & Q_R: 4 MGD
C2 – Q:10 MGD & Q_R: 1 MGD
C3 – Q:10 MGD & Q_R:1.5MGD
C4 – Q:10 MGD & Q_R:2.5MGD

C5 – Q: 10 MGD & Q_R: 5 MGD
C6 – Q: 11 MGD & Q_R: 1.1 MGD
C7 – Q: 11 MGD & Q_R: 2.75 MGD
C8 – Q: 11 MGD & Q_R: 4.0 MGD

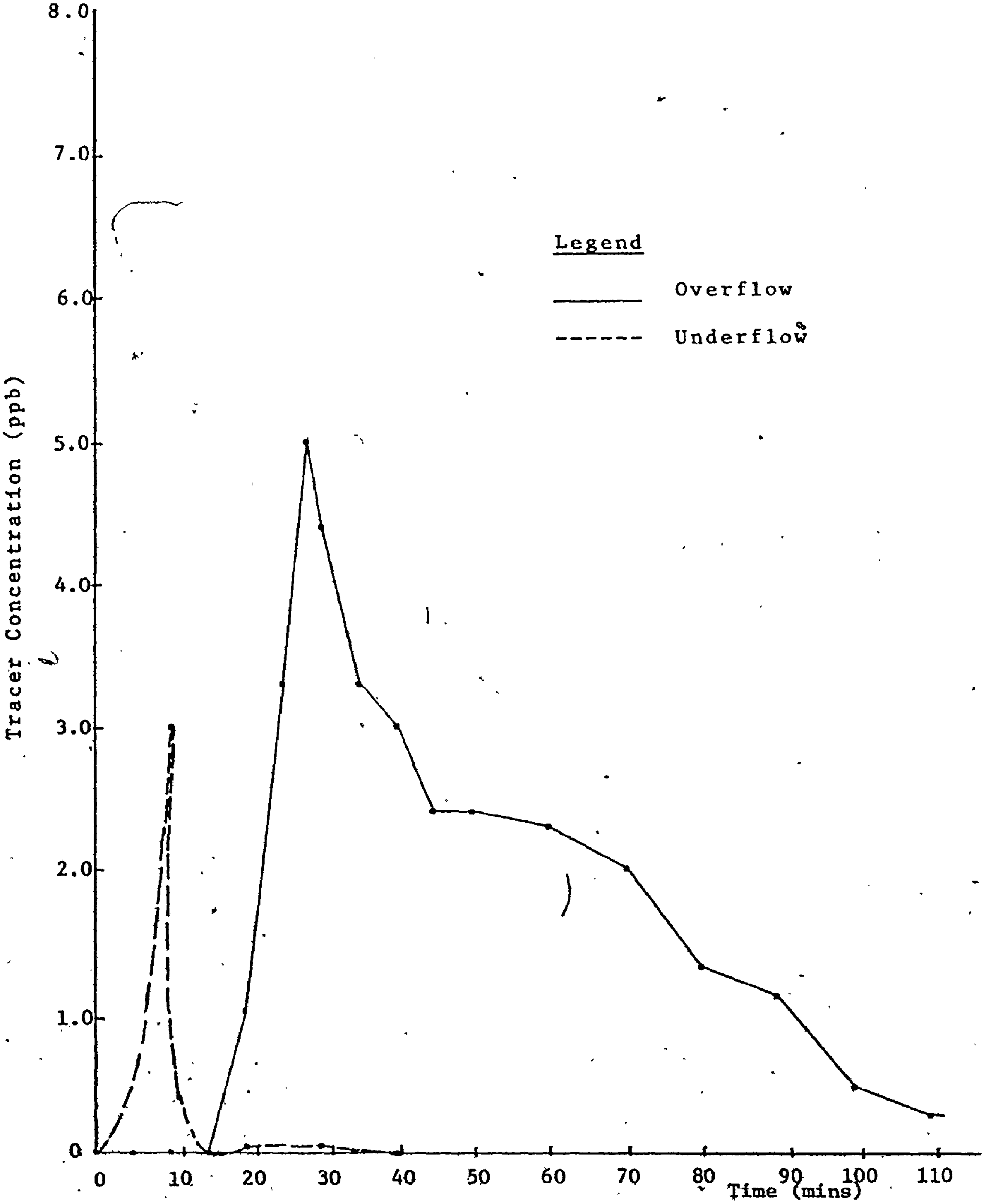


Fig.A.1 - Tracer Dispersion Curve for Q:8 MGD &Q_R;4MGD

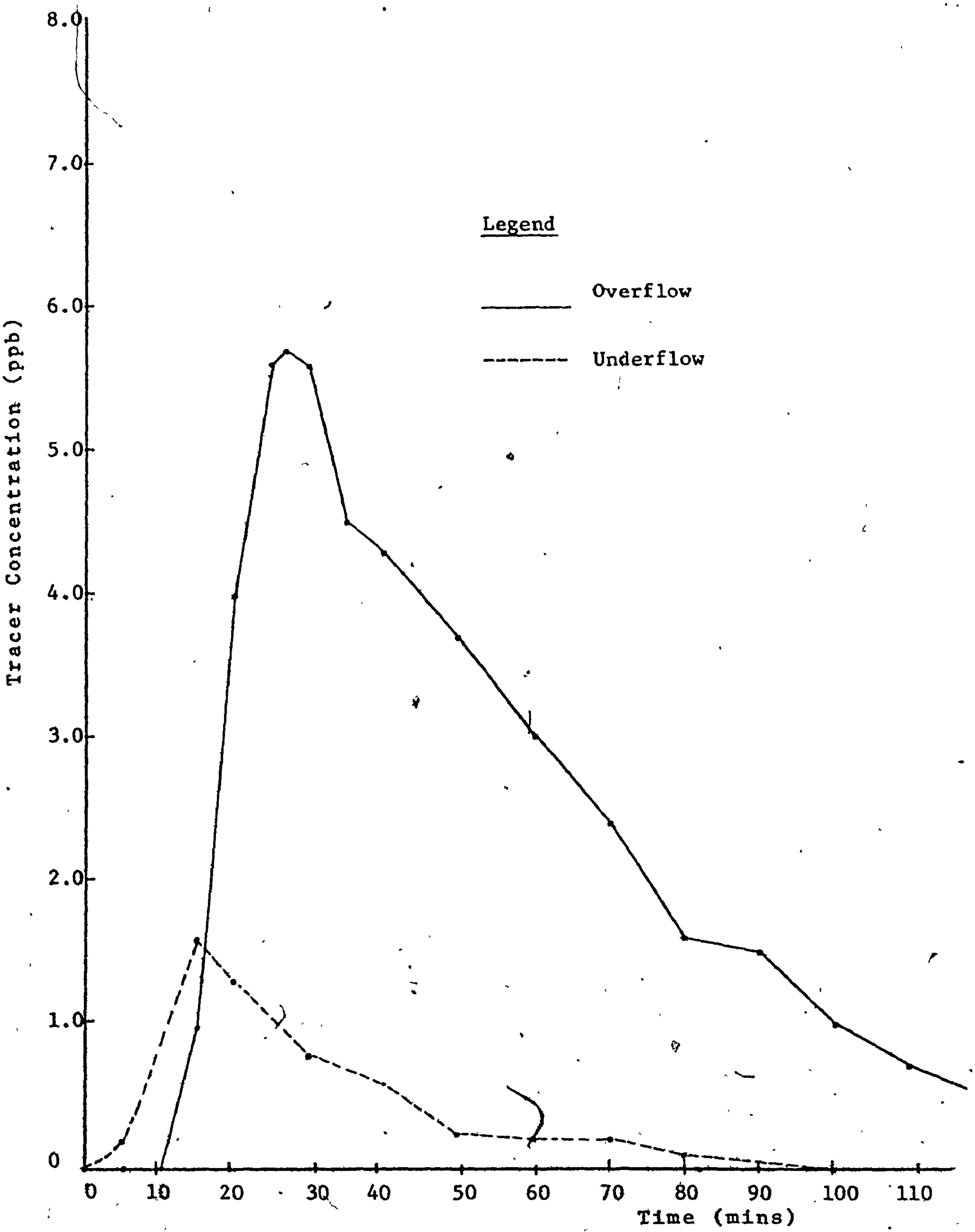


Fig.A.2 - Tracer Dispersion Curve for Q:10 MGD & Q_R:1MGD

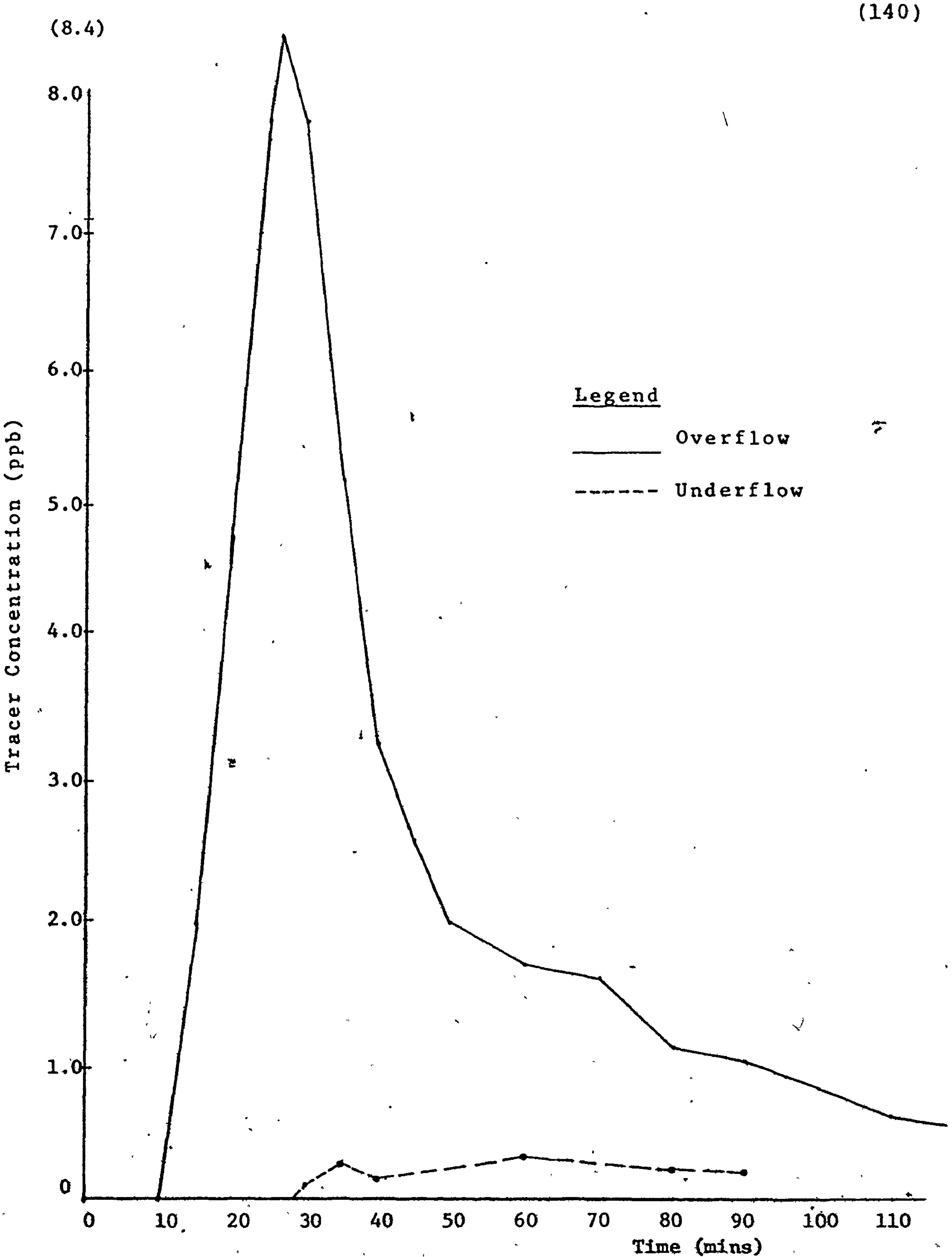


