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PERFORMANCE AND DESIGN OF THE INTERCEPTOR
TANK FOR SMALL-BORE SEWERS

by
Samir Jomaa

A thesis
submitted under the supervision of
Dr. Eric J. Schiller and Dr. Ronald L. Droste

in partial fulfillment of the
requirements for the degree of
Master of Applied Science
in
Civil Engineering

Department of Civil Engineering
University of Ottawa
Ottawa, Canada
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October, 1988

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ISBN 0-315-46779-7



UNIVERSITÉ D'OTTAWA
UNIVERSITY OF OTTAWA

ACKNOWLEDGEMENTS

Special appreciation is expressed to Drs. E.J. Schiller and R.L. Droste for their assistance, professional guidance and support throughout this project. Also valuable assistance was given by Dr. M. McGarry from Cowater International.

The author would also like to thank Mrs. Francisco Aposaga, Robert Moore, and Claude Lavigne for their technical support and assistance with the laboratory analyses. Sincere appreciation is also expressed to Mr. Jason Russel who helped the author with sample analyses and typed part of this thesis.

Finally deep appreciation is expressed to the Tunisian Government and the National Research Council for providing financial assistance.

ABSTRACT

The purpose of this laboratory investigation was to study the performance and to improve the design of the interceptor tank for small-bore sewers. A full scale model was constructed for this purpose.

It is not desirable to design interceptor tanks to completely remove suspended solids because small-bore sewer systems have capability of handling some solids in suspension, particularly fines. This study was designed to test the influence of tank geometry and influent flush volume on the effluent suspended solids concentration. Once the solids carrying capacity of the small-bore system is determined, the results can be used to design a tank that would allow the maximum permissible solids concentration in the small-bore sewer and then extend the desludging interval to the maximum possible time period.

The following parameters were studied: (i) the inlet and the outlet locations, (ii) the flushed volume and (iii) the bottom slope of the interceptor tank. Because of the large number of possible combinations of the factors listed above, a fractional factorial design was used to select and perform experimentally only one third of all possible combinations. To predict the effluent suspended solids concentration for the combinations of variables which were not performed experimentally, a second-degree polynomial model was assumed.

The coefficients of the assumed model were estimated using the least squares method.

The effluent suspended solids concentration was found to depend mainly on the flushed volume and the outlet location. The bottom slope of the tank affected the effluent suspended solids concentration mainly at low sludge levels. The inlet location was not found to be critical, especially at high sludge levels.

To find an expression of the effluent and the tank suspended solids concentrations, different mathematical regressions were tested on the obtained values. The power regression was found to best describe this relationship.

Since small-bore sewers have the ability to carry fine and colloidal solid particles, optimum concentrations of suspended solids do not create major problems in the sewers. Based on this fact, locations of the inlet and the outlet were suggested.

The settleability of the effluent suspended solids from the interceptor tank was also tested for some selected combinations of variables. A laboratory settling column was used for this purpose. The settling velocities of the solid particles in the effluent flow were found to be very low when compared with the settling velocities of the solids taken from the tank. This demonstrates clearly that only fine and colloidal particles were conveyed to the sewers.

TABLE OF CONTENTS

	Page
ACKNOWLEDGEMENTS.....	i
ABSTRACT.....	ii
TABLE OF CONTENTS.....	iv
LIST OF TABLES.....	viii
LIST OF FIGURES.....	xi
GLOSSARY.....	xvii
CHAPTER 1 - INTRODUCTION.....	1
1.1 GENERAL.....	1
1.2 OBJECTIVES.....	5
CHAPTER 2 - LITERATURE REVIEW.....	8
2.1 INTRODUCTION.....	8
2.2 SEPTIC TANKS.....	10
2.2.1 Design of Septic Tank.....	12
2.2.2 Subsurface Absorption Field.....	14
2.2.3 Limitations.....	15
2.2.4 Performance.....	15
2.3 SMALL-BORE SEWER SYSTEMS.....	17
2.3.1 Historical Development.....	18
2.3.2 Design.....	19
2.3.2.1 Interceptor Tank.....	19
2.3.2.2 Small-Bore Sewers.....	20
2.3.3 Operation and Maintenance.....	22

2.3.4 Performance.....	22
2.3.5 Benefits.....	23
CHAPTER 3 - THEORY.....	26
3.1 SEDIMENTATION.....	26
3.1.1 Types of Sedimentations.....	26
3.1.2 Discrete or Type-I Sedimentation.....	27
3.1.2.1 Basic Principles.....	27
3.1.2.2 Turbulence and Scour Effect.....	30
3.1.2.3 Theory of Sediment Suspension by Turbulence.....	33
3.1.3 Type-II or Flocculent Sedimentation.....	35
3.1.4 Zone and Compression Settling.....	38
3.2 MASS BALANCE EQUATION.....	40
3.2.1 Introduction.....	40
3.2.2 Set Up of Mass Balance.....	41
3.2.3 Completely Mixed Tank.....	42
3.2.4 Plug Flow System.....	44
3.2.5 Case When the System is Non-Ideal.....	46
CHAPTER 4 - DESIGN AND EXPERIMENTAL PROCEDURE.....	48
4.1 APPARATUS AND EXPERIMENTAL SET UP.....	48
4.1.1 Design of the Interceptor Tank.....	48
4.1.2 Simulation of Domestic Wastewater.....	52
4.2 PRELIMINARY MEASUREMENTS.....	53
4.2.1 Settling Characteristics of the OM.....	53
4.2.2 Measurement of the Outlet Flow Rate.....	56
4.2.2.1 Calibration of the Supply Reservoir.....	56

4.2.2.2	Outlet Flow Rate.....	57
4.2.3	Measurement of the Short Circuiting.....	57
4.2.3.1	Calibration of the Fluorometer....	59
4.2.3.2	Determination of the Amount of Short Circuiting.....	60
4.3	EXPERIMENTAL DESIGN.....	62
4.3.1	Fractional Factorial Design.....	62
4.3.2	Choice of the Mathematical Model.....	64
4.4	EXPERIMENTAL PROCEDURE.....	65
4.4.1	Preparation of the Filters.....	65
4.4.2	Preparation of the OM.....	65
4.4.3	Procedure.....	66
4.4.3.1	Settling Column Tests.....	69
CHAPTER 5	- RESULTS AND DISCUSSION.....	71
5.1	RESULTS AND ANALYSIS OF THE EFFLUENT SS.....	71
5.1.1	Determination of the Regression Coefficients.....	72
5.1.2	Prediction of the Effluent SS.....	73
5.1.3	Comparison Between Measured and Predicted SS.....	77
5.1.4	Sensitivity Analysis.....	78
5.1.4.1	Effect of the Bottom Slope (S).....	78
5.1.4.2	Effect of the Flushed Volume (V)...	79
5.1.4.3	Effect of the Inlet Location (D)...	79
5.1.4.4	Effect of the Outlet location (L)..	79
5.1.5	Relation Between Effluent and Tank SS.....	80
5.1.6	Determination of the Rate of Accumulation.....	82
5.1.6.1	Assumption of the Inlet SS.....	82

5.1.6.2 Assumption of the Rate Constant K.....	83
5.1.6.3 Application of the Mass Balance Equation.....	84
5.1.6.4 Selection of Convenient Combinations of Variables for a Practical Design Case.....	91
5.2 SETTLING ANALYSIS OF THE EFFLUENT AND THE TANK OM.....	92
5.2.1 Settling Characteristics of the Tank Effluent.....	92
5.2.2 Settling Characteristics of the Tank OM.....	93
CHAPTER 6 - CONCLUSIONS.....	94
CHAPTER 7 - RECOMMENDATIONS FOR FUTURE RESEARCH.....	96
REFERENCES.....	97
APPENDIX A - FIGURES OF THE EFFLUENT SS CONCENTRATION VS TANK SS.....	104
APPENDIX B - COMPARISON BETWEEN MEASURED AND PREDICTED EFFLUENT SS.....	118
APPENDIX C - DATA FOR THE SETTLING COLUMN TESTS.....	132
APPENDIX D - FIGURES OF THE SETTLING COLUMN TESTS.....	138
APPENDIX E - SAMPLE PROCEDURE FOR ESTIMATING THE 15 COEFFICIENTS AND THE ANALYSIS OF VARIANCE...153	153
APPENDIX F - PROGRAM LISTING AND SAMPLE COMPUTER OUTPUT FOR THE RUNGE-KUTTA METHOD.....	162
APPENDIX G - NUMERICAL EXAMPLE.....	165

LIST OF TABLES

Table		Page
2.1	Characteristics of Typical Residential Wastewater (USEPA, 1980).....	13
2.2	Residential Water Use Estimates (USEPA, 1980).....	16
2.3	Summary of Effluent Data from Various Septic Tank Studies (USEPA, 1980).....	16
2.4	South Australia Criteria for Common Effluent Drainage Design (Otis, 1986).....	21
4.1	Proposed Testing Levels.....	50
4.2	Mass of OM for Different Sludge Volumes.....	53
4.3	Coded Variables.....	63
4.4a	Fractional Factorial Design.....	64
4.4b	Sample of the Four Possible Runs.....	64
4.5a	Experimental Runs (S = flat).....	67
4.5b	Experimental Runs (S = 1:2).....	67
4.5c	Experimental Runs (S = 1:1).....	67
4.6	Selected Settling Tests of the Effluents.....	70
4.7	Selected Settling Tests of the Tank OM.....	70
5.1	Summary of the Experimental Data for C_{eff}	73
5.2	Regression Coefficients for the Empirical Model....	74
5.3	Predicted Effluent SS Conc.....	75
5.4	Regression Coefficients for the Power Curve.....	81
5.5a	Time Required to Reach C_{eff} (Case when $C_{\text{in}} = 200 \text{ mg/L}$).....	85

5.5b	Time Required to Reach C_{xxx}	86
	(Case when $C_{in} = 400$ mg/L)	
5.5c	Time Required to Reach C_{xxx}	87
	(Case when $C_{in} = 600$ mg/L)	
C.1	Settling Column Analysis of the Tank Effluent (S= flat, Sludge Level= 21%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	133
C.2	Settling Column Analysis of the Tank Effluent (S= flat, Sludge Level= 43%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	133
C.3	Settling Column Analysis of the Tank Effluent (S= flat, Sludge Level= 64%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	133
C.4	Settling Column Analysis of the Tank Effluent (S= 1:2, Sludge Level= 21%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	134
C.5	Settling Column Analysis of the Tank Effluent (S= 1:2, Sludge Level= 43%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	134
C.6	Settling Column Analysis of the Tank Effluent (S= 1:2, Sludge Level= 64%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	134
C.7	Settling Column Analysis of the Tank Effluent (S= 1:1, Sludge Level= 21%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	135
C.8	Settling Column Analysis of the Tank Effluent (S= 1:1, Sludge Level= 43%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	135
C.9	Settling Column Analysis of the Tank Effluent (S= 1:1, Sludge Level= 64%) (V= 30 L, D= 0.3 m, L= 0.1 m).....	135
C.10	Settling Column Analysis of the Tank OM (S= flat, Sludge Level= 43%).....	136
C.11	Settling Column Analysis of the Tank OM (S= flat, Sludge Level= 64%).....	136
C.12	Settling Column Analysis of the Tank OM (S= 1:1, Sludge Level= 21%).....	137
C.13	Settling Column Analysis of the Tank OM (S= 1:1, Sludge Level= 43%).....	137

C.14 Settling Column Analysis of the Tank OM
(S= 1:1, Sludge Level= 64%).....137

E.1 Procedure for the Evaluation of the
15 Coefficients.....152

E.2 Summary of the Estimated Coefficients and
Analysis of Variance for the Fitted Model.....159

LIST OF FIGURES

Figure	Page
2.1	The Historical Development of Sewage Systems.....10
2.2	A Typical Septic Tank System.....11
2.3	Two-Compartment Septic Tank.....14
3.1	Relation Between C_D and R_N28
3.2	Settling Column and Iso-Percentage Plot.....37
3.3	Typical Plot of % SS Settled vs. Time.....37
3.4	Typical Plot of Interface Settling Curve.....39
3.5	Completely Mixed Tank.....43
3.6	Plug Flow Reactor.....45
4.1	Full Scale Interceptor Tank Model.....51
4.2a	Settling Column Test of the Simulated Domestic OM (Percentage SS Settled vs Time).....55
4.2b	Settling Column Test of the Simulated Domestic OM (Average Percentage SS Settled vs Time).....55
4.3	Supply Reservoir Calibration Curve.....56
4.4a	Flowrate Hydrograph $V = 30$ L.....58
4.4b	Flowrate Hydrograph $V = 45$ L.....58
4.5	Fluorometer Calibration Curve.....59
4.6a	Dye Outlet Conc. $V = 30$ L.....61
4.6b	Dye Outlet Conc. $V = 45$ L.....61
5.1	Plot of the Predicted Values of the Effluent SS Conc. ($S = \text{flat}$, $D = 0.1$ m, $L = 0.3$ m).....82