Fig.A.3 - Tracer Dispersion Curve for Q:10 MGD & Q_R:1.5 MGD

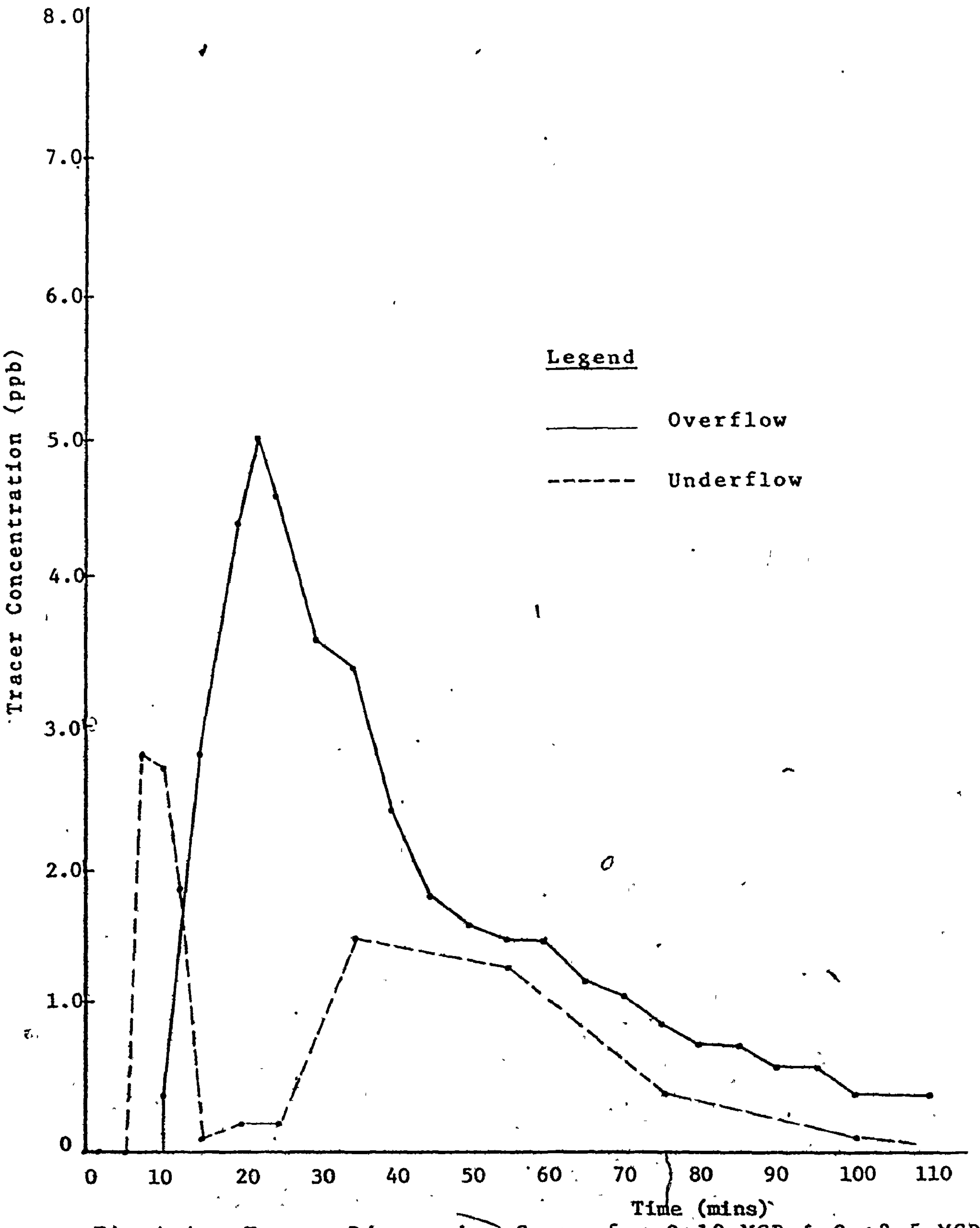


Fig.A.4 - Tracer Dispersion Curve for Q:10 MGD & Q_R:2.5 MGD

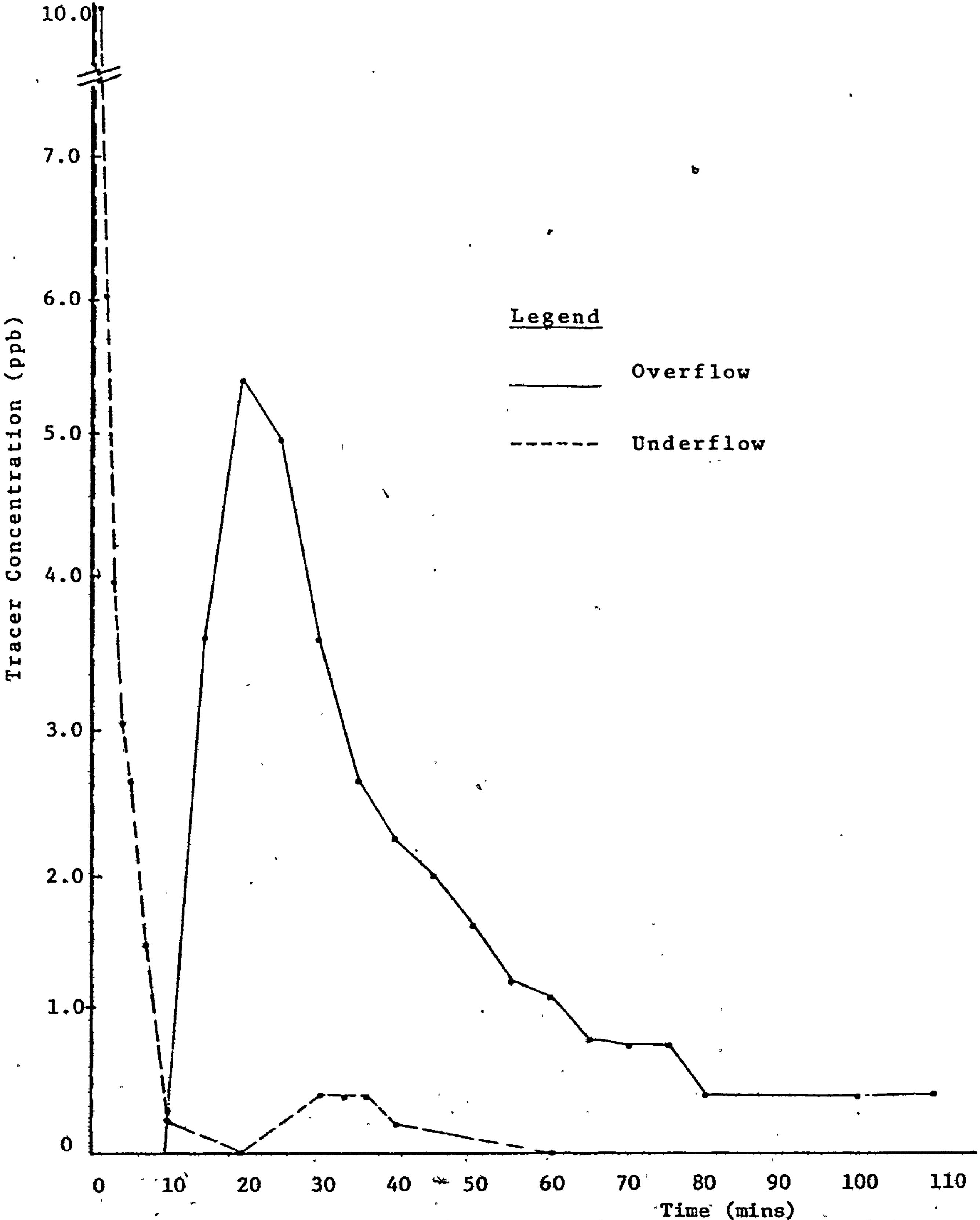


Fig.A.5- Tracer Dispersion Curve for Q:10 MGD & Q_R:5 MGD

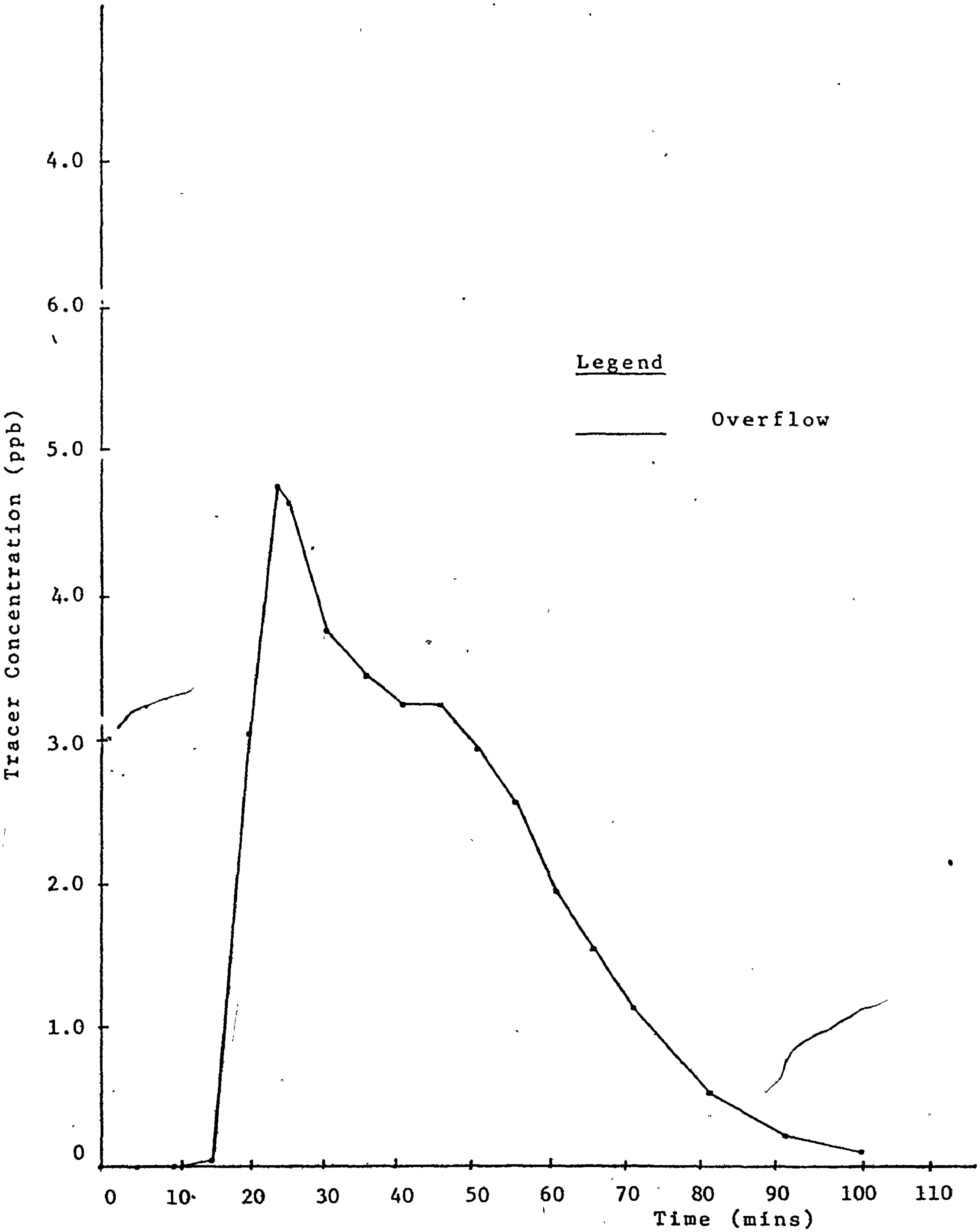


Fig.A.6- Tracer Dispersion Curve for Q:11 MGD & Q_R: 1.1 MGD

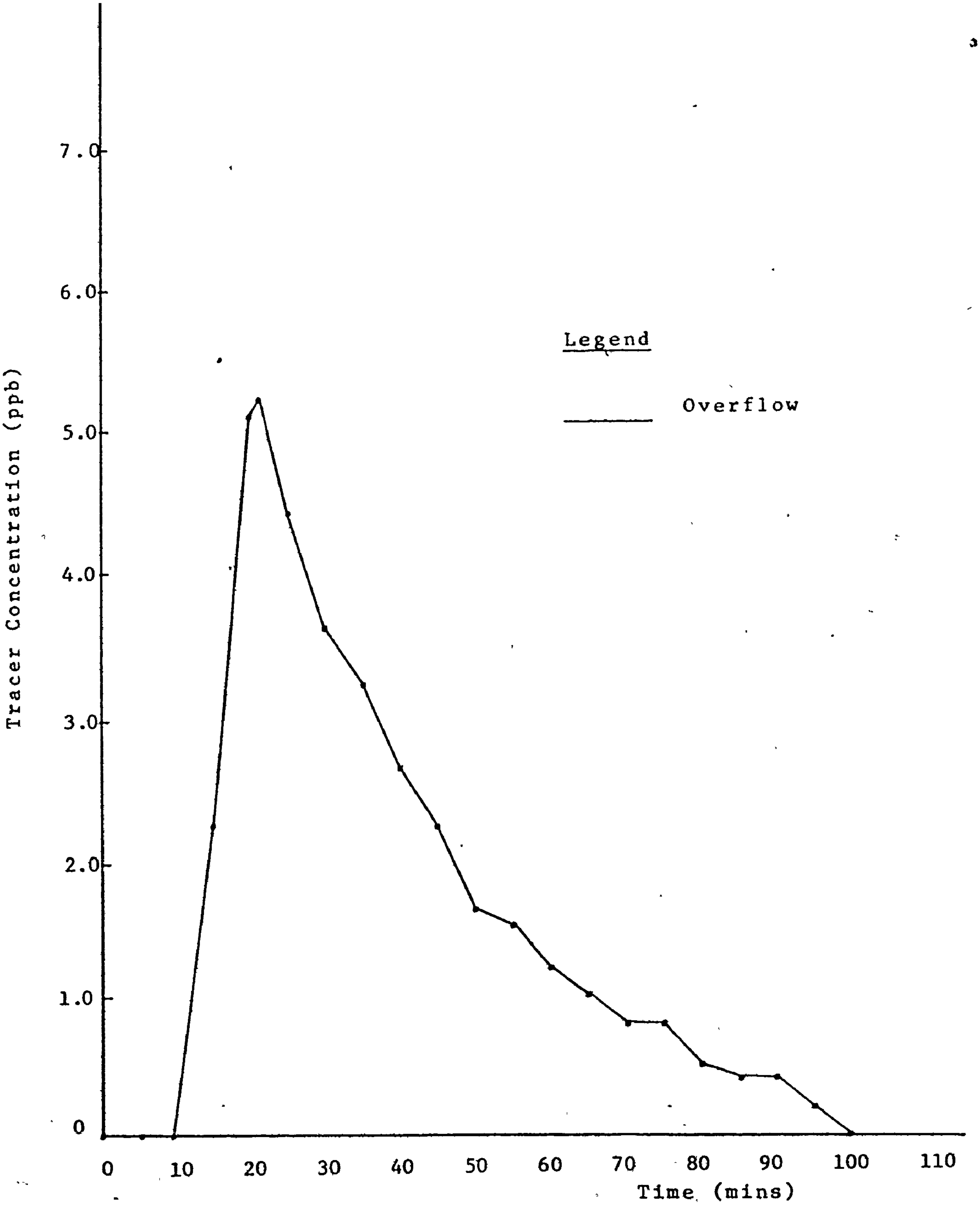
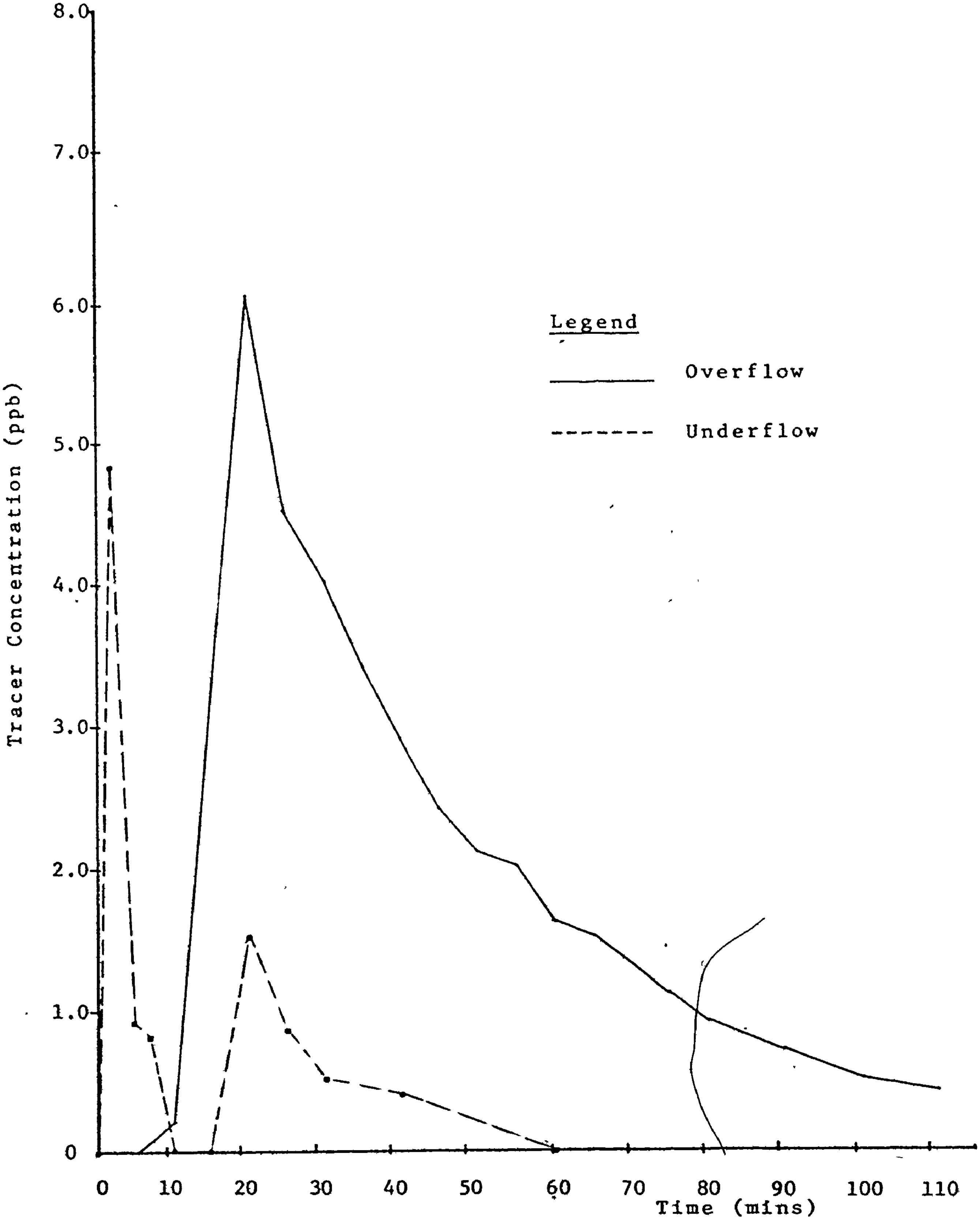