5.2	Tank SS Conc. vs Time (K= 0.0 1/d, C _{in} = 200 mg/L) (S= flat, D= 0.1 m, L= 0.3 m).....	88
5.3	Effluent SS Conc. vs Time (K= 0.0 1/d, C _{in} = 200 mg/L) (S= flat, D= 0.1 m, L= 0.3 m).....	88
5.4	Tank SS Conc. vs Time (K= 0.0 1/d, C _{in} = 400 mg/L) (S= flat, D= 0.1 m, L= 0.3 m).....	89
5.5	Effluent SS Conc. vs Time (K= 0.0 1/d, C _{in} = 400 mg/L) (S= flat, D= 0.1 m, L= 0.3 m).....	89
5.6	Tank SS Conc. vs Time (K= 0.0 1/d, C _{in} = 600 mg/L) (S= flat, D= 0.1 m, L= 0.3 m).....	90
5.7	Effluent SS Conc. vs Time (K= 0.0 1/d, C _{in} = 600 mg/L) (S= flat, D= 0.1 m, L= 0.3 m).....	90
A.1	Effluent SS Conc. vs Tank SS Conc. (S= flat, V= 30 L, D= 0.5 m, L= 0.3 m).....	105
A.2	Effluent SS Conc. vs Tank SS Conc. (S= flat, V= 30 L, D= 0.1 m, L= 0.3 m).....	105
A.3	Effluent SS Conc. vs Tank SS Conc. (S= flat, V= 15 L, D= 0.3 m, L= 0.3 m).....	106
A.4	Effluent SS Conc. vs Tank SS Conc. (S= flat, V= 45 L, D= 0.3 m, L= 0.3 m).....	106
A.5	Effluent SS Conc. vs Tank SS Conc. (S= flat, V= 30 L, D= 0.3 m, L= 0.1 m).....	107
A.6	Effluent SS Conc. vs Tank SS Conc. (S= flat, V= 30 L, D= 0.3 m, L= 0.5 m).....	107
A.7	Effluent SS Conc. vs Tank SS Conc. (S= 1:1, V= 30 L, D= 0.5 m, L= 0.3 m).....	108
A.8	Effluent SS Conc. vs Tank SS Conc. (S= 1:1, V= 30 L, D= 0.1 m, L= 0.3 m).....	108
A.9	Effluent SS Conc. vs Tank SS Conc. (S= 1:1, V= 15 L, D= 0.3 m, L= 0.3 m).....	109
A.10	Effluent SS Conc. vs Tank SS Conc. (S= 1:1, V= 45 L, D= 0.3 m, L= 0.3 m).....	109

A.11	Effluent SS Conc. vs Tank SS Conc. (S= 1:1, V= 30 L, D= 0.3 m, L= 0.1 m).....	110
A.12	Effluent SS Conc. vs Tank SS Conc. (S= 1:1, V= 30 L, D= 0.3 m, L= 0.5 m).....	110
A.13	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 30 L, D= 0.1 m, L= 0.1 m).....	111
A.14	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 30 L, D= 0.1 m, L= 0.3 m).....	111
A.15	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 15 L, D= 0.1 m, L= 0.3 m).....	112
A.16	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 45 L, D= 0.1 m, L= 0.3 m).....	112
A.17	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 15 L, D= 0.3 m, L= 0.5 m).....	113
A.18	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 45 L, D= 0.3 m, L= 0.5 m).....	113
A.19	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 15 L, D= 0.3 m, L= 0.1 m).....	114
A.20	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 45 L, D= 0.3 m, L= 0.1 m).....	114
A.21	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 15 L, D= 0.5 m, L= 0.3 m).....	115
A.22	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 45 L, D= 0.5 m, L= 0.3 m).....	115
A.23	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 30 L, D= 0.5 m, L= 0.1 m).....	116
A.24	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 30 L, D= 0.5 m, L= 0.5 m).....	116
A.25	Effluent SS Conc. vs Tank SS Conc. (S= 1:2, V= 30 L, D= 0.3 m, L= 0.3 m).....	117
B.1	Plot of the Measured and the Predicted SS Conc. (S= flat, V= 30 L, D= 0.5 m, L= 0.3 m).....	119
B.2	Plot of the Measured and the Predicted SS Conc. (S= flat, V= 30 L, D= 0.1 m, L= 0.3 m).....	119
B.3	Plot of the Measured and the Predicted SS Conc. (S= flat, V= 15 L, D= 0.3 m, L= 0.3 m).....	120

B.4	Plot of the Measured and the Predicted SS Conc. (S= flat, V= 45 L, D= 0.3 m, L= 0.3 m).....	120
B.5	Plot of the Measured and the Predicted SS Conc. (S= flat, V= 30 L, D= 0.3 m, L= 0.1 m).....	121
B.6	Plot of the Measured and the Predicted SS Conc. (S= flat, V= 30 L, D= 0.3 m, L= 0.5 m).....	121
B.7	Plot of the Measured and the Predicted SS Conc. (S= 1:1, V= 30 L, D= 0.5 m, L= 0.3 m).....	122
B.8	Plot of the Measured and the Predicted SS Conc. (S= 1:1, V= 30 L, D= 0.1 m, L= 0.3 m).....	122
B.9	Plot of the Measured and the Predicted SS Conc. (S= 1:1, V= 15 L, D= 0.3 m, L= 0.3 m).....	123
B.10	Plot of the Measured and the Predicted SS Conc. (S= 1:1, V= 45 L, D= 0.3 m, L= 0.3 m).....	123
B.11	Plot of the Measured and the Predicted SS Conc. (S= 1:1, V= 30 L, D= 0.3 m, L= 0.1 m).....	124
B.12	Plot of the Measured and the Predicted SS Conc. (S= 1:1, V= 30 L, D= 0.3 m, L= 0.5 m).....	124
B.13	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 30 L, D= 0.1 m, L= 0.1 m).....	125
B.14	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 30 L, D= 0.1 m, L= 0.3 m).....	125
B.15	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 15 L, D= 0.1 m, L= 0.3 m).....	126
B.16	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 45 L, D= 0.1 m, L= 0.3 m).....	126
B.17	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 15 L, D= 0.3 m, L= 0.5 m).....	127
B.18	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 45 L, D= 0.3 m, L= 0.5 m).....	127
B.19	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 15 L, D= 0.3 m, L= 0.1 m).....	128
B.20	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 45 L, D= 0.3 m, L= 0.1 m).....	128
B.21	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 15 L, D= 0.5 m, L= 0.3 m).....	129

B.22	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 45 L, D= 0.5 m, L= 0.3 m).....	129
B.23	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 30 L, D= 0.5 m, L= 0.1 m).....	130
B.24	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 30 L, D= 0.5 m, L= 0.5 m).....	130
B.25	Plot of the Measured and the Predicted SS Conc. (S= 1:2, V= 30 L, D= 0.3 m, L= 0.3 m).....	131
D.1	Settling Column Test of the Tank Effluent (S= flat, Sludge Level= 21%) (V= 30 L, D= 0.3 m, L= 0.1 m)	
	(a) Percentage SS Settled vs Time.....	139
	(b) Average Percentage SS Settled vs Time.....	139
D.2	Settling Column Test of the Tank Effluent (S= flat, Sludge Level= 43%) (V= 30 L, D= 0.3 m, L= 0.1 m)	
	(a) Percentage SS Settled vs Time.....	140
	(b) Average Percentage SS Settled vs Time.....	140
D.3	Settling Column Test of the Tank Effluent (S= flat, Sludge Level= 64%) (V= 30 L, D= 0.3 m, L= 0.1 m)	
	(a) Percentage SS Settled vs Time	141
	(b) Average Percentage SS Settled vs Time.....	141
D.4	Settling Column Test of the Tank Effluent (S= 1:2, Sludge Level= 21%) (V= 30 L, D= 0.3 m, L= 0.3 m)	
	(a) Percentage SS Settled vs Time.....	142
	(b) Average Percentage SS Settled vs Time.....	142
D.5	Settling Column Test of the Tank Effluent (S= 1:2, Sludge Level= 43%) (V= 30 L, D= 0.3 m, L= 0.3 m)	
	(a) Percentage SS Settled vs Time.....	143
	(b) Average Percentage SS Settled vs Time.....	143
D.6	Settling Column Test of the Tank Effluent (S= 1:2, Sludge Level= 64%) (V= 30 L, D= 0.3 m, L= 0.3 m)	
	(a) Percentage SS Settled vs Time.....	144
	(b) Average Percentage SS Settled vs Time.....	144
D.7	Settling Column Test of the Tank Effluent (S= 1:1, Sludge Level= 21%) (V= 30 L, D= 0.3 m, L= 0.1 m)	
	(a) Percentage SS Settled vs Time.....	145
	(b) Average Percentage SS Settled vs Time.....	145

D.8	Settling Column Test of the Tank Effluent (S= 1:1, Sludge Level= 43%) (V= 30 L, D= 0.3 m, L= 0.1 m)	
(a)	Percentage SS Settled vs Time.....	146
(b)	Average Percentage SS Settled vs Time.....	146
D.9	Settling Column Test of the Tank Effluent (S= 1:1, Sludge Level= 64%) (V= 30 L, D= 0.3 m, L= 0.1 m)	
(a)	Percentage SS Settled vs Time.....	147
(b)	Average Percentage SS Settled vs Time.....	147
D.10	Settling Column Test of the Tank OM (S= flat, Sludge Level= 43%)	
(a)	Percentage SS Settled vs Time.....	148
(b)	Average Percentage SS Settled vs Time.....	148
D.11	Settling Column Test of the Tank OM (S= flat, Sludge Level= 64%)	
(a)	Percentage SS Settled vs Time.....	149
(b)	Average Percentage SS Settled vs Time.....	149
D.12	Settling Column Test of the Tank OM (S= 1:1, Sludge Level= 21%)	
(a)	Percentage SS Settled vs Time.....	150
(b)	Average Percentage SS Settled vs Time.....	150
D.13	Settling Column Test of the Tank OM (S= 1:1, Sludge Level= 43%)	
(a)	Percentage SS Settled vs Time.....	151
(b)	Average Percentage SS Settled vs Time.....	151
D.14	Settling Column Test of the Tank OM (S= 1:1, Sludge Level= 64%)	
(a)	Percentage SS Settled vs Time.....	152
(b)	Average Percentage SS Settled vs Time.....	152

GLOSSARY

A	projected area of the particle, m^2
A_c	cross-sectional area of the plug flow reactor, m^2
a_p	acceleration of the particle, m/s^2
B_0	first coefficient of the assumed mathematical model, mg/L
B_1	second coefficient of the assumed mathematical model, mg/L
B_2	third coefficient of the assumed mathematical model, mg/L
B_3	fourth coefficient of the assumed mathematical model, mg/L
B_4	fifth coefficient of the assumed mathematical model, mg/L
B_{11}	sixth coefficient of the assumed mathematical model, mg/L
B_{22}	seventh coefficient of the assumed mathematical model, mg/L
B_{33}	eighth coefficient of the assumed mathematical model, mg/L
B_{44}	ninth coefficient of the assumed mathematical model, mg/L
B_{12}	tenth coefficient of the assumed mathematical model, mg/L
B_{13}	eleventh coefficient of the assumed mathematical model, mg/L
B_{14}	twelfth coefficient of the assumed mathematical model, mg/L
B_{23}	thirteenth coefficient of the assumed mathematical model, mg/L
B_{24}	fourteenth coefficient of the assumed mathematical model, mg/L
B_{34}	fifteenth coefficient of the assumed mathematical model, mg/L
C	mass concentration of material, mg/L
C_D	drag coefficient
C_{eff}	effluent suspended solids concentration, mg/L
C_{in}	influent suspended solids concentration, mg/L
C_T	tank suspended solids concentration, g/L
C_v	volume of particles divided by the total volume of the suspension
C_0	concentration of material at $y=0$, mg/L
C_1	concentration of material at y_1 , mg/L
C_2	concentration of material at y_2 , mg/L
D	distance of the inlet pipe from the bottom of the tank, m
d_p	diameter of the particle, m
F_D	drag force, N
f	Darcy-Weisbach friction factor
g	acceleration due to gravity, m/s^2
H	sludge depth, m

H_0	initial height of the compacted zone, m
H_z	final sludge depth, m
i	constant characterizing the suspension
j	letter representing any substance
K	rate constant, d^{-1}
k	coefficient related to the type of material being scoured
L	extended length of the outlet pipe, m
m	mass of the fluid displaced by the particle, kg
m_p	mass of the particle, kg
Q	volumetric flow rate, m^3/s
R	correlation coefficient
Re	Reynolds number
r	rate of reaction or digestion, $mg/L/d$
S	slope of the bottom surface of the tank
s_g	specific gravity of the particles
t	time, d, min, or s
t_c	critical time, d or s
t_d	hydraulic detention (or retention) time, d
t_m	time required to the particle to reach y_m , s
V	volume of water in the supply reservoir, L
V_z	flow velocity, m/s
V_H	horizontal flow velocity that will just produce scour, m/s
V_h	hindered settling velocity, m/s
V_p	volume of the particle, m^3
V_s	settling velocity of the particle, m/s
V_T	volume of the tank, m^3
v	velocity of the particle, m/s
v_0	upward initial velocity of the particle, m/s
x	axial distance, m
X_1	coded variable for the slope S
X_2	coded variable for the volume V
X_3	coded variable for the distance D
X_4	coded variable for the length L
y	elevation above the bed material, m
y_m	maximum height reached by a single particle, m
w	added mass coefficient
α	first constant of the power regression
β	second constant of the power regression
ρ	mass density of the fluid, kg/m^3
ρ_p	mass density of the particles, kg/m^3
μ	viscosity of the fluid, $N\cdot s/m^2$
ϵ	mixing coefficient

Abbreviations

Avg.	average
BOD	biological oxygen demand
BOD ₅	five days BOD
COD	chemical oxygen demand
Conc.	concentration
exp	exponential
ln	logarithmic base e
g/c/d	gram per capita per day
gal	US gallon
L/c/d	liters per capita per day
m ³ /c/y	cubic meters per capita per year
OM	organic matter
Setl.	settled
SS	suspended solids
TSS	total suspended solids

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Since the beginning of the 20th century, engineers have been searching for the simplest and the most economical method of wastewater disposal. A safe and convenient wastewater disposal is of paramount importance to human health and the well-being of any society. However, in many developing countries, most low-income areas have very poor sanitation. Most of these communities are not able to afford sanitary sewers. They use simple wastewater disposal techniques, such as pit latrines and septic tanks. It is not possible, however, to rely on pit latrines since, in many places, they do not provide the minimum required health standards. In addition, they are often not convenient and have fly and odour problems. On the other hand, it has been realized that septic tanks with subsurface absorption fields have improved the sanitary state of many communities (World Bank, 1986). However, in areas with high population densities, limited land space, poor soil conditions and high groundwater table, the subsurface absorption fields are not feasible or have failed shortly after their installation or before they reach the end of the designed lifespan (Laak, 1977).