Fig.A.7- Tracer Dispersion Curve for Q:11 MGD & Q_R:2.75 MGD



A.8- Tracer Dispersion Curve for Q:11 MGD & Q_R:4.0 MGD

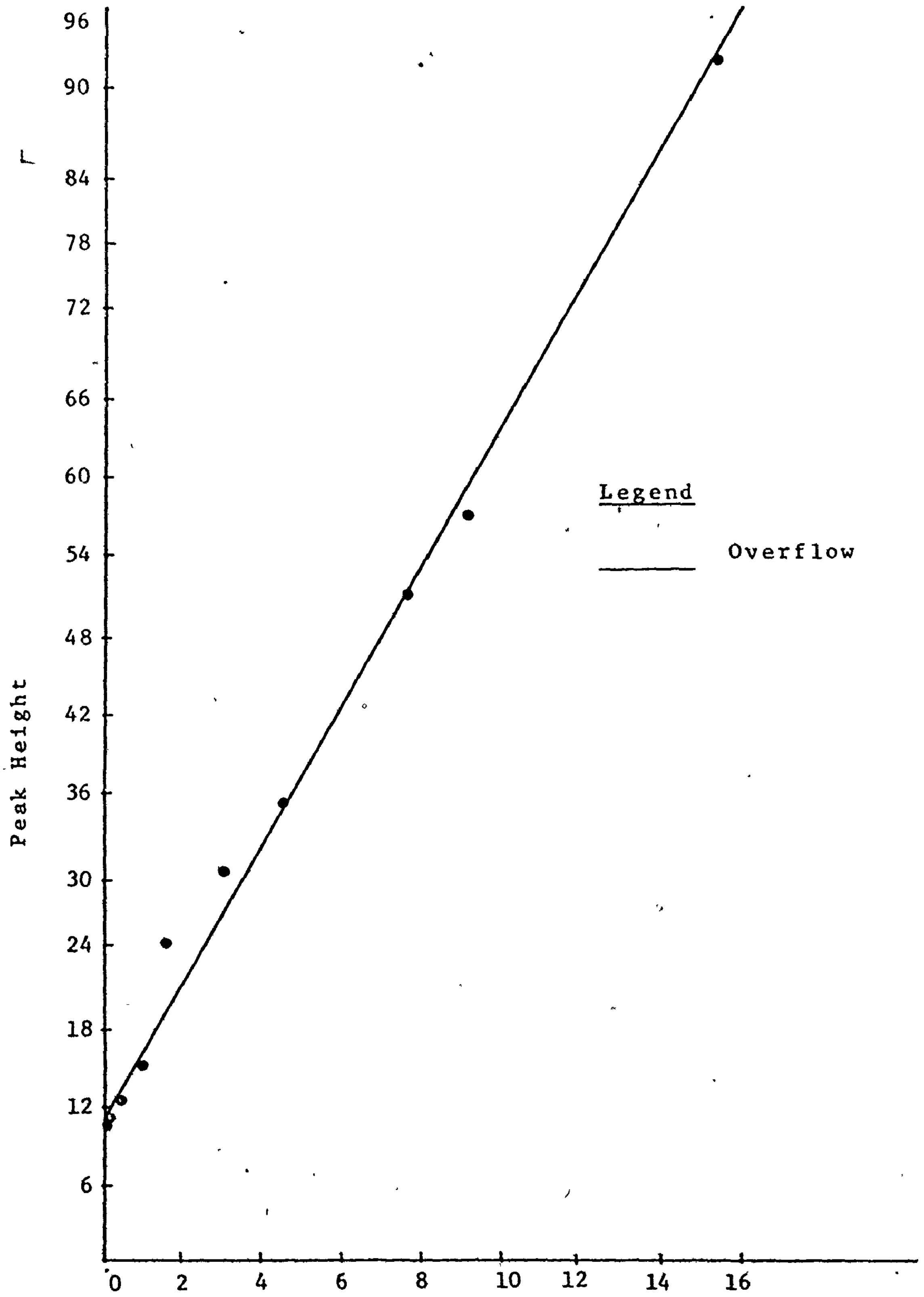


Fig.A.9- Standard Curve

TABLE A III

Formulae Used in Computing Tracer Results

$$tg = \frac{\sum Ct}{\sum C} \qquad \ldots (A)$$

Index of Short Circuiting,

$$Is = \frac{tg - tp}{tg} \qquad \ldots (B)$$

Dispersion Index, $d = \frac{D}{U'L'}$ ... (C)

$$\sigma^2 = 2d + 3d^2 \qquad \ldots (D)$$

$$\sigma_t^2 = \left(\frac{\sum t^2 c}{\sum c}\right) - \left(\frac{\sum tc}{\sum c}\right)^2 \qquad \ldots (E)$$

$$\sigma^2 = \frac{\sigma t^2}{tg^2} \qquad \ldots (F)$$

A.2 Sample Calculation of Hydraulic Limits from Flux Plots

Clarification Limit :(Please refer to Fig.29)

Assume the following conditions :

Influent MLSS = 3000 mg/l

Runs 10,11, 12 & 13

Zone Settling Velocity (ZSV) $= 27.0 \times 10^{-2}$ ft/mins

$= 16.2$ ft/hr.

Plan area of the basin available for clarification

= 5811 sq. ft

Based on, Overflow Rate = ZSV (ft/hr) x clarf.area(sq.ft) x 6.23(galls/cu.ft)x24 hrs/d,

the Limiting Hydraulic Loading = 16.2 x 5811 x 6.23x24 gall/d

(Q) = 14.1 MGD

Thickening Limit : (Please refer to Fig.30)

Assume the following conditions :

Influent MLSS = 3000 mg/l

Runs 10, 11, 12 & 13

Limiting total solids flux = 3.6 lbs/hr/sq.ft.

= 86.4 lbs/day/sq.ft.

$$\text{Based on Solids Loading} = \frac{\text{MLSS(mg/l x (MGD) x10(lbs/gall)}}{\text{Basin cross sectional area(sq.ft)}}$$

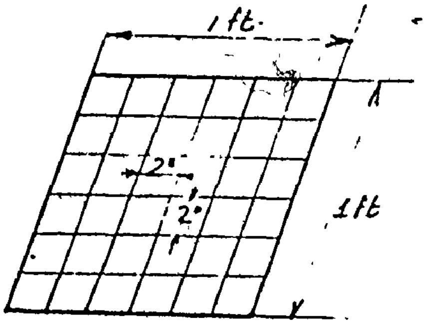
$$\text{the Limiting Hydraulic Loading, (Q+R)} = \frac{86.4 \times 5811}{3000 \times 10}$$

(Q+R) = 16.7 MGD

<u>Conclusion</u> : The conditions for Run Nos. 10, 11, 12 & 13 were such that the controlling regime was clarification.

<u>Note</u> : A similar calculation for Run Nos. 14, 15 & 16 indicates that the limiting condition had shifted to the thickening or solids transportation regime (see Table IX for the computed values).

A.3 Calculation of Reynolds Number in MicroFloc Tube Settler



Equivalent Diameter = 4 x Hydraulic Radius (Hr)

$$Hr = \frac{\text{Cross Sectional Area}}{\text{Wetted Perimeter}}$$

$$Hr = \frac{4\ in^2}{8\ in^2} = 0.5$$

Therefore, Equivalent Diameter = 4x0.5 = 2.0 ins

$$\text{As Rnd} = \frac{D\ V}{V_k}$$

Where V = Velocity through tube (ft./sec.)

V_k = Kinematic viscosity (sq.ft./sec.)

D = Diameter

As Reynolds No. depends on (V_k) and temperature and velocity (Q) in addition to "D"

As "D" is fixed = $\frac{2.0}{12}$ ft.

and Area = 4 sq. in.

$\therefore$ Assumptions:

$T = 32^oF$

$\therefore$ $V_k = 1.8 \times 10^{-5}$ sq. ft/sec.

If Q = 2 US GPM/sq. ft. as upflow rise through the the tubes and there is 36 tube/sq.ft.

$\therefore$ Q/tube = $\frac{2}{36} = \frac{1}{18}$ gpm

.... contd/

cont'd Reynolds No.

$$Q = \frac{1}{18x449} \quad \text{ft.}^3 \quad \text{sec.}$$

$$\therefore V = \frac{1}{18x449x4/144} \quad \text{ft./sec.}$$

$$= \frac{1}{18x449 \ x \ 4}$$

$$\therefore \quad Rnd = \frac{2.0}{12} \ x \ \frac{144}{18x449x4} \ \frac{1}{1.8x10^{-5}} = 41.32$$

∴ If Q increases to 3 gpm/sq.ft.

$$Q \ \text{Tube} = \frac{3}{36} = \frac{1}{12}$$

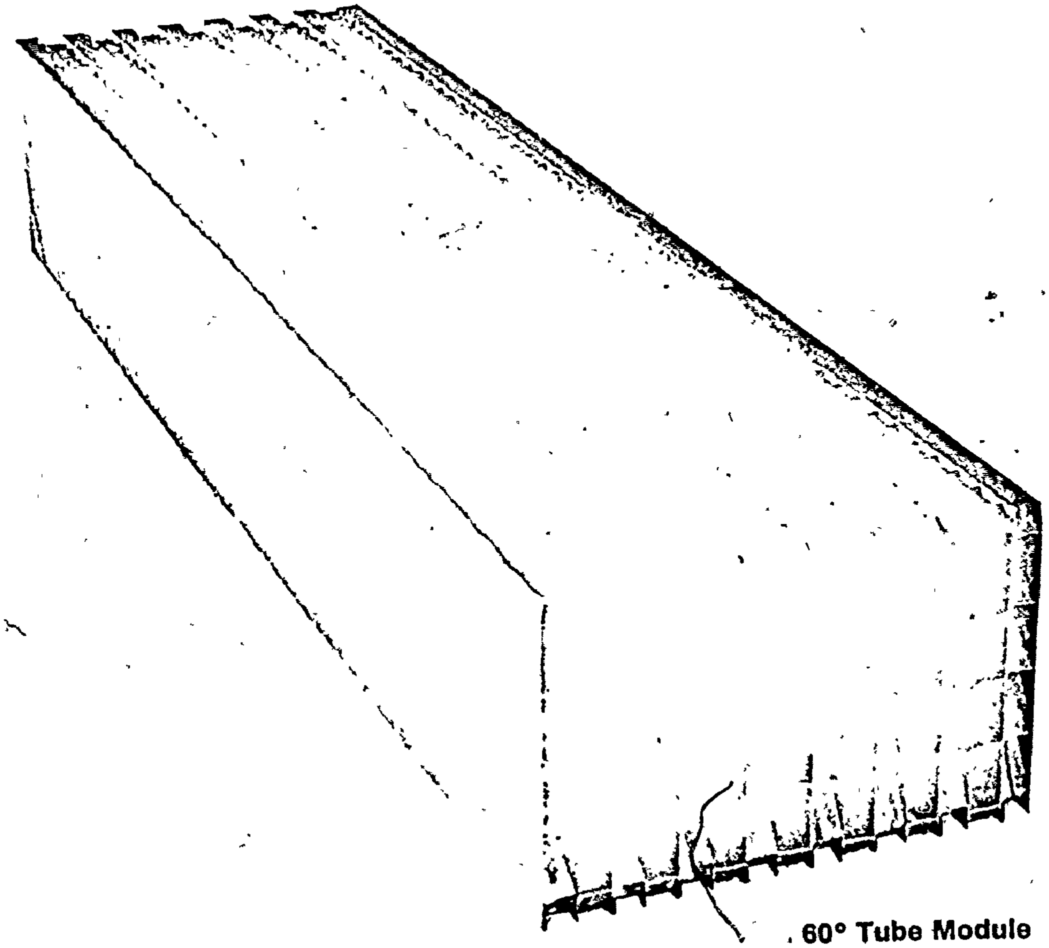
$$\therefore Rnd = \frac{41.32x18}{12}$$

$$= 62$$

A.4 Calculation of Extended Clarification Area provided by the MicroFloc Tube Settlers

The standard dimensions of a MicroFloc Tube Module are :

30 inches wide x 12 feet long x 24 inches deep, and inclined @ 60°. (As shown below)



Each tube is 2 inches square. There are 15 tubes across and 72 tubes along the length.

Plan Area per Standard Module = 30 sq. ft.

Total number of tubes per module = 1080

Therefore, number of tubes per unit plan area = 36.

Each tube has an inclined settling surface area of (2 inches x 24 inches) 48 sq. inches.

Therefore, tube settling area per sq. ft. of

$$\text{plan area} = \frac{48}{144} \times 36 \quad \frac{\text{sq.ft.}}{\text{sq.ft.}}$$

$$= 12.0$$

APPENDIX B

B.1 North End Water Pollution Control Centre(NEWPCC)

Sewage received at the treatment plant (see Table below) flows by gravity through a main interceptor into a surge well below ground level. Five high lift pumps transfer the sewage from the surge well to an above ground discharge chamber, from where it flows by gravity through the plant.

Flow	Initial Desg.	Ultimate Desg.
Average Annual Flow	55 MGD	82.5 MGD
Maximum Monthly Flow	70 "	105 "
Peakflow(Clarification)	143 "	182 "
Maximum Flow(Secondary)	110 "	165 "
Minimum Flow (For less than 1 hr.)	26 "	39 "

At the first stage, four aerated grit chambers receive the raw sewage and waste activated sludge from the secondary plant.

The degritted sewage next enters three primary settling tanks. The settled sludge is periodically directed to the digesters and the overflow is directed to the secondary.