Although onsite wastewater collection systems from single dwellings have been used for many years, they have often been considered an inappropriate solution. In many instances conventional sewers, which are a typical solution for wastewater collection system in large urban areas, have been applied to small and sub-urban communities. The total cost involved for such facilities imposes severe financial burdens on those communities (World Bank, 1986).

In areas where pit latrines and septic tanks with subsurface absorption fields cannot be implemented, the remaining sanitation option is an offsite alternative, where the sewage is collected and sent to a centralized treatment plant or a pond. This option is usually conventional sewerage which is a suitable and acceptable solution for the whole community. Since conventional sewers are costly to construct and maintain, often isolated communities and many developing countries cannot afford to pay for them. What is needed is an adequate solution for wastewater collection, that requires little or no government subsidies to construct, little operation and maintenance and provides the minimum required health standards. Small-bore sewerage is one solution. It eliminates the subsurface absorption field and replaces it by small diameter pipes. These pipes are connected to an existing septic tank or to an interceptor tank (usually smaller than a septic tank) which retains most of the large solid particles coming from the household.

The new technology of small-bore sewerage is gaining widespread acceptance and use. Small-bore sewerage, which is a combination of an onsite and an offsite disposal system, is considered as a viable alternative system for domestic wastewater collection. The system consists mainly of three components: the interceptor tank, the small diameter sewer pipes, and a collection area where treatment of the effluent and tank solids is done.

Small-bore sewer systems achieve a number of economies compared to conventional sewerage (Environment Protection Authority, 1979). The most obvious advantage is the small diameter pipes (100-150 mm) needed for the collection of domestic wastewater from dwellings. Pipe sizes are smaller and therefore plastic pipes, which are non-corrosive and lightweight, may be used. The most significant advantage is the saving in excavation costs since the pipes may follow the contour of the ground. Furthermore, constant and uniform slopes are not needed. Manholes are eliminated and only cleanouts are recommended. Since solids are trapped in the interceptors, high scouring velocities do not need to be maintained in the system.

The disadvantages are mainly the costs involved in the installation and desludging of the tank. Furthermore, to ensure the proper functioning of the system, a periodic flushing of the pipes may be necessary. Overall, small-bore sewer systems provide an equivalent level of service and health

benefits comparable to conventional sewers for at least one half of the cost (Sinnatamby et al., 1986).

The small-diameter pipes carry the partially treated effluent to a treatment plant, a conventional sewer system, or a stabilization pond for further treatment. These pipes have the ability to carry a portion of the solids load in the effluent flow. In fact, smaller diameter pipes have better solids transport properties (Sinnatamby et al., 1986) than larger pipes. To avoid deposition of solids and clogging of the pipes, an interceptor tank is provided to capture the solids coming from the dwelling. Ideally, the coarse particles will be settled out in the tank and only colloidal and fine solid particles, which have low settling velocities, will be carried away in the small-bore sewers. By allowing the maximum permissible solids in the sewers, the desludging interval of the tank will be maximized which will reduce operating cost.

Since tank desludging is a major operating expense and excess solids in the sewers will cause major problems, the design of the interceptor tank is critical. To control the concentration of solids in the effluent from the tank, the inlet and the outlet pipes must be properly designed and their respective locations carefully selected. Since the rate of accumulation of solids will be decreased, mainly due to the escape of solids in the sewers and some biodegradation effects, the size of the tank may be minimized, thus decreasing the frequency and hence the cost of desludging.

The implementation of small-bore sewerage is most appropriate in densely-populated, low-income areas in developing countries and in isolated communities in developed countries. Small-bore sewer systems are functioning without major problems in Australia (South Australian Health Commission, 1982), Brazil and Pakistan (Sinnatamby et al., 1986). In Australia for instance, small-bore sewer systems have been operating for as long as 20 years (Cowater International, 1985). These systems did not require high initial capital investment and most of the routine work (e.g., operation and maintenance) has been carried out by the community.

1.2 OBJECTIVES

The main function of interceptor tanks in small-bore sewerage, as well as in septic tanks, is to remove settleable matter from the wastewater passing through them. Improperly designed septic tanks cause the failure of the disposal system and create major problems to the householders. For instance, seepage beds (or subsurface absorption fields) become clogged due to the escape of solids from septic tanks into the effluent flow. However, the replacement of the seepage bed by small-bore sewers has solved this problem. Although these pipes are small in diameter, they have the ability to carry a portion of suspended solids (SS) in the effluent flow. The rate of accumulation of sludge, however, depends greatly on solids concentration leaving the inter-

ceptor tank. Therefore, the tank should be designed, considering among other factors, how much SS concentration is allowed in the sewers and what level of sludge accumulation is desired.

The wastewater entering the tank from the inlet pipe is used to stir up the sludge at the bottom. Since the inflow kinetic energy is dissipated in the tank, the lightweight particles are released into the small-bore sewers and the larger ones are retained and digested by aerobic and anaerobic bacteria. The flushing action introduces oxygen into the sludge and a large amount of turbulence is generated. The flow patterns in the tank and settling phenomena are complex. The flow pattern will be turbulent during the initial stages of inflow at low SS concentrations and as discharge continues the flow patterns will subside. It is difficult, however, to formulate theoretically the turbulence created in the interceptor tank since the phenomenon is complex and three dimensional. Therefore, a physical model study is most appropriate.

In this research a full scale model has been constructed to study the performance of the interceptor tank. This study has been designed to examine the influence of the following factors on the effluent SS concentration and settling characteristics in the tank effluent. These are:

- Interceptor tank geometries (size and shapes),
- Inlet and outlet locations and pipe geometry,
- Volume of water flushed into the interceptor tank,
- Level of sludge in the interceptor tank.

The purpose of this research is to determine experimentally overall performance of the interceptor tank as a function of these factors. The final result of this study will be to improve interceptor tank design by determining the suspended solids concentration that will escape for different geometries. Specifically this experimental study has the following main objectives:

1. Determine the amount of SS in the effluent for given flows and tank geometries and hence the escape rate of solids from the interceptor tank,
2. Study the settling characteristics of the effluent SS and tank solids, and
3. Determine the rate of accumulation of sludge in the interceptor tank.

Ideally it is desirable to remove all fine suspended and colloidal solids from the tank. However, the small-bore sewers have a certain sediment-carrying capacity limit, which should not be exceeded. The main idea is to maximize the concentration of lightweight solids in the small-bore sewers without overloading them. Quantitatively, on average the optimal amount of SS escaping from the tank will be estimated at the end of this research.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Onsite disposal systems have been used for many years throughout the world to dispose of wastewater generated from rural dwellings. In the United States, for instance, almost 30% of the population lives in unsewered areas and relies on onsite systems for wastewater treatment and disposal (McClelland, 1980). In developing countries the majority of the population use onsite disposal systems. The use of offsite disposal systems (i.e., sanitary sewers) is mostly restricted to large cities due to the high costs involved for their construction and operation.

In the early stages of development of onsite treatment and disposal, pit privies and cesspools are the most commonly used systems. They are only appropriate where moderate or small quantities of wastewater are generated. During the period following their installation, these systems are usually economical and safe. However, problems arise when the availability of water supply to developing communities increases. Rural communities usually change to septic tanks with subsurface absorption fields to accommodate the increase in wastewater flow. A rural population is generally satisfied

with the performance of septic tanks for the first few years following their installation. However, these systems often start to fail due to the incapability of the soil to handle the further increases in wastewater flow. Furthermore, when maintenance of the tanks is neglected, solid washout from septic tanks accumulates at the invert of the perforated pipes in the drainfield and accelerates the failure of the system.

The concept of small-bore sewers, also called septic tank effluent drains, was introduced to overcome the problem of drainfield failure. The main functions of the septic tank effluent drains were to replace the drainfield and to carry the effluent to an offsite disposal site. Solids washout is not a problem in this system as long as the concentration of solids is kept below the critical value. Systems are presently functioning without any major problems in developed and developing countries (Cowater International, 1985). Rural communities served by small-bore sewers are generally satisfied with the level of service, which is very close to conventional sanitary sewers.

Fig. (2.1) illustrates the most important steps in the development of onsite wastewater disposal systems. The septic tank and the small-bore sewer systems will be discussed briefly in the following paragraphs. Their advantages, disadvantages and limitations will be presented as well.

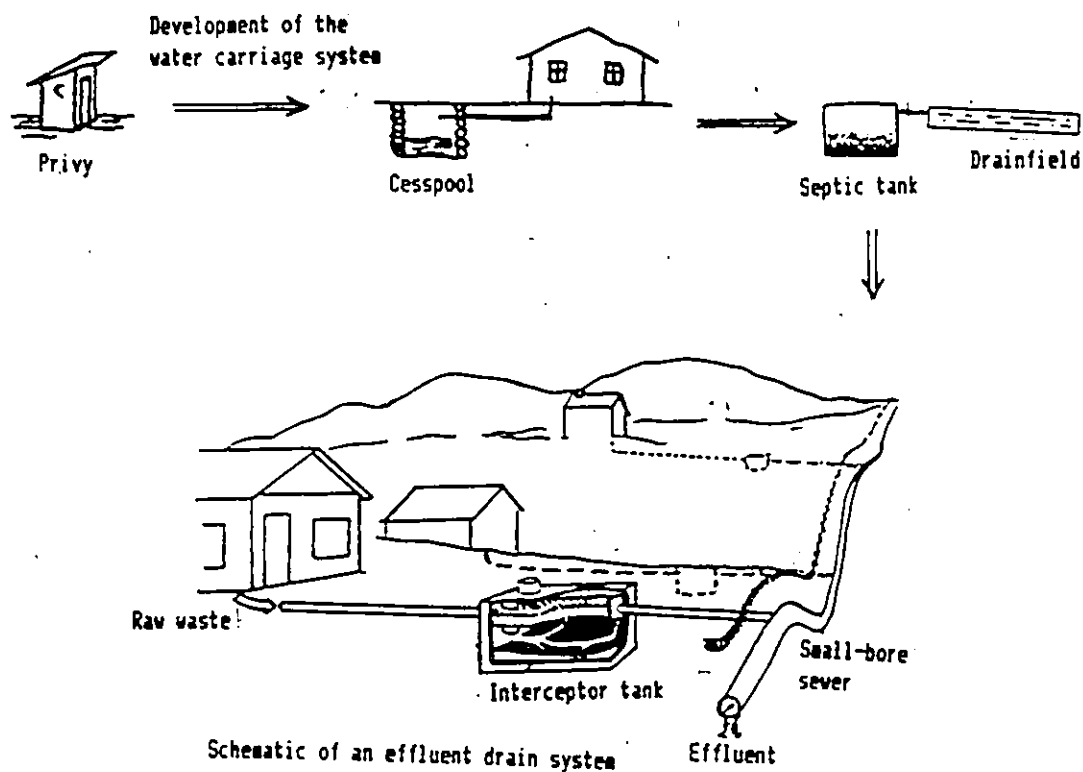


Figure 2.1 : The Historical Development of Sewage Systems

2.2 SEPTIC TANKS

The septic tank system is a very common means of wastewater disposal method in sewerless areas in the United States and other developed countries (World Bank, 1986). In developing countries most septic tanks are found in the wealthier suburbs. These systems require higher costs for construction and installation compared to other onsite disposal systems (World Bank, 1986). Fig. (2.2) shows a typical septic tank system.

The septic tank is an underground rectangular or cylindrical chamber that receives sewage flow from a household or other establishments. Its capacity varies between 1.1 and

3.8 m³ or even more (National Demonstration Water Project, 1982). Almost all settleable solids entering the septic tank are settled on the bottom of the tank, where they are anaerobically digested by microorganisms. The lightweight materials such as fats and grease rise to the top and form a scum. The partially treated effluent from a septic tank is discharged to a subsurface absorption field for further treatment and disposal. To prevent sludge from discharging into the subsurface absorption field, the accumulated material in septic tanks must be pumped out periodically, in general every 2 to 3 years (Metcalf and Eddy, 1972). However, larger tanks can be desludged every 3 to 6 years.

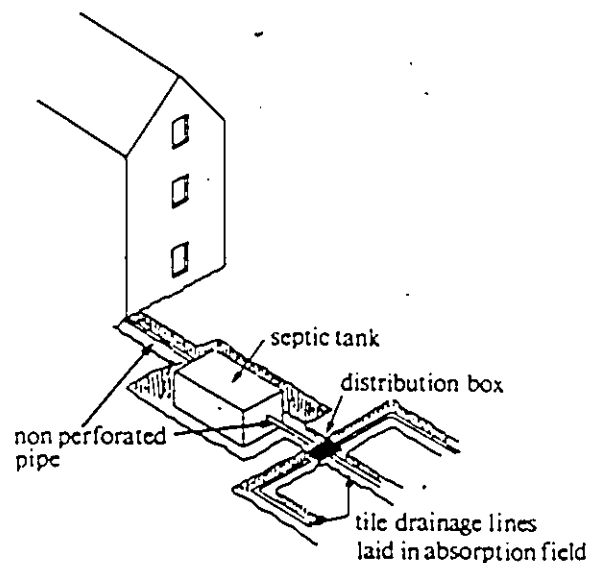


Figure 2.2 : A Typical Septic Tank System
(after World Bank, 1986)

2.2.1 Design of Septic Tank

An efficient septic tank, which removes almost all settleable solids, must provide (USEPA, 1980):

1. At least 24 hr fluid retention time at maximum sludge depth and scum accumulation.
2. Inlet and outlet tee structures to prevent the escape of solids in the effluent.
3. Adequate storage space for sludge accumulation.
4. A venting device to allow the accumulated gases such as methane and hydrogen sulfide to escape.

The most important consideration in designing a septic tank is the selection of its capacity. The tank volume is selected based on the average volume of wastewater produced per day. To estimate the volume of wastewater produced, factors such as the availability of water to the community, the wastewater-producing devices and the number of people per household must be taken into account. It was estimated that the average daily wastewater flow from a typical residential dwelling in industrialized countries is approximately 170 L/c/d (USEPA, 1980). The range of sewage flow from private homes varied between 150-300 L/c/d (National Demonstration Water Project, 1982).

The accumulation of sludge in a septic tank was estimated to be 0.03-0.04 m³/user/year (Kalbermatten et al., 1980a). More detailed estimates for the mass loadings and the wastewater characteristics for a typical residential

dwelling in North America are presented in Table (2.1). For design purposes two-thirds of the tank volume is normally reserved for sludge and scum accumulation (World Bank, 1986). This provides a three-day fluid retention time at start up. A theoretical minimum septic tank volume is 2.8 m³. However, septic tanks with volumes of 1.4 and 1.6 m³ for six persons have been used satisfactorily in Australia (Laak, 1980).

Table 2.1 : Characteristics of Typical Residential Wastewater (USEPA, 1980)

Parameter	Mass Loading (g/c/d)	Concentration (mg/L)
Total Solids	115-170	600-1000
Volatile Solids	65-85	380-500
SS	35-50	200-290
Volatile SS	25-40	150-240
BOD ₅	35-50	200-290
COD	115-125	680-730

To prevent the discharge of sludge and scum in the effluent, proper locations of the inlet and outlet must be provided. The inlet can be a sanitary tee or an elbow. The outlet can also be a tee or a baffle. To inspect the septic tank and to empty the settled sludge, manholes should be provided.

Although a single-compartment septic tank is widely used, a multi-compartment septic tank performs better in reducing

effluent solids and turbulence. Some research studies show that multichamber tanks are up to 50% more efficient in reducing SS and BOD in the effluent than single chamber tanks (Laak, 1980). A typical two-compartment septic tank is shown in Fig. (2.3). The first compartment is used for sedimentation, digestion and storage of the sludge. The second compartment provides additional safety against the discharge of solids that might escape the first compartment.

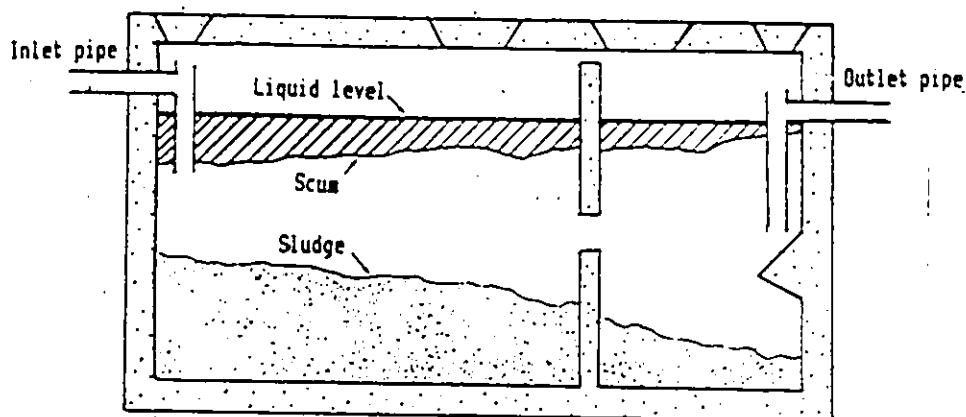


Figure 2.3 : Two-Compartment Septic Tank
(after World Bank, 1986)

2.2.2 Subsurface Absorption Field

The most commonly used method of disposal of septic tank effluent is by means of a subsurface absorption field (also called seepage bed, trenches, leachfield, or drainfield).

A subsurface absorption field consists of open-joint or perforated pipes laid at shallow depths, usually 0.3 to 1.5 m (USEPA, 1980), and covered with soil. These pipes allow the flow coming from septic tanks to percolate into the ground. To ensure the proper functioning of the subsurface absorption field, the soil must be at least moderately permeable and unsaturated throughout the year (USEPA, 1980).

2.2.3 Limitations

The septic tank system must be properly designed, constructed and installed in order to perform satisfactorily. However, some limitations and restrictions apply to ensure its performance:

1. The tank should be of watertight construction and made of materials not subject to excessive corrosion or decay.
2. No surface or subsurface drainage is allowed to enter the septic tank.
3. It has been shown that the rate of sludge and scum accumulation is increased by 37% when garbage grinders are used (USEPA, 1980). To keep the desludging frequency down, garbage grinders should be avoided.
4. If groundwater is the source of water supply for the community, the seepage bed must be sufficient distance from wells.

2.2.4 Performance

The quality of the effluent flow from septic tanks depends on many factors such as the volume of the tank, the number

of compartments, the inlet and outlet arrangements, and the quantity of wastewater flow entering the tank (refer to Table (2.2)). Table (2.3) shows a summary of effluent data from various septic tank studies in terms of BOD₅, COD and SS. The variation in the collected data from different studies is mainly attributed to the factors mentioned above.

Table 2.2 : Residential Water Use Estimates (USEPA, 1980)

Activity	L/use	Uses/c/d	L/c/d
Toilet Flush range	16 15-19	3.5 2.3-4.1	61 35-76
Bathing range	93 81-103	0.43 0.32-0.50	35 24-47
Clotheswashing range	142 127-151	0.29 0.25-0.31	38 28-44
Dis.washing range	33 26-47	0.35 0.15-0.50	12 4-19
Garbage Grinding range	8 - -	0.58 0.40-0.75	5 3-6
Miscellaneous range	- -	- -	25 22-30
Total range	- -	- -	173 157-197

Table 2.3 : Summary of Effluent Data from Various Septic Tank Studies (USEPA, 1980)

Parameter	7 sites	10 tanks	19 sites	4 sites	1 tank
BOD ₅ range (mg/L)	138 7-480	138 64-256	140 - -	240 70-385	120 30-280
COD range (mg/L)	327 25-780	- -	- -	- -	200 71-360
SS range (mg/L)	49 10-695	155 43-485	101 - -	95 48-340	39 8-270

The quality of the effluent flow very much affects the performance of the subsurface absorption field. It has been shown that the soil clogging is related to the SS and BOD concentrations in the effluent (National Demonstration Water Project, 1962). To extend the life of the system, the concentration of SS and BOD in the effluent must be very low at all times. Because of the difficulty in controlling the SS and BOD concentrations in the effluent, many septic tank systems have failed. The failure of the system could be also the result of improper design, maintenance of the tank and/or over estimation of the soil permeability. Based on the soil permeability, it is very difficult to predict the long term performance of the seepage bed to absorb the effluent flow. The increased water consumption and the failure to pump out the sludge from the septic tank have contributed to clogging of the soil.

To solve the problem of the seepage bed failure, some engineers suggested the replacement of the perforated pipes by small diameter pipes connected to the septic tank to carry the effluent flow to an offsite disposal.

2.3 SMALL-BORE SEWER SYSTEMS

The concept of small-bore sewers emerged as an immediate solution to onsite wastewater collection. The system is mainly composed of an interceptor tank in which most settleable solids are accumulated and digested, and the small

diameter pipes that convey the interceptor tank effluent to an offsite disposal.

2.3.1 Historical Development

The failure of septic tanks and leach fields in large numbers in South Australia prompted the Department of Public Health to think of another solution for domestic wastewater collection. The department proposed the use of small diameter drains as a corrective measure to the problem. In 1960 they constructed the first scheme of small diameter drains (also called common effluent schemes) in Pinnarco, South Australia. Following that, 800 km of drains serving 26,000 connections had been constructed by late 1982. Other common effluent schemes are in the planning stages. One 100-mm diameter pipe could serve up to 160 connections (South Australian Health Commission, 1982).

In the United States, however, the small-bore sewer system has been in existence since 1975 (Willson et al., 1975; Simmons et al., 1982). The first was installed in Mt. Andrew, Alabama. The system was constructed to test the performance of small-diameter pipes with uphill and downhill sections (also called variable grade gravity sewers) for domestic wastewater collection. Its operation and performance have been satisfactory.

In developing countries small-bore sewers have been used since late 1960. The system is working very well in Zambia and Nigeria (Cowater International, 1985). In Brazil and

Pakistan small diameter pipes are used to collect household wastewater. These pipes, usually of 100-mm diameter laid on a slope of 1 in 167, receive wastewater directly from homes and no interceptor tanks are used, and yet they perform very well (Sinnatamby et al., 1986).

2.3.2 Design

-2.3.2.1 Interceptor Tank

The design criteria for interceptor tanks follow closely those of septic tanks. However, the size of the former can be reduced significantly since solids washout into the sewers does not represent a major problem as long as large solid particles are retained in the interceptors. Some studies (Cowater International, 1986) show that an interceptor tank with a volume of 2.7 m³ is adequate regardless of household size. Knowing that the tank is the second highest cost in the system (Otis, 1986), and colloidal and fine solid particles do not create major problems, the volume of the interceptor tank could be even less than 2.7 m³. Other studies (Otis, 1986) suggested 3.8 m³ (1000 gal) as a minimum size for interceptor tanks although no justification was given for this particular number.

Since the disposal method of septic tank effluent was changed from onsite to offsite, the design criteria for interceptor tanks should not be the same as for septic tanks. To reduce the size and hence the cost of the interceptor

tank, further investigation is required to determine the design principles that an interceptor tank volume must be based on. Nevertheless, four design factors must be taken into account when choosing a proper volume for interceptor tank. These are:

1. The number of people in a household,
2. The wastewater generation per capita per day,
3. The expected rate of accumulation of sludge, and
4. The frequency of sludge removal.

2.3.2.2 Small-Bore Sewers

The small diameter pipes used for wastewater collection could be divided into two categories:

1. Small-bore gravity sewers, which are similar to conventional sewers.

The pipes must always be sloped downhill and uniform gradients must be maintained between manholes. These pipes are not allowed to flow under pressure at any time.

2. Small-bore variable grade sewers, which could follow the topography of the ground, sometimes running uphill over short distances.

These pipes are allowed to flow under pressure. Therefore, they are cheaper to construct than small-bore gravity sewers since excavation is minimized.

Generally design practice follows closely the South Australian guidelines (1982) shown in Table (2.4). The design criteria were believed to be too conservative. Other studies

(Otis, 1986; Cowater International, 1985) show that the minimum diameter could be as low as 50 mm (2 inches). According to these studies, no minimum slope is needed and no minimum flow velocity is recommended.

Table 2.4 : South Australia Criteria for Common Effluent Drainage Design (Otis, 1986)

Parameter	Recommended value
Design flow	Avg.: 136 L/c/d
Design depth of flow	1/2 full at capacity
Minimum pipe diameter	100 mm
Minimum velocity	0.5 m/s
Minimum slope	100 mm 0.67% 150 mm 0.40% 200 mm 0.33%
Pipe material	Earthenware and plastic
Pipe Roughness coefficient	Kutter's $n=0.013$
Alignment	Curved alignment in horizontal and vertical plane permitted between manholes
Manholes	At: intersection of 4 drains; intersection of 2 or more drains where depth is 2.4 m; 240 m intervals on minimum grades
Flushing points	At: terminal ends; changes in direction; junction of 2 drains 210 m intervals

2.3.3 Operation and Maintenance

Any system must be properly operated and maintained in order to perform properly during its design lifetime. Unlike conventional sewers, small-bore sewers require little operation and maintenance. Since pipes do not carry large solids and debris, simple routine operation and maintenance of the interceptor tank and the drains are needed. Each household must be responsible for the maintenance of its interceptor tank. The tank should be checked every year for scum and sludge accumulation. When necessary, the sludge from the interceptor tank must be removed. Although solids accumulation has not been a serious problem in any of the studied projects (Otis, 1986), an annual flushing of the drains is recommended.

Due to poor operation and maintenance, many problems have occurred. A major reported problem is the odor from lift stations and cleanouts (Otis, 1986). Broken cleanouts and cracked septic tanks caused the infiltration of clear water into the drains, which became surcharged. Concrete-made drains had also contributed to the problem of infiltration. The use of plastic pipes was found to be a very effective alternative.

2.3.4 Performance

Experience with small-bore sewers in rural communities showed good performance. In many communities, where small-

bore sewers were installed, maintenance was totally neglected and yet no major problems were reported (Otis, 1986). Although pipe diameters smaller than 100 mm were used in South Australia and the United States, the systems were successfully operated for over 20 years. According to the South Australian Health Commission (1982), "Although in some instances maintenance work has been less than recommended, the systems have functioned satisfactorily". Problems, where they did occur, were not associated with the small-bore sewers. Rather, they resulted mostly from inadequate inspection and maintenance of the pumps and the electrical equipment.