A summary of the aerators and secondary clarifiers are given below;

Aeration Tanks: Number of Tanks 4, Each with 4 passes. Each pass is 30 ft. wide x 244 ft. long x 15 ft. average water depth.

Aeration Tanks: - contd.

Nominal detention @ 55 MGD & 25% return: 318 hrs. Air supply through coarse bubble spargers @ 1.5 Cu. ft. per gallon. Tank loading (design) 63 lbs. B.O.D. per day per 1000 Cu. ft.

Final Settling Tanks : 10 Units. Each 80 feet square x 12 ft.S.W.D.
detention time @ 55 MGD, 2.0 hrs.
Sudface loading @ 55 MGD, 860 gpd/sq. ft.
Weir loading @ 55 MGD, 9600 gpd/lin ft.
No surface skimming.
Centre feed with sludge scraper mechanism.

The settled sewage from the primary flow via a common conduit into the four aeration tanks. The flow to No. 3 aeration tank was split 60 per cent to No. 2 pass and 40 per cent to No. 3 pass (see Fig.B1). No. 1 pass (or 25% of the aeration tank) is used for re-aeration of the returned sludge from the clarifiers (only the return sludge from No. 7 clarifier entered No. 3 aeration tank during the entire period). Two air headers per tank supply the required air via spargers. The operators are instructed to maintain a minimum dissolved oxygen content of 1.5 to 2.0 ppm. The MLSS from No. 4 pass flows through a 10 foot wide conduit to the clarifier No. 7 and No. 8. This conduit is aerated but with no real control.

The two return pumps are inter-connected and therefore could draw sludge from either thank. The total capacity of these two pumps is 12.0 MGD.

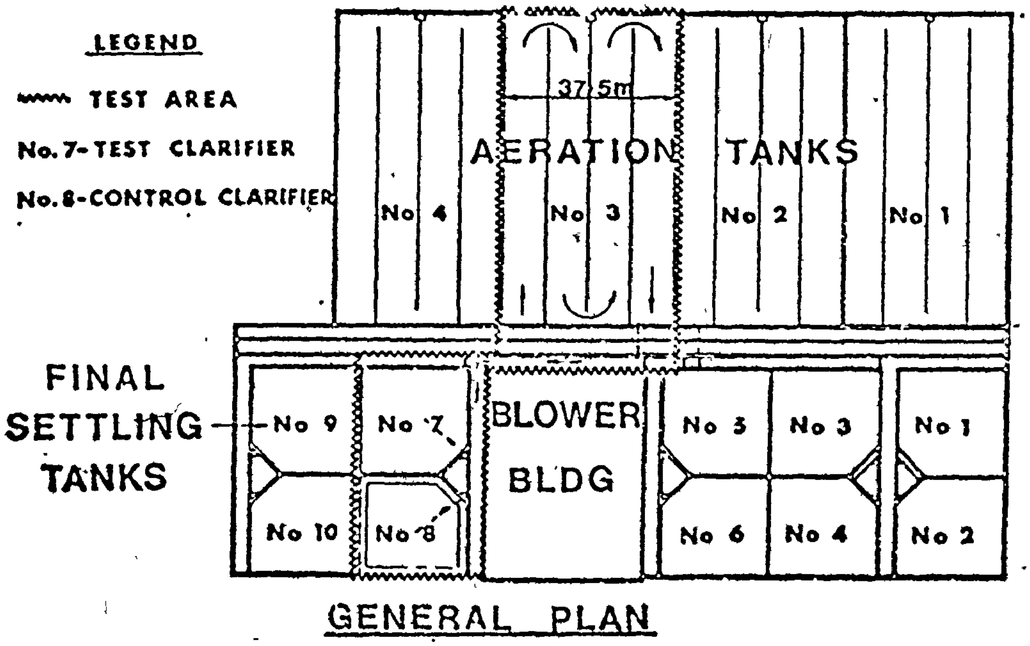


Fig.B.1 - Aeration Tanks - Final Clarifiers at NEWPCC

B.2 BOD/TOC Correlation on NEWPCC Raw Sewage and Treated Final Effluent

The following summarizes BOD/TOC correlations obtained on NEWPCC raw and treated effluent (final) from data gathered in 1974 and 1975.

NEWPCC Raw Sewage

1. 1974: BOD_{calc} (17 + 1.603 (TOC)) ± 51 mg/l
   R = 0.886
   n = 155
   BOD_{avg} = 277 mg/l
2. 1975: BOD_{calc} (18 + 1.482 (TOC)) ± 40 mg/l
   R = 0.889
   n = 175
   BOD_{avg} = 268 mg/l
3. 1974/75 Combined: BOD_{calc} (18 + 1.532 (TOC)) ± 47 mg/l
   R = 0.880
   n = 330
   BOD_{avg} = 272 mg/l

NEWPCC Final Effluent

4. 1974: BOD_{calc} = (1.7 + 1.027 (TOC)) ± 8.1 mg/l
   R = 0.874
   n = 166
   BOD_{avg} 34.6 mg/l
5. 1975: BOD_{calc} = (4.6 + 0.874 (TOC) + 8.5 mg/l
   R = 0.793
   n = 197
   BOD_{avg} = 40.3 mg/l

6. <u>1974/75 Combined:</u> $BOD_{calc} = (0.9 + 0.960\ (TOC)) \pm 8.4$ mg/l

$R = 0.848$
$n = 363$
$BOD_{avg} = 37.7$ mg/l

The above curves are shown on the accompanying graphs(I, II, and III).

It should be noted that acid preserved TOC samples were omitted from the calculation of the coefficients. Using 1974/75 data as a base BOD's can be calculated from corresponding TOC data according to the following relationships.

<u>Raw or Untreated Sewage</u>

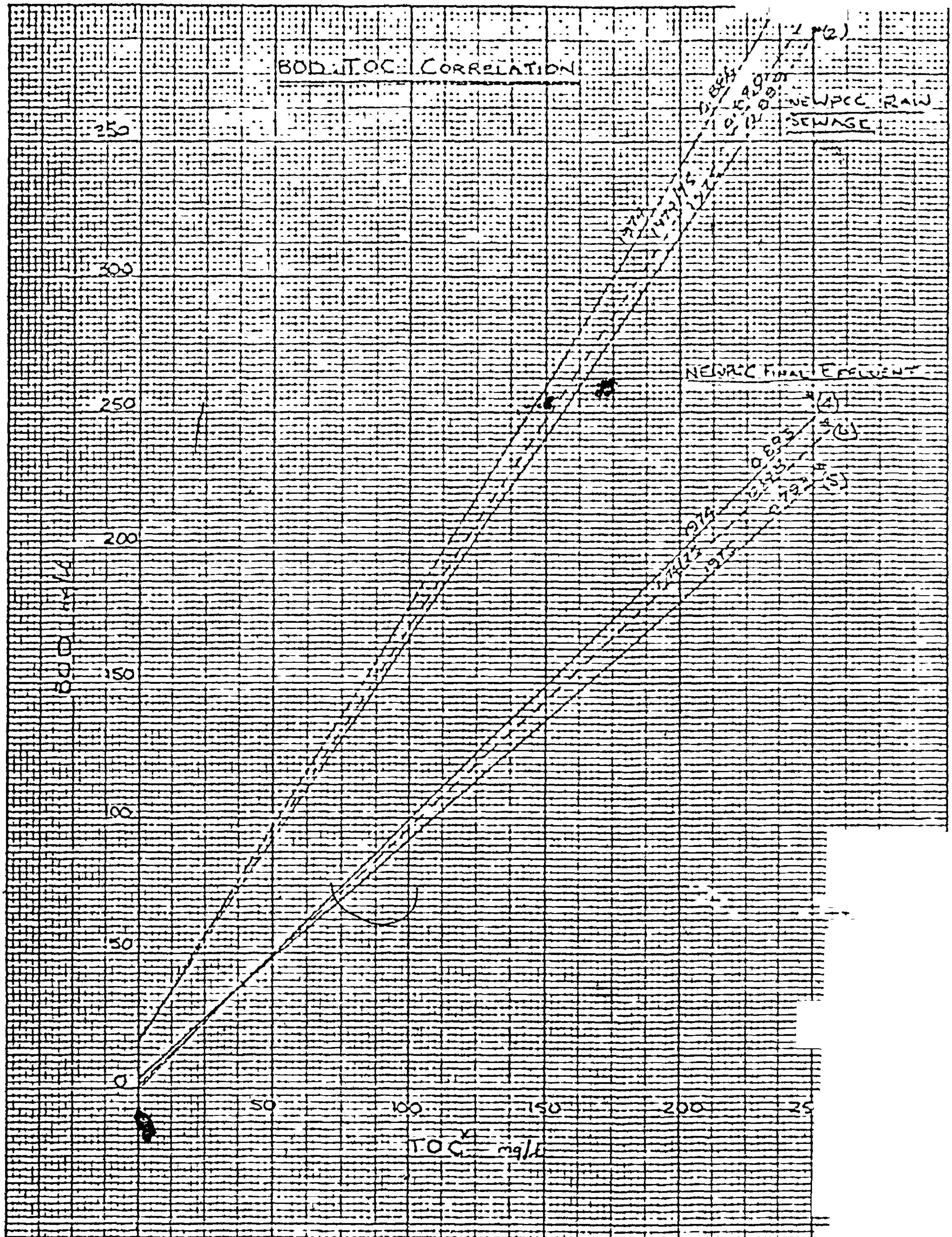
$BOD = (18 + 1.532\ (TOC)) \pm 47$ mg/l ..... (1)

<u>Secondary Treated Effluent</u>

$BOD = (0.9 + 0.960\ (TOC)) \pm 8.4$ mg/l .... (2)

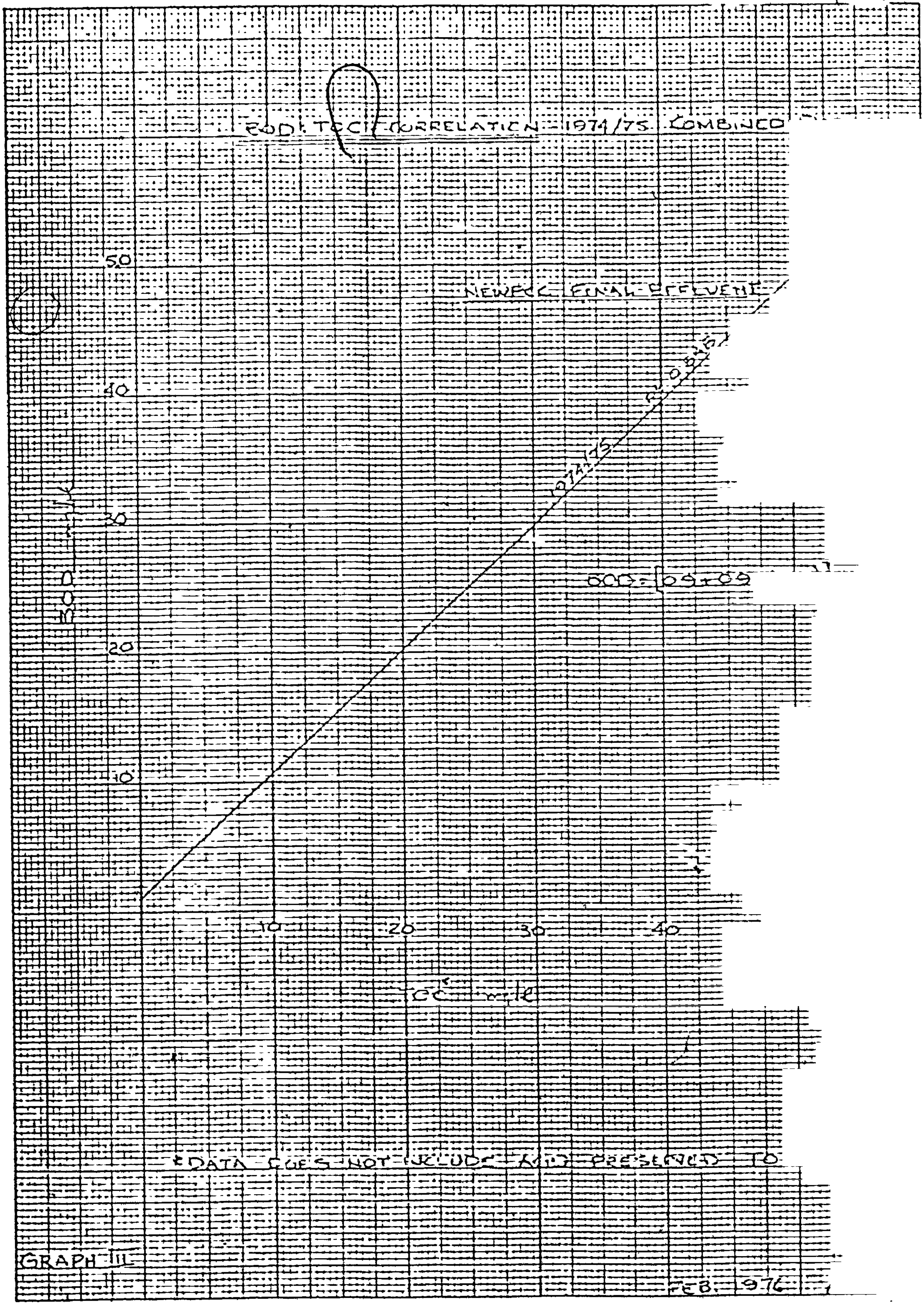
Utilising formula (1) above; "normal" sewage which according to City By-Law No.505/73 is defined as having a BOD of 300 mg/l will have a corresponding TOC level of 184 mg/l.

BOD-TOC CORRELATION



GRAPH I *DATA DOES NOT INCLUDE ACID PRESERVED TOC'S

FEB 1976 A ZALESKI



*DATA DOES NOT INCLUDE AND PRESENTED TO

GRAPH III

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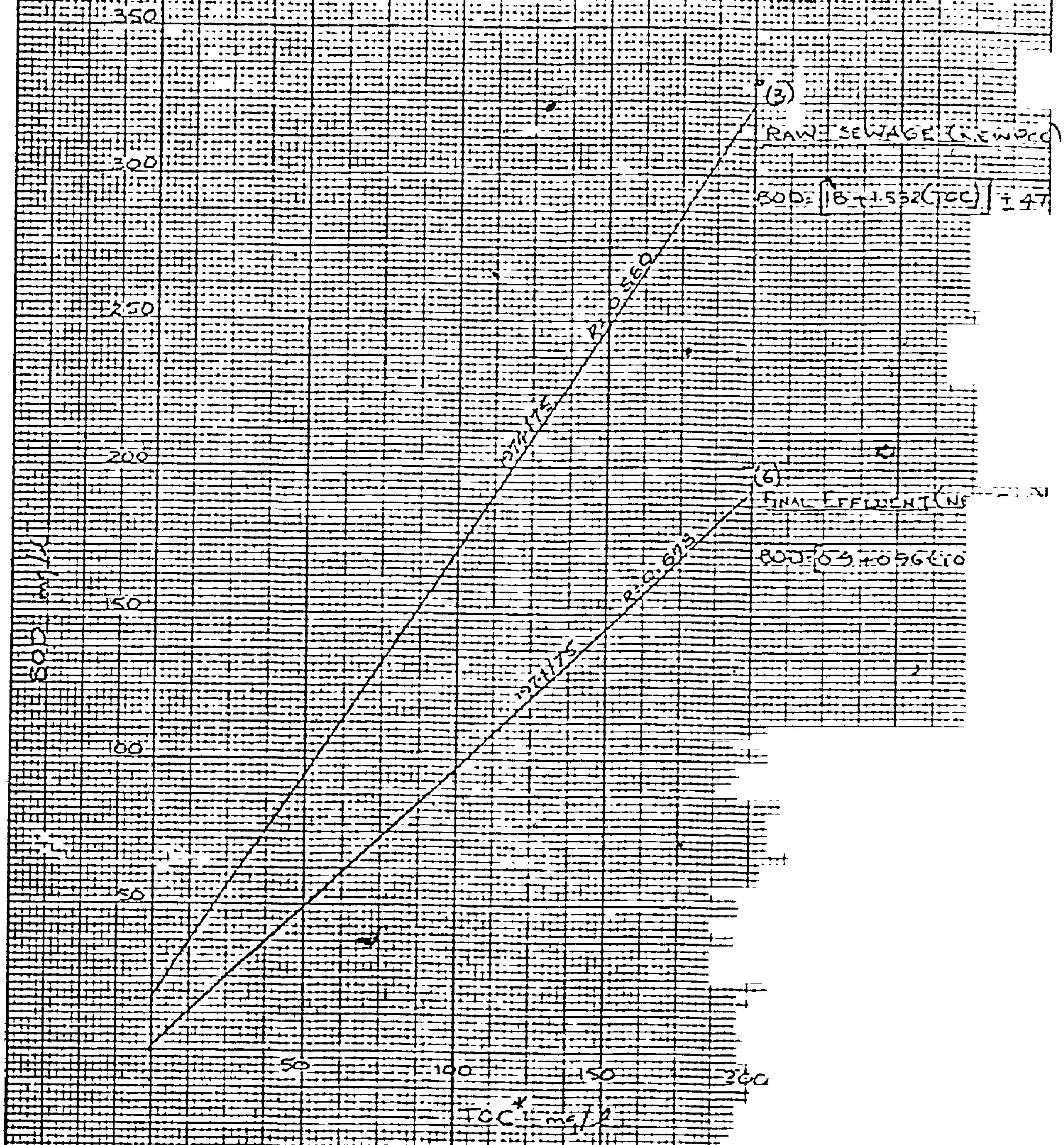
BOD:TOC* CORRELATION - 1974/75 COMBINED DATA



* DATA DOES NOT INCLUDE ACID PRESERVED

GRAPH 11

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APPENDIX C

C.1 Fluorescence

Before exploring the physical nature of fluorescence, it is well to distinguish this phenomenon from that of two other commonly confused terms "phosphorescence" and "luminescence". Fluorescence is the emission of radiant energy from matter, under the influence of an exciting agent. Phosphorescence isthe continuation of radiant energy emission after the removal of the exciting agent. Luminescence is a loose term covering both phenomena.