The good performance of small-bore sewers was believed to be the result of the high standards imposed for their construction, shown earlier in Table (2.4). Despite the conclusive evidence of successful operation of small-bore sewers, gravity-sewage standards are still used by engineers to design small-bore sewer systems.

2.3.5 Benefits:

Small-bore sewers with interceptor tanks are constructed to replace the single or multichamber septic tanks and drainfields. This results in many advantages:

- Increasing the design life of the system:

Because most settleable solids are removed in the interceptor tanks, clogging of the sewers is not expected to occur. These sewers should carry only colloidal and fine solid particles.

- Reducing the volume of the tank:

Since some solids are allowed to be washed into the sewers, the rate of solids accumulation in the interceptor tank is expected to be lower than in a septic tank. Therefore, a lower storage space for sludge can be considered when choosing the proper capacity of the tank.

- Reducing land use:

In the septic tank system, a considerable amount of land area is used to install the subsurface absorption field. This space is eliminated since the drains are used instead of the subsurface absorption fields.

- Reducing material needs and excavation:

Manholes in conventional sewers are replaced by less costly cleanouts or flushing points in small-bore sewers. Since interceptor tank provides some storage, the peak flows are attenuated. Therefore, pumping equipment may be reduced in size.

The drains need not be maintained on a straight path with a uniform gradient. They can be laid with curvilinear alignment and can follow the ground contour.

- Reducing infiltration and maintenance:

Because of fewer manholes, shallow installation and plastic material used for the drains, the infiltration of clear water is much less than that experienced with conventional sewers.

Maintenance is only limited to periodic flushing of the drains and pumping the sludge out of the interceptor tanks, when necessary.

- Reducing treatment:

Since screening, grit removal and primary sedimentation are performed by individual interceptor tanks, these unit processes can be eliminated from the treatment works.

Much more savings could be achieved if the design criteria for small-bore sewers were used which are less conservative than those of conventional sewers. Since designers are still uncertain about their long-term performance, they continue to apply conventional sewer design criteria.

CHAPTER 3

THEORY

3.1 SEDIMENTATION

3.1.1 Types of Sedimentations

Sedimentation is the most widely used unit operation in water and wastewater treatment. It is defined as the removal or separation of both inorganic and organic materials, which have densities greater than the density of water, by gravitational settling. To describe this process, other terms such as clarification and thickening are used. The main purpose of sedimentation is to obtain a relatively clarified effluent free from debris and large solid particles.

Depending on the concentration and the characteristics of the particles in suspension, four general categories of settling can be distinguished. These four types could occur at the same time or simultaneously in the same suspension.

Type-I Settling or Discrete Particle Sedimentation

This refers to the settlement of non-flocculent, discrete particles in a quiescent system. These particles settle as individuals and no interaction with other particles in suspension occurs.

Type-II Settling or Flocculent Sedimentation

Most particles in wastewater do not act as discrete particles. Rather they interact with each other during the process of sedimentation. The mass of the particles increases as they coalesce and flocculate, resulting in a faster downward movement.

Type-3 or Zone Settling

Zone settling occurs in suspensions of intermediate to high concentrations. A zone or blanket will be formed in which settling particles act as a group. The particles remain in fixed positions with respect to each other and their mass settles as a whole unit. This causes an upward displacement of the fluid and hindered settlement.

Type-4 or Compression Settling

Compression settling occurs when further settling can only happen by compression of the structure formed by the high concentration of solids in the lower layers of deep sludge masses. Due to the weight of the added particles to the structure, compression and consolidation of the material take place. Since pore volume between particles is decreased, the rate of settlement decreases dramatically.

3.1.2 Discrete or Type-I Sedimentation

3.1.2.1 Basic Principles

The fundamental laws of discrete particle settling in a stationary fluid were first developed by Newton and Stokes.

Newton proposed the law of drag, which stated that the drag force applied on a single particle is proportional to the square of its velocity:

$$F_D = C_D \frac{A_D \rho}{2} v^2 \quad (3.1)$$

in which F_D = drag force
 C_D = drag coefficient
 A_D = projected area of the particle in the direction of motion
 ρ = mass density of the fluid
 v = velocity of the particle

As shown in Fig. (3.1) the drag coefficient C_D was found to be a function of the Reynolds number (R_N).

$$R_N = \frac{\rho v d_D}{\mu} \quad (3.2)$$

where d_D = diameter of the particle
 μ = viscosity of the fluid

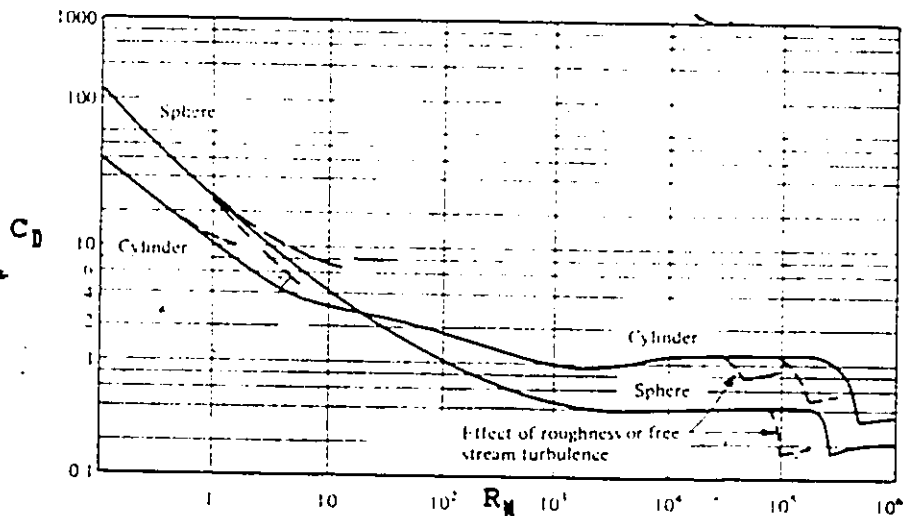


Figure 3.1 : Relation Between C_D and R_N
 (after Clark et al., 1977)

A particle released in a quiescent fluid will accelerate until its terminal velocity becomes nearly constant. This is usually known as the settling velocity of the particle. It is obtained by equating the effective weight to the drag force:

$$(\rho_s - \rho)gV_D = C_D \frac{A_D \rho}{2} v^2 \quad (3.4)$$

where ρ_s = mass density of the particle
 V_D = volume of the particle

Substituting the settling velocity V_s for v and solving:

$$V_s = \left[\frac{2 g V_D}{C_D A_D} \left(\frac{\rho_s - \rho}{\rho} \right) \right]^{0.5} \quad (3.5)$$

For spherical particles Eq. (3.5) may be rewritten:

$$V_s = \left[\frac{4 g d_D}{3 C_D} \left(\frac{\rho_s - \rho}{\rho} \right) \right]^{0.5} \quad (3.6)$$

To express C_D as a function of R_N a mathematical equation was proposed to approximate the curve shown in Fig. (3.1). This equation has the following form:

$$C_D = \frac{24}{R_N} + \frac{3}{R_N^{0.5}} + 0.34 \quad (3.7)$$

It was found experimentally that the first term in Eq. (3.7) predominates when R_N is less than 0.3. Substituting $24/R_N$ for C_D in Eq. (3.6) yields:

$$V_s = \frac{g d_D^2}{18\mu} (\rho_s - \rho) \quad (3.8)$$

Eq. (3.8) was first developed by G.G. Stokes, who found that the drag force in laminar flow conditions has the following expression:

$$F_D = 3\pi\mu v d_p \quad (3.9)$$

Equating Eq. (3.9) to the effective weight of the particle also yields Eq. (3.8).

In developing the above formulas for type-I sedimentation the following assumptions were made:

- Solid particles are single and discrete,
- Particles are spherical in shape,
- Fluid is viscous,
- Ideal conditions are maintained during sedimentation with no turbulent effects (quiescent fluid),
- Wall effects are insignificant (infinite size settling basin).

3.1.2.2 Turbulence and Scour Effects

In settling basins, turbulence generated by the incoming flow must be kept to a minimum, so that settled particles are not scoured from the bottom of the basin. The critical velocity of the inflow that will just produce scour was first developed by Camp using Shields' studies:

$$V_H = \left[\frac{8k (s_g - 1) g d_p}{f} \right]^{0.5} \quad (3.10)$$

in which V_H = horizontal flow velocity that will just produce scour
 s_g = specific gravity of the particles

k is a coefficient related to the type of material being scoured. It takes the values 0.04 for sand, and 0.06 for sticky and interlocking matter.

f is the Darcy-Weisbach friction factor which depends on the characteristics of the surface over which flow is taking place and the R_w . Typical values range from 0.02 to 0.03.

If a large degree of turbulence is created in the sedimentation basin, the movement of the particles will be affected. Particles tend to move vertically, horizontally or in all directions depending on the type and direction of the turbulent inflow.

In many instances the inflow conduit may be located in the center and close to the bottom of the basin. The flow generated is in an upward movement which tends to lift particles to the surface of the fluid. Fine solid particles, already settled on the bottom of the basin, are affected the most by the turbulent inflow. Large and heavy particles rise to short heights and rapidly fall as the turbulent inflow stops or is reduced in intensity. However, colloidal and fine solid particles stay in suspension for longer periods of time.

In the development of the following equations, the particles are assumed to move upward (due to the turbulent inflow) with an initial velocity v_0 caused by the friction of water flowing over the particles. The maximum height reached by a particle is obtained when its velocity becomes zero, at which time its movement will be reversed. The maximum height

reached by a single particle must be expressed in terms of the characteristics of the particle (i.e., size and density), and the characteristics of the fluid (i.e., density and viscosity) at standard pressure and temperature. Due to the complexity of the differential equation obtained when R_N is greater than 0.3, the equation developed will be only applicable for low R_N .

A single particle lifted with an initial velocity v_0 rises until its velocity reduces to zero. Applying Newton's Second Law:

$$F_D + (\rho_s - \rho)gV_D = (m_D + w_m) a_D \quad (3.11)$$

where m_D = mass of the particle
 m = mass of the fluid displaced by the particle
 w = added mass coefficient = 0.5 for a sphere
 a_D = acceleration of the particle

Let $v' = a_D$, and $F_D = 3\pi\mu v d_D$ for spherical particles at low R_N . Then the previous equation becomes:

$$v' - \frac{36\mu}{d_D^2(2\rho_s + \rho)} v = g \frac{2(\rho_s - \rho)}{(2\rho_s + \rho)} \quad (3.12)$$

Which is a first order differential equation with a general solution:

$$v \exp(Mt) = \frac{N}{M} \exp(Mt) + \left[v_0 - \frac{N}{M} \right] \quad (3.13)$$

$$\text{where } M = - \frac{36\mu}{d_D^2(2\rho_s + \rho)}, \text{ and } N = g \frac{2(\rho_s - \rho)}{(2\rho_s + \rho)}$$

t = time

v_0 = upward initial velocity

Substituting $v = 0$ in Eq. (3.13), the corresponding time, t_m is obtained:

$$t_m = \frac{1}{M} \ln \left[1 - \frac{M}{N} v_o \right] \quad (3.14)$$

Knowing that $v = dy/dt$, the maximum height y_m reached by a single particle may be obtained using Eqs. (3.13) and (3.14):

$$y_m = \frac{N}{M^2} \ln \left[1 - \frac{M}{N} v_o \right] + \frac{1}{M} \left[v_o - \frac{N}{M} \right] \left[1 - \frac{N}{N - M v_o} \right] \quad (3.15)$$

Because of the interaction between the material in suspension, particles having the same size and density will not be able to reach the same maximum height. To take into account this fact, a simplified theory of the suspension of sediments due to turbulence will be discussed in the following section.

3.1.2.3 Theory of Sediment Suspension by Turbulence

In a turbulent system, the eddies created by the turbulent inflow carry material settled on the bed into suspension. Due to their weights this action will be opposed by the settling of the particles.

For simplicity it is assumed that a steady state condition exists, in which equilibrium is maintained between the rate of upward transport due to turbulent eddies and the rate of fall due to the weight of particles at every elevation above the bed. This statement was expressed mathematically by Dobbins (1944) :

$$V_m C = - \epsilon \frac{\partial C}{\partial y} \quad (3.16)$$

where V_s = settling velocity of the particles
 y = elevation above the bed
 C = concentration of material at elevation y
 ϵ = mixing coefficient which may be expressed as a function of y

Integrating Eq. (3.16) between C_0 at $y = 0$ and C at any elevation y , and assuming that the mixing coefficient ϵ is independent of y the following expression of C at elevation y is obtained:

$$C = C_0 \exp\left[-\frac{V_s}{\epsilon} y\right] \quad (3.17)$$

If the concentrations C_1 at y_1 and C_2 at y_2 , and the settling velocity of the particles are known, the value of ϵ could be estimated by the following equation:

$$\epsilon = (y_2 - y_1) \frac{V_s}{\ln(C_2/C_1)} \quad (3.18)$$

In the real suspension, however, ϵ varies with the distance from the bed. If the inflow is unsteady, ϵ will also vary from time to time at a particular location. The formulas presented above concern the case of the sedimentation of discrete, single particles of uniform size in a system in which turbulence is fully developed. Different size particles with different settling velocities may exist in the same suspension. Under these conditions, it is difficult to formulate theoretically the effect of turbulence and mixing on a suspension of particles, and a much advanced theory with minimum assumptions must be developed. Since the flow pattern is complex and three dimensional, the formulas pre-

sented earlier will not be used, and an experimental study is most appropriate.

3.1.3 Type II or Flocculent Sedimentation

Particles in real wastewater do not settle individually, they interact and affect one another's path during the process of sedimentation. The interaction between the single particles is more pronounced as their concentration in the solution is high. During the process particles coalesce and cluster forming larger masses which accelerate as they move downward until they reach the bottom surface of the basin.

Unfortunately there are no theoretical relationships which describe the behavior of particles during flocculent settling. Rather, settling basin designers commonly use the settling column test to describe the characteristics of the wastewater during the process of flocculent sedimentation.

A settling column consists of a plastic tube with diameter of at least 10 cm (4 in) and height of a few meters. The height of the column should be equal to the depth of the proposed tank. Sampling ports located at various depths should be provided. The wastewater solution, placed in the settling column, is allowed to settle in a quiescent manner. The initial concentration of suspended matter should be as uniform as possible throughout the settling column. This concentration should be below 2000 mg/L (Zanoni and Blomquist, 1975) since beyond this value zone and compression settling (refer to Section (3.1.4)) take place. To

eliminate any possible convection currents, the temperature must be maintained uniform throughout the test.

The procedure for a flocculent settling test consists of withdrawing samples from the ports at various selected times. The concentration of suspended solids is determined and the percentage of material removed or settled may be computed for each sample taken. The values are plotted as numbers in a depth versus time plot (Zanoni and Blomquist, 1975). Through those numbers iso-concentration or equal concentration lines may be drawn. For design and comparison purposes the overall percentage suspended solid settled versus time could be obtained from the iso-concentration plot.

To simplify the procedure, other methods only require the computation of the average suspended solids concentration for the whole column at a particular time and assume it to be the overall percentage suspended solids settled. Some studies (Krishnam, 1976) compared the two methods, they found no particular difference between the two obtained curves. In fact, they recommend the use of the second method over the first since it is faster and the same accuracy was obtained. For illustration purposes a typical settling column, and an iso-percentage plot are shown in Fig. (3.2). An overall percentage suspended solids removed versus time is shown in Fig. (3.3).

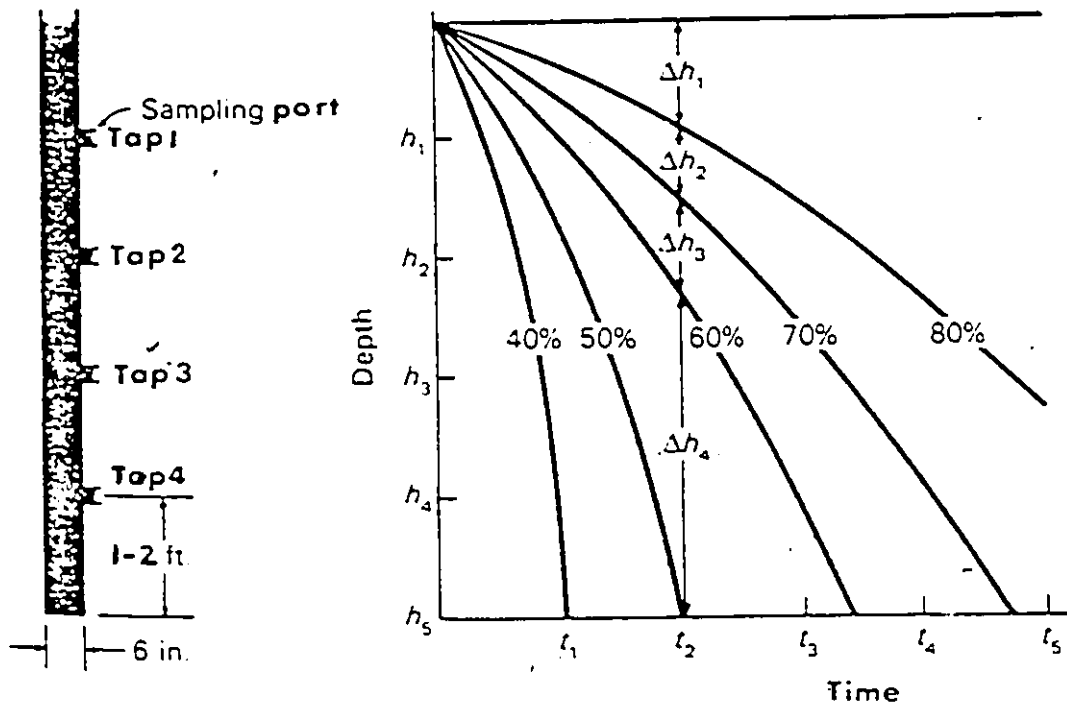


Figure 3.2 : Settling Column and Iso-Percentage Plot (after Metcalf and Eddy, 1972)

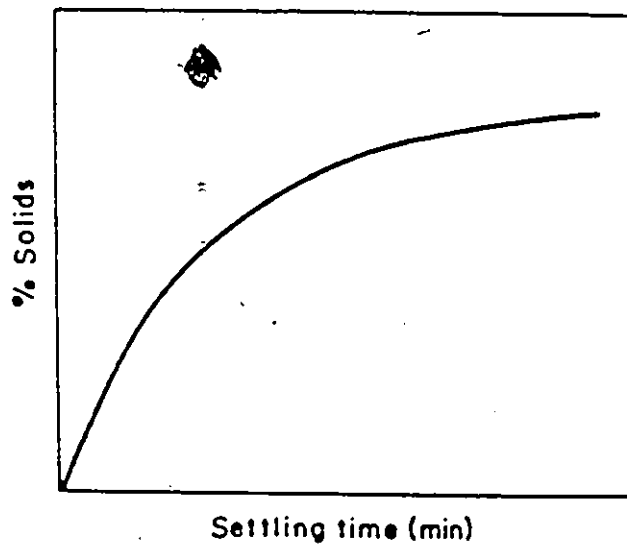


Figure 3.3 : Typical Plot of % SS Settled vs. Time

3.1.4 Zone and Compression Settling

Zone and compression settling usually occur in suspensions which contain high concentrations of suspended solids in the lower levels of the sedimentation basin. The free settling particles in the upper level of the basin reach the sludge zone in which the particles decelerate and become a part of the sludge blanket (also called hindered settling). In this region particles act as a group and settle as a blanket. The settling rate of the blanket was estimated to have the following expression (Steel and McGhee, 1979):

$$\frac{V_h}{V_s} = (1 - C_v)^z \quad (3.19)$$

in which V_h = hindered settling velocity
 V_s = free setting velocity as calculated from Eqs. (3.6) or (3.8)
 C_v = volume of particles divided by the total volume of the suspension

A typical value of z is 4.65. Eq. (3.19) is valid only for R_N less than 0.2.

Particles in the compression region are part of a structure in which complete physical contact exists. Consolidation of the sludge takes place and the rate of settlement decreases dramatically. The consolidation rate in this region may be approximated by (Clark, 1977):

$$\frac{dH}{dt} = -i(H - H_f) \quad (3.20)$$

where H = sludge depth at time t
 H_f = final sludge depth
 i = constant characterizing the suspension

To obtain the required time for the sludge layer to drop from a critical height H_c to H , Eq. (3.20) could be integrated, which results in the following expression:

$$t = t_c + \frac{1}{i} \ln \left[\frac{H_c - H_p}{H - H_p} \right] \quad (3.21)$$

where H_c = initial height of the compacted zone
at time t_c

The stirring of the sludge at the bottom of the tank may accelerate compaction and allows water to escape.

During zone and compression settling the interface height of the sludge blanket had been found experimentally to follow the curve shown in Fig. (3.4).

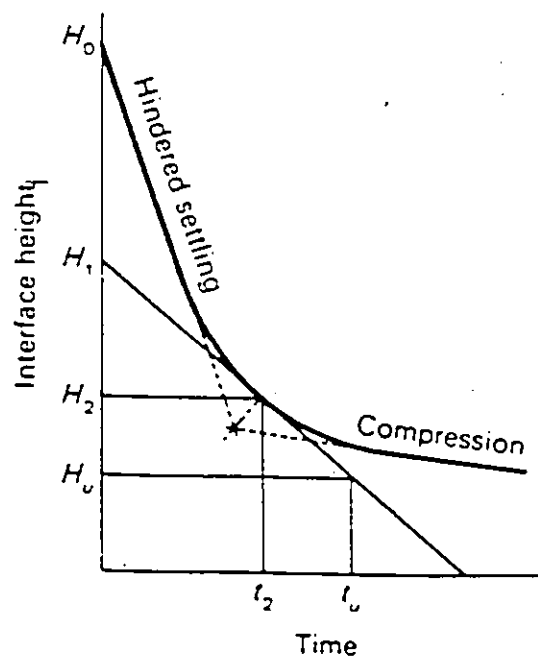


Figure 3.4 : Typical Plot of Interface Settling Curve
(after Metcalf and Eddy, 1972)

3.2 Mass Balance Equation

3.2.1 Introduction

A treatment system could be designed effectively if the mass balance equation is applied correctly within the proposed system. Given the desired level of treatment to be achieved, the mass balance equation is applied to find the volume of a reactor or treatment basin. It is also used to estimate the quantity of a substance entering, leaving or accumulating in the system within a specified time. The analysis could be applied to rivers, lakes or any treatment basin where the problem is to find the concentration of a substance at a particular location or its rate of change in a section. To apply the mass balance equation it is important to know the rate at which the studied substance is consumed or generated within the system. The substance could be physical such as solid particles, a chemical product or microbes.

In this section an overview of the mass balance equation will be discussed. Then it will be applied to the two extreme ideal cases of a completely mixed tank and a plug flow reactor, as well as a non-ideal case. Generally, it is acceptable to assume a steady state situation prevailing in the system. However, if the rate of accumulation of a particular substance is to be determined in the system, the steady state situation cannot be assumed.

3.2.2 Set Up of Mass Balance

A substance enters a given volume, known as a control volume, by bulk flow or other hydraulic phenomena such as diffusion and turbulence. This substance may be produced or consumed depending on the type of substance and the time spent in the system. This is known as the reaction of the substance and may be caused by chemical, biological or physical phenomena. The substance can eventually leave the system. Some of this substance may neither react nor leave the system. It eventually accumulates in the system.

The general unsteady state mass balance equation for a given substance j around the control volume is:

$$\left[\begin{array}{l} \text{Rate of flow of a} \\ \text{substance } j \text{ into} \\ \text{the control volume} \end{array} \right] - \left[\begin{array}{l} \text{Rate of flow of a} \\ \text{substance } j \text{ out of} \\ \text{the control volume} \end{array} \right] + \left[\begin{array}{l} \text{Rate of generation} \\ \text{of a substance } j \text{ in} \\ \text{the control volume} \end{array} \right] = \left[\begin{array}{l} \text{Rate of accumulation} \\ \text{of a substance } j \text{ in} \\ \text{the control volume} \end{array} \right] \quad (3.22)$$

or simply

$$\text{In} - \text{Out} + \text{Generation} = \text{Accumulation} \quad (3.23)$$

In Eq. (3.23) in - out refers to the net transport, generation refers to the net amount of reaction, and the accumulation is what is leftover. In the case of a steady state situation, the accumulation term in Eq. (3.23) is equal to zero. The assumption of steady state means that the concentration of a substance j is constant in each location in the system and does not change with time.

The rate of reaction, r , to be used in Eq. (3.23) is usually assumed to have the following form:

$$r = \pm K C^n \quad (3.24)$$

where K = rate constant
 C = concentration of the substance j
 in the tank (mg/L or g/L)
 n = a constant

r , has the unit mass/volume/time regardless of the value of n . The units of K are adjusted to make Eq. (3.24) dimensionally consistent. A positive r corresponds to a production, and a negative r corresponds to a destruction or consumption. The most commonly used form of Eq. (3.24) is first order (i.e., n is equal to 1).

The rate of reaction, r , depends on many factors such as the temperature, pH, the composition of the solution in which the reaction is taking place, etc. Unfortunately the rate expression cannot be predicted, it must be determined experimentally. In this section, the use of the rate expression in the mass balance equation is the primary concern. Therefore, the rate expression is assumed to be known and it will be combined with the mass balance equation.

3.2.3 Completely Mixed Tank

A completely mixed tank is also known as a continuous flow stirred tank reactor (Fig. 3.5). The mixing in the tank is assumed to be perfect. Thus the effluent concentration is the same as the tank concentration for a given substance j .

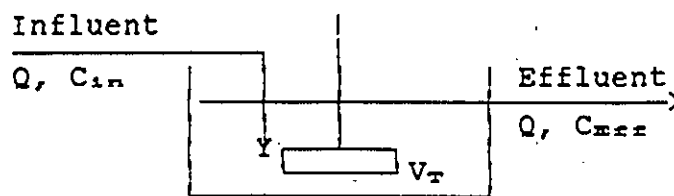


Figure 3.5 : Completely Mixed Tank

Taking the control volume as the whole volume of the tank Eq. (3.23) may be rewritten:

$$QC_{in} - QC_{eff} + rV_T = \frac{\partial C}{\partial t} V_T \quad (3.25)$$

where Q = volumetric flow rate (volume/time)
 C_{in} = influent concentration (mass/volume)
 C_{eff} = effluent concentration (mass/volume)
 r = rate of reaction (mass/volume/time)
 V_T = volume of the tank or reactor
 t = time

Let r be a first order decay (or consumption), i.e.:

$$r = -KC \quad (3.26)$$

Substituting Eq. (3.26) into Eq. (3.25) and rearranging the terms, the following equation is obtained:

$$\frac{Q}{V_T} C_{in} - \frac{Q}{V_T} C_{eff} - KC = \frac{dC}{dt} \quad (3.27)$$

For completely mixed tank C_{eff} is equal to C , the concentration in the tank. C_{in} in most practical cases is assumed to be constant.

The hydraulic detention time, t_d , is defined as the volume of the tank divided by the volumetric flow rate, or:

$$t_d = \frac{V_T}{Q} \quad (3.28)$$

Substituting Eq. (3.28) into Eq. (3.27) yields:

$$\frac{1}{t_a} C_{in} - \frac{1}{t_a} C - KC = \frac{dC}{dt} \quad (3.29)$$

If a steady state condition prevails around the system; i.e., dC/dt is equal to zero, then the concentration of a given substance j in the tank is a constant value, and it can be obtained from Eq. (3.29):

$$C = \frac{C_{in}}{(1 + Kt_a)} \quad (3.30)$$

In the case when dC/dt is not equal to zero, Eq. (3.29) becomes:

$$\frac{dC}{C_{in} - (1 + Kt_a)C} = \frac{1}{t_a} dt \quad (3.31)$$

Solving Eq. (3.31) with the initial condition $C = 0$ at $t = 0$; the tank concentration C is expressed in function of the time t as follows:

$$C = \frac{C_{in}}{(1 + Kt_a)} \left[1 - \exp\left[-\frac{(1 + Kt_a)}{t_a} t\right] \right] \quad (3.32)$$

For a given time t , the concentration, C , of a substance j in the tank can be obtained from Eq. (3.32) since C_{in} , K , and t_a are constants.

3.2.4 Plug Flow System

A plug flow reactor is also known as a piston-flow reactor. It is a simple tube. Through the inlet, the influent, containing the substance j , is fed continuously, while the

effluent is removed from the outlet. For ideal plug flow the velocity of the flow inside the tube is assumed to be constant and no longitudinal mixing of the substance j is allowed. Therefore, the concentration of the substance j varies only with the axial distance, x . As shown in Fig. (3.6) the appropriate control volume could be an infinitesimal value ΔV , in which the concentration may be considered uniform.

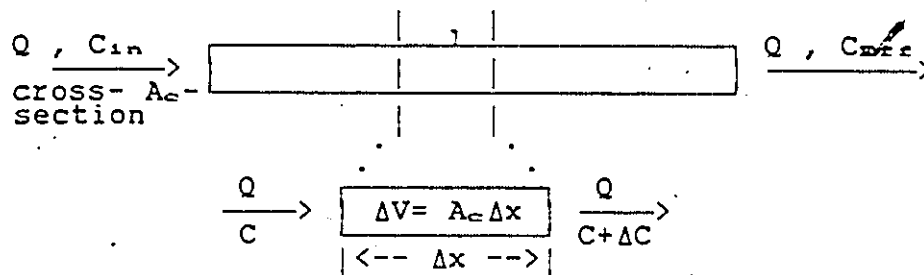


Figure 3.6 : Plug Flow Reactor

The mass balance equation on the substance j around the control volume is written as follows:

$$QC|_x - QC|_{x+\Delta x} + r\Delta V = \frac{\partial C}{\partial t} \Delta V \quad (3.33)$$

or:

$$-Q\Delta C + rA_c\Delta x = \frac{\partial C}{\partial t} A_c\Delta x \quad (3.34)$$

Eq. (3.34) expresses C as a function of x and t . The equation can be solved if the initial conditions are known. Eq. (3.34) states that at every location x the concentration of the substance j is changing with time.

In the case of steady state Eq. (3.34) reduces to:

$$-\frac{Q}{A_c} \frac{dC}{C} = K dx \quad (3.35)$$

Solving Eq. (3.35) with the boundary condition $C = C_{in}$ at $x = 0$ the general expression of C is obtained:

$$C = C_{in} \exp \left[-\frac{A_c}{Q} Kx \right] \quad (3.36)$$

If the flow rate, Q , and the cross-sectional area, A_c , are assumed to be constants, then the velocity, v_x , of the flow is also constant. Under these conditions Eq. (3.36) becomes:

$$C = C_{in} \exp \left[-\frac{K}{v_x} x \right] \quad (3.37)$$

3.2.5 Case When the System is Non-Ideal

In previous sections the mass balance equation was applied to two ideal systems, i.e., completely mixed and plug flow systems. Real treatment systems, however, are neither completely mixed tanks nor plug flow reactors. In fact their behaviors lie between the above described two ideal extremes. For instance, the effluent or the outflow concentration may depend on many factors such as the quantity of the flow passing through the system, the geometry and the concentration in the tank, the type of the substance in question, etc.

Assuming the effluent concentration is only a function of the tank concentration, i.e.:

$$C_{eff} = f(C) \quad (3.38)$$

Substituting Eq. (3.38) into Eq. (3.27) gives:

$$\frac{1}{t_a} C_{in} - \frac{1}{t_a} f(C) - KC = \frac{dC}{dt} \quad (3.39)$$

Depending on the type and complexity of the function $f(C)$ Eq. (3.39) may be solved numerically if an exact solution cannot be obtained.

Once the function $f(C)$ is known and Eq. (3.39) is solved for C , the accumulation of a given substance j in the system within a given period of time can be obtained.

CHAPTER 4

DESIGN AND EXPERIMENTAL PROCEDURE

4.1 APPARATUS AND EXPERIMENTAL SET UP

4.1.1 Design of the Interceptor Tank

The tank that precedes the small-bore sewers in the system is smaller in size than a typical septic tank. In the old design of onsite disposal systems, the septic tank attempted to capture all suspended solids. However, in the new concept of small-bore sewerage, the interceptor tank allows the lightweight solids to escape to extend desludging intervals. Since the rate of accumulation of sludge in the interceptor tank is expected to be lower than that for a septic tank, the volume of the former can be reduced.

In the design of the interceptor tank, the following factors were considered:

- The number of people the tank will serve,
- The expected levels of water used,
- The hydraulic retention time (or detention time),
- The rate at which sludge is expected to accumulate, and
- The frequency of sludge removal from the tank.

Since the tank design was oriented toward developing countries, the assumed number of people in a single dwelling

is 5 to 6 persons. However, in many areas of the world this number could be significantly higher. Assuming an adequate piped water supply, the average volume of wastewater was estimated from the literature to be 170 L/c/d. For the purpose of this research a nominal detention time of 24 hours was selected (Laak, 1972), which is calculated on the basis of wastewater flow per interceptor tank capacity.

Solids build up in septic tanks is assumed to be 0.03-0.04 m³/c/y (Kalbermatten et al., 1980a). For the purpose of designing the interceptor tank, the tank is assumed to be desludged every two years. The volume of the septic tank designed under the above conditions is calculated by adding the volume of wastewater generated per day and the volume of sludge accumulated during two years. Note here that no provisions were made for scum accumulation and garbage grinders. The real rate of accumulation of sludge and the frequency of desludging of interceptor tank are two of the main objectives, which will be estimated at the end of this research. The sludge capacity of small-bore sewer interceptor tanks will be dictated by the allowable solids concentration in the effluent. One of the objectives of this research is to determine the tank sludge capacity for different geometries and other factors based on different effluent suspended solids concentration criteria.

To minimize the effect of model scaling, a full size model was constructed. A cylindrical tank was designed with 1.2 m diameter, and an effective volume of 1.58 m³, which is the

total liquid volume below the outlet pipe. The inlet pipe is located in the center and the outlet pipe is located at 1.4 m (effective depth) from the bottom of the tank. The supply reservoir was located 1 m above the tank (see Fig. (4.1)).

The main factors thought to cause the disturbance of solids inside the interceptor tank were:

- Volume and time-pattern of water flushed from the supply reservoir into the tank (assigned the symbol V),
- Distance of the inlet pipe from the bottom of the tank (D),
- Slope of the bottom surface of the tank (S), and
- Extended length of the outlet pipe (L).

These dimensions and characteristics of the tank are illustrated in Fig. (4.1). Each of the above variables was tested at three different levels. Table (4.1) summarizes these levels.

Table 4.1 : Proposed Testing Levels

Slope, S	flat	1:2	1:1
Volume, V (L)	15	30	45
Distance, D (m)	0.1	0.3	0.5
Length, L (m)	0.1	0.3	0.5

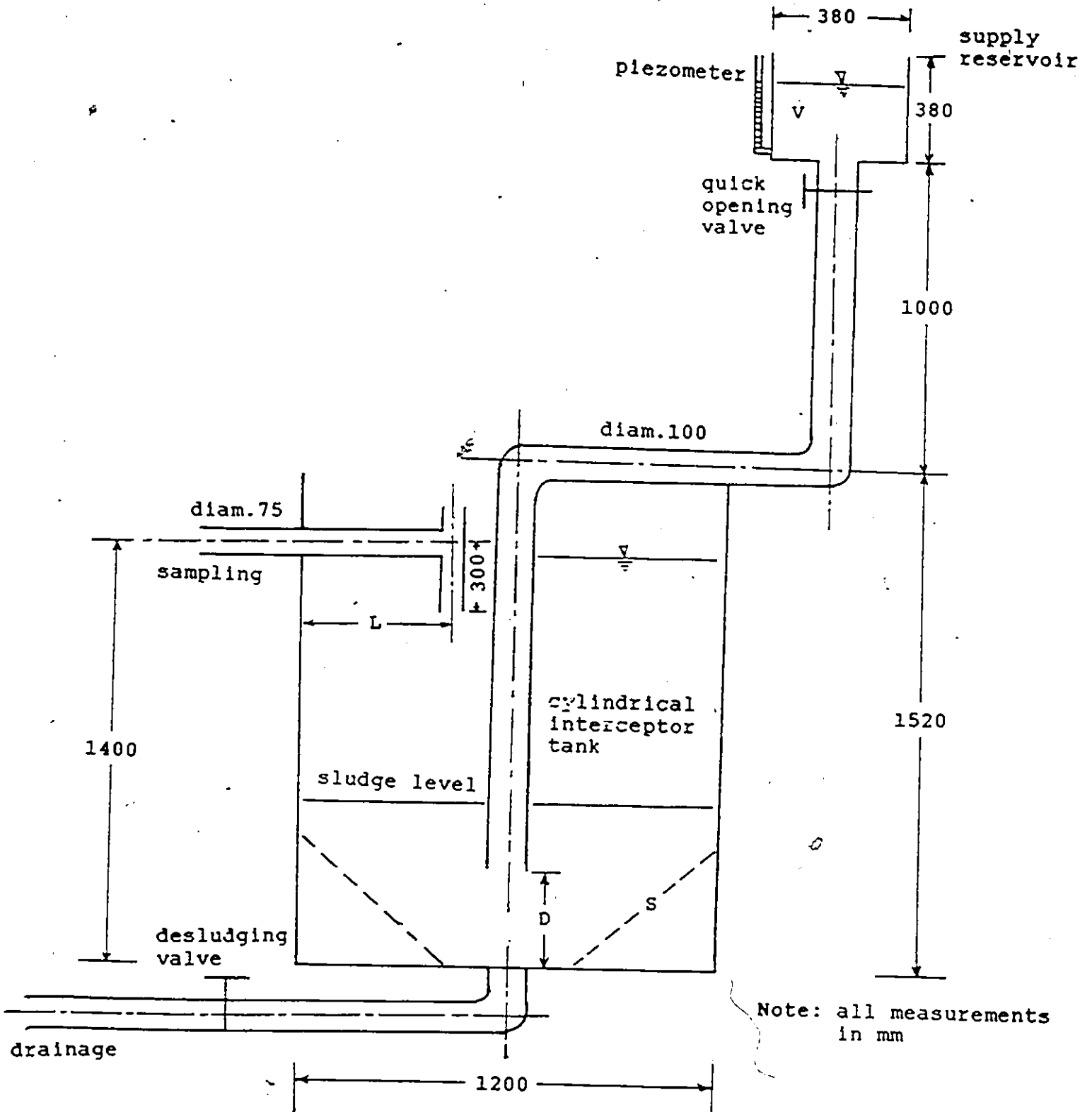


Figure 4.1 : Full Scale Interceptor Tank Model

4.1.2 Simulation of Domestic Wastewater

Raw wastewater is highly variable in nature. The quality of solids in sewage differs from place to place in terms of shape, size, specific gravity and settling characteristics. Therefore, it is not possible to define with precision the characteristics that would apply for all domestic wastewaters. However, it is important to know that particles in wastewater have different settling velocities. Based on this fact, a sludge that satisfies this condition can be used in laboratory or experimental work to approximate the actual wastewater.

A simulated domestic wastewater was used to produce sludge for the experimental study, consisting of organic matter (OM) composed of ground corn, wheat shorts, soybean and animal fat (i.e., dog food). The characteristics of this simulated wastewater are close to typical wastewater. The settling characteristics of the simulated material were tested (refer to Section (4.2.1)), and its relative density was found to be 1.13 (the relative density of OM in wastewater varied between 1.00 to 1.20). The moisture content and the soluble solids in the actual OM represented 17% of the total mass. The simulated OM had particle sizes varying from several μm to 1.18 mm and when mixed with water, 0.83 kg (1 kg of starting material) of OM occupied a volume of 7 L as a result of natural settling. As an example, to fill half the

tank, a volume of approximately 795 L of sludge is needed, which is equivalent to 94 kg of OM.

Experiments were done with different sludge volumes in the tank because it was recognized that the amount of sludge would have a significant influence on turbulent energy dissipation and solids escape. Table (4.2) illustrates the masses of OM (after eliminating moisture content and dissolved solids) needed for different volumes of sludge. In the later chapters the OM will be referred to as total suspended solids (TSS). In this table the nominal solids concentration was based on the mass of solids divided by the volume of liquid in the tank.

Table 4.2 : Mass of OM for Different Sludge Volumes

Mass of OM (kg)	20	40	60	80	100	120	140
Sludge volume (L)	169	337	506	675	843	1012	1180
% of tank volume filled with OM	11	21	32	43	53	64	75
Conc. of OM (g/L)	12.7	25.3	38.0	50.6	63.3	75.9	88.6

4.2 PRELIMINARY MEASUREMENTS

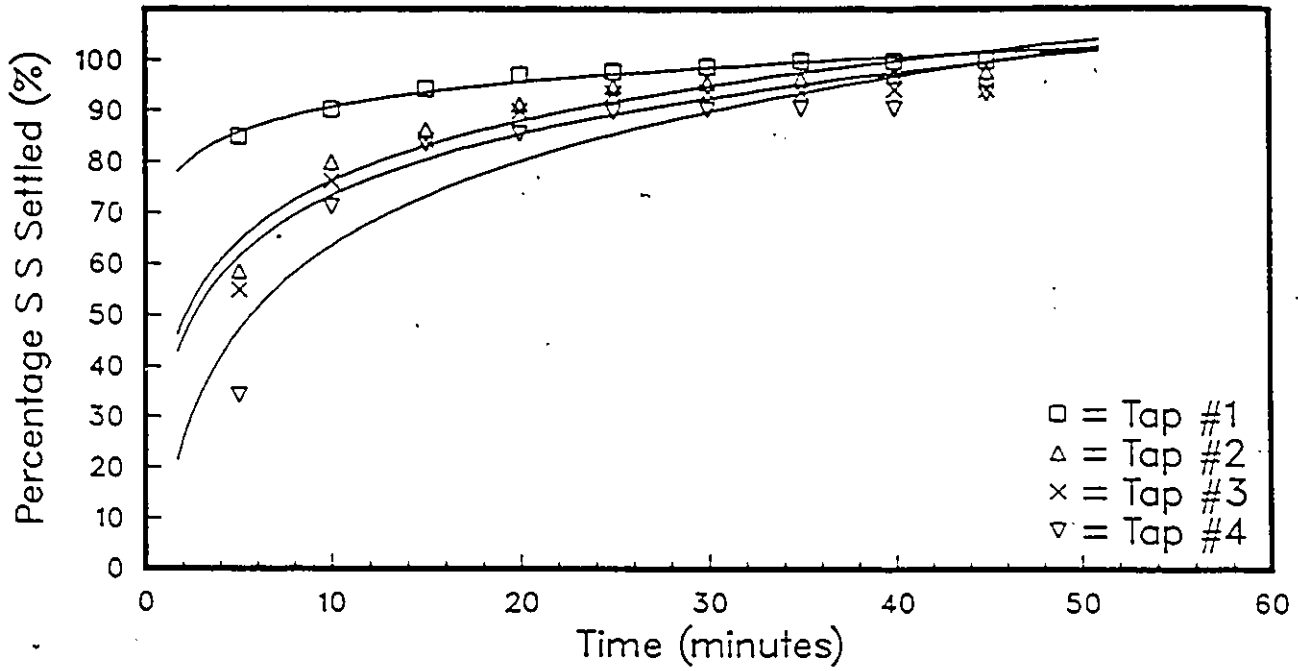
4.2.1 Settling Characteristics of the OM

The simulated domestic wastewater was tested in the laboratory to determine its settling characteristics. For this purpose a settling column made of plexiglass was used. The height of the column was selected to be the same height as

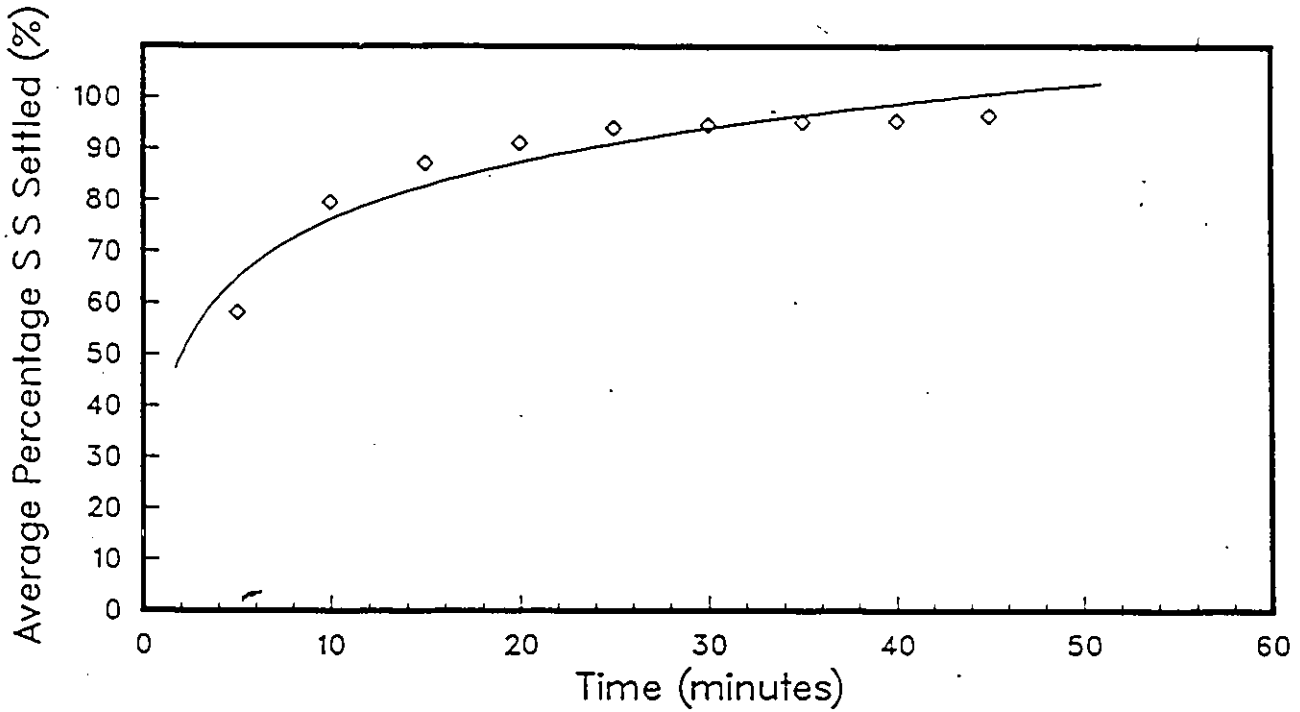
the proposed tank, which was 1.52 m (5 ft). Its diameter was 15 cm (6 in). Four taps located at 30 cm (1 foot) intervals were provided for sample withdrawals.

The settling test procedure consisted of diluting a certain amount of the OM in the column already filled with water. The temperature of the water was maintained between 19 and 21 °C. A mixing device and a pump for recycling the solution were used to ensure a uniform concentration of solids throughout the column. After approximately half an hour of mixing and recycling the suspension, samples were taken from each tap for initial SS determination. Then the mixing device and the pump were turned off and the timing was started. At selected times, samples were taken from each tap.

The standard procedure (APHA, 1980) was followed for filtering the solutions, drying and weighing the filters. The SS concentration for each solution was then determined. The percentage SS settled was calculated and plotted versus time for each tap as shown in Fig. (4.2a). The average SS settled for the whole column was also calculated and then plotted versus time as shown in Fig. (4.2b). It can be seen from this figure that almost 90% of the solids in the suspension settle within the first 20 minutes of the settling test period.



(a) Percentage S S Settled vs Time.



(b) Average Percentage S S Settled vs Time.

Figure 4.2 : Settling Column Test of the Simulated Domestic O M.

4.2.2 Measurement of the Outlet Flow Rate

4.2.2.1 Calibration of the Supply Reservoir

The reservoir that supplied the interceptor tank with water had a piezometer mounted on its side. The calibration of the reservoir was carried out as follows. For different volumes of water in the reservoir, the corresponding readings of the piezometer (see Fig. (4.1)) were recorded and then plotted as piezometer reading versus water volume in the supply reservoir as shown in Fig. (4.3). This plot was used when different volumes of water in the supply reservoir were needed to run the experiment.

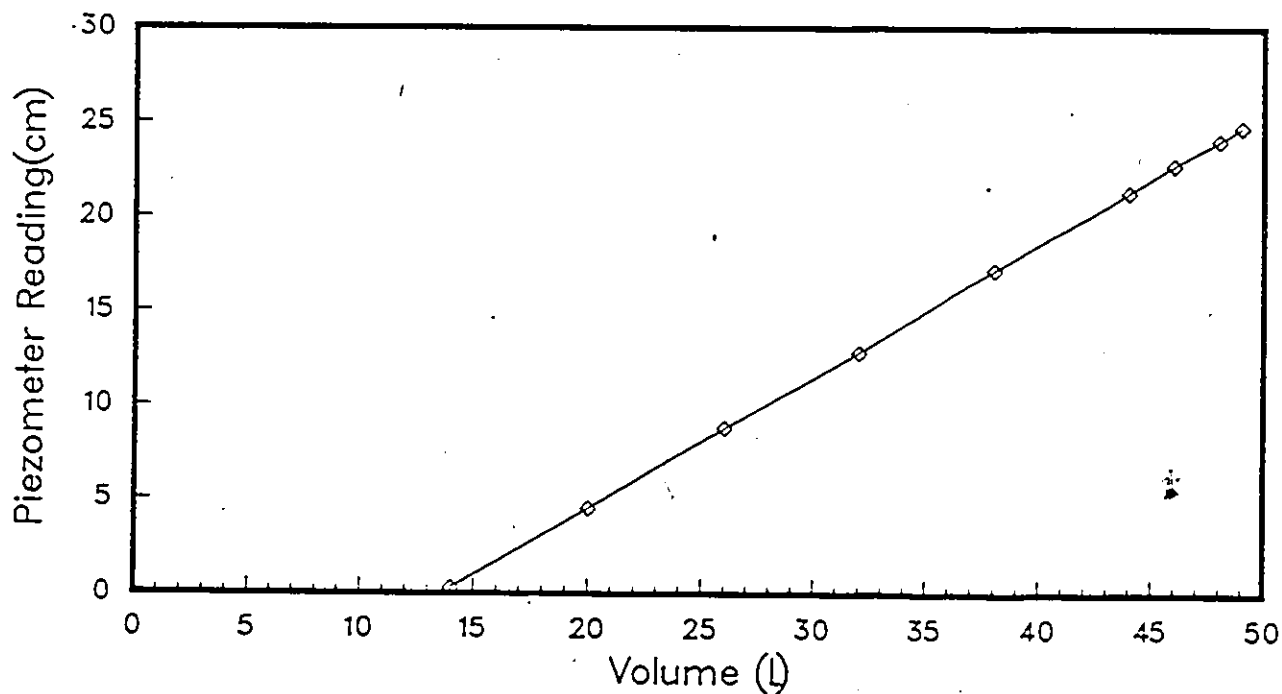


Figure 4.3: Supply Reservoir Calibration Curve

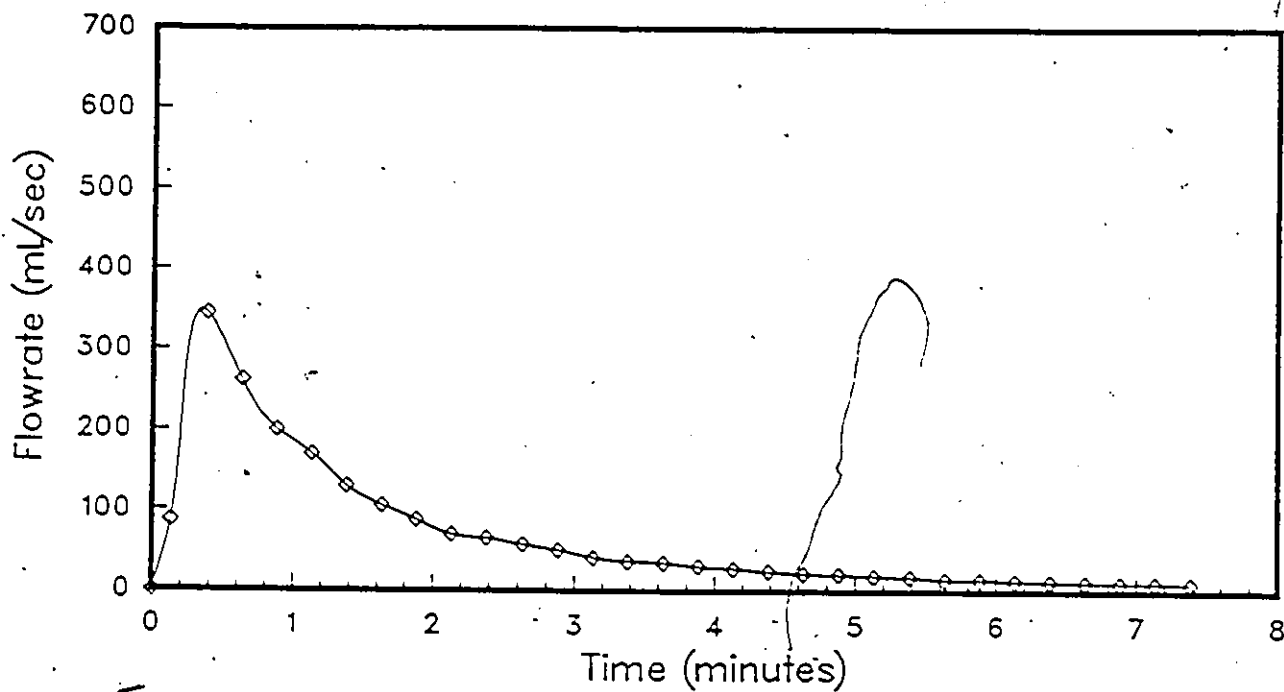