When a molecule of fluorescent material absorbs a photon of energy and an electron is raised to a higher energy level, this places the molecule in an excited state. Provided the molecule does not decompose as a result of the increase in energy and/or all the energy is not dissipated by subsequent collisions with other molecules, then after a short period of time characteristic of the atom or molecule the electron returns to the original lower energy level, emitting a photon in the process. The difference between the energy of the initial state and final state determines the energy of the emitted radiation, and it is this emitted radiation that is called fluorescence. Due to a small amount of energy being dissipated in the overall process (thermal deactivation) the emitted fluorescence has a lower energy and therefore a greater wave length and lower frequency than the energy which is absorbed. With certain frequencies of incident energy an electron is only excited to a higher vibrational level and not to a higher energy state. No energy is dissipated in the overall process in this case, and the photon of emitted energy is of the same value and wave length as the exciting photon. Ligh emitted in this manner is referred to as "Rayleigh Scattering."

C.2 Selection of Tracers

The ideal tracer could be defined as one possessing all of the following attributes :

a) It should be stable and inert to light, bacterial attack, chemical association, and absorption by suspended matter.

b) It must not be toxic in the concentration at which it shall be present.

c) It should be highly soluble, and therefore homogenous in solution.

d) It must have no large or variable "background" level of concentration in the waste stream normally, and

e) It must be capable of accurate analysis at the concentrations at which it shall be present,

Priesing (57) reports that one of the earlier dyes used, Fluqroscien, could not be used in studies involving sewage as the background fluorescence of sewage is near the same wave-length as Fluoroscien dye. Nevertheless, it had other attributes such as availability, non-toxic and low in cost. Rhodamine B, on the other hand exhibits appreciable absorption on solids and, consequently cannot be used strictly for quantitative studies.

Rhodamine WT not only fluoresces at wave-lengths not normally found in sewage (57) but also has low adsorptive properties on suspended matter.

A laboratory study compared Pontacyl Pink B, Rhodamine WT with respect to photochemical decay and adsorption properties. It was concluded that Pontacyl Pink B is superior to the two Rhodamines in these respects, with Rhodamine WT superior to Rhodamine B. Commercially, Pontacyl is four times more expensive than Rhodamine WT and is found as a powder which is not very pleasant to handle Due to which, Rhodamine WT has a wider acceptance.

C.3 Properties of 20% Rhodamine WT Solution

Appearance	Clear, very dark red aqueous solution, substantially free from insoluble matter.
Specific Gravity	Approx. 1.15 at 20/20°C. Gravity on a specific lot will be provided on request.
pH	10.8 ± 0.7 at 20°C.
Dispersion in Sea Water	Shows complete dispersion when dropped into sea water.

Tinctorial Strength	Minimum of 20% of Du Pont Rhodamine B Extra Standard (Spectrophotometric).
Bleachability	Bleachable with sodium hypochlorite.
Optimum Excitation Wave-length	About 556 nm.
Optimum Analyzing Wave-length	About 580 nm.
pH Sensitivity	No significant change in fluorescence between 5.5 and 11.0
Freezing Point	Approximately -10°C.
Viscosity	Less than 25 centipoises at 25°C.

C.4 Fluorescent Detection Instrumentation

There are three principal elements involved in the make up of instruments for direct detection of fluorescent material. They are (a) An energy source to cause excitation of the atoms or molecules, (b) Two filters to isolate specific regions of the energy source spectrum to coincide more precisely with the peak absorption wave-lengths of the compound under analysis and to isolate that region of the spectrum corresponding to the fluorescence wave-lengths and thus permitting only those wave-lengths of energy to pass on to the detector system, and (c) a photodetector.

The operation of these three elements is better understood if a particular fluorometer is examined (see Figs.C.1 & C.2).

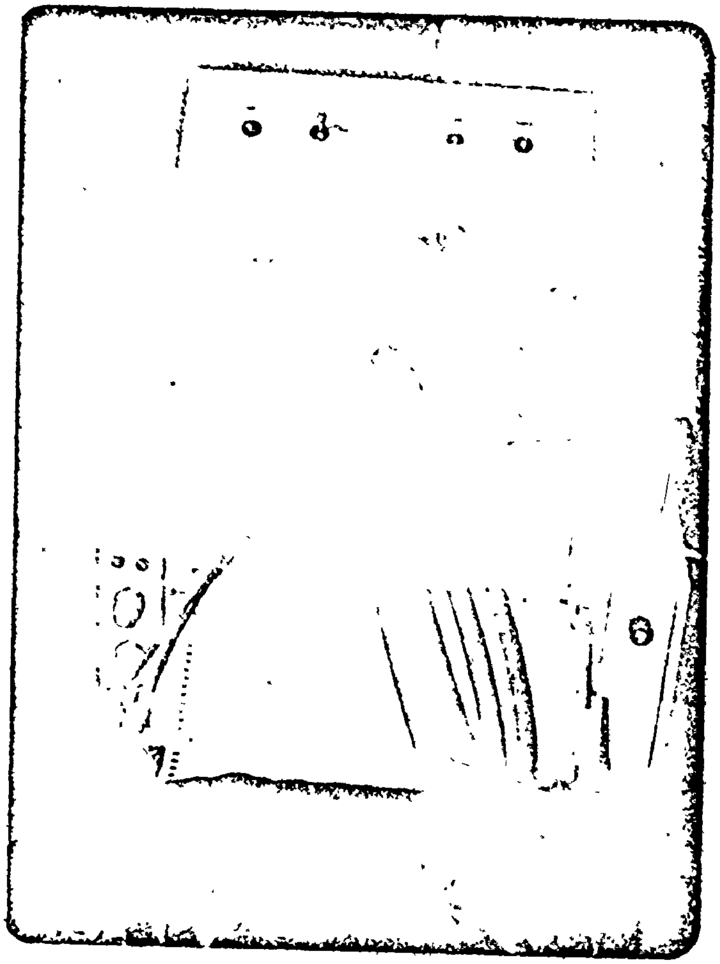

Fig. C.1 - The Turner 110 Fluorometer with matching recorder and variable speed drive chart for continuous monitoring of tracers.

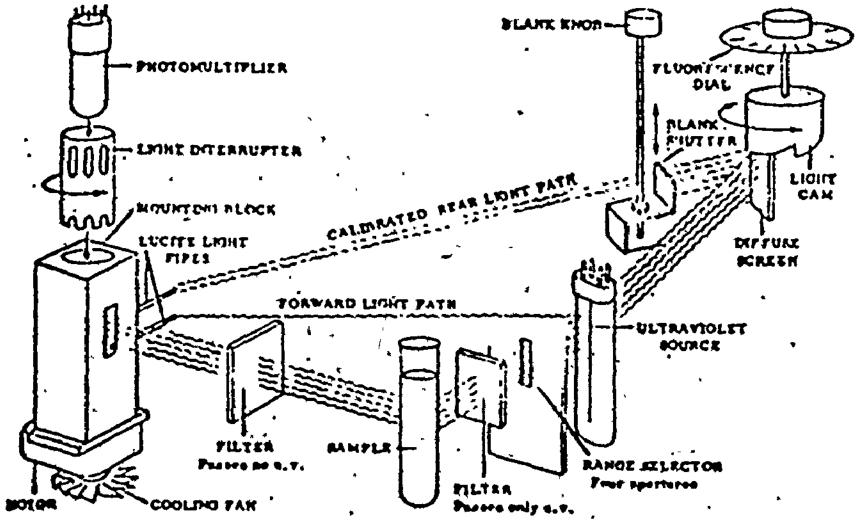

Fig. C.2 - The Turner 110 Fluorometer optical system.

In the Turner 110 Fluorometer the standard energy source is a low pressure 4 watt mercury lamp emitting mainly at 360 mu. Fluorescent energy from the sample passes through a slit into the photomultiplier tube. No lenses are used, so fluorometer applications which require excitation energy below 300 mu in wave-lengths are possible. Rather than measure the fluorescence directly, this energy is balanced against a small amount of radiant energy from the source. Balancing is carried out by a rapidly revolving interrupter which alternatively presents to the photomultiplier tube the fluorescent energy and the source energy. The two are balanced by appropriate electronics. This stable null-point machine can also be blanked against any fluorescence by use of a light-cam mounted on the rear light path. Sensitivity can be increased ten-fold by use of a high sensitivity kit. This is simply a combination of mirrors which pass the exciting light twice through the solution, and which reflect all fluorescent light toward the secondary filter by means of a collimated beam. A forward light path permits the machine to be zeroed to an "O" reading. Since only one light source is used, any aging of the bulb etc. will not affect the reading since all light beams are affected to the same extent. The machine is not light tight and is therefore sensitive to direct sunlight.

In order to record the concentration of tracer detected, the Turner fluorometer can be easily coupled with either a millivolt or milliampere recorder which has a variable speed chart drive. Specific instructions for this adaption is found within the instruction manual of the Turner fluorometer.

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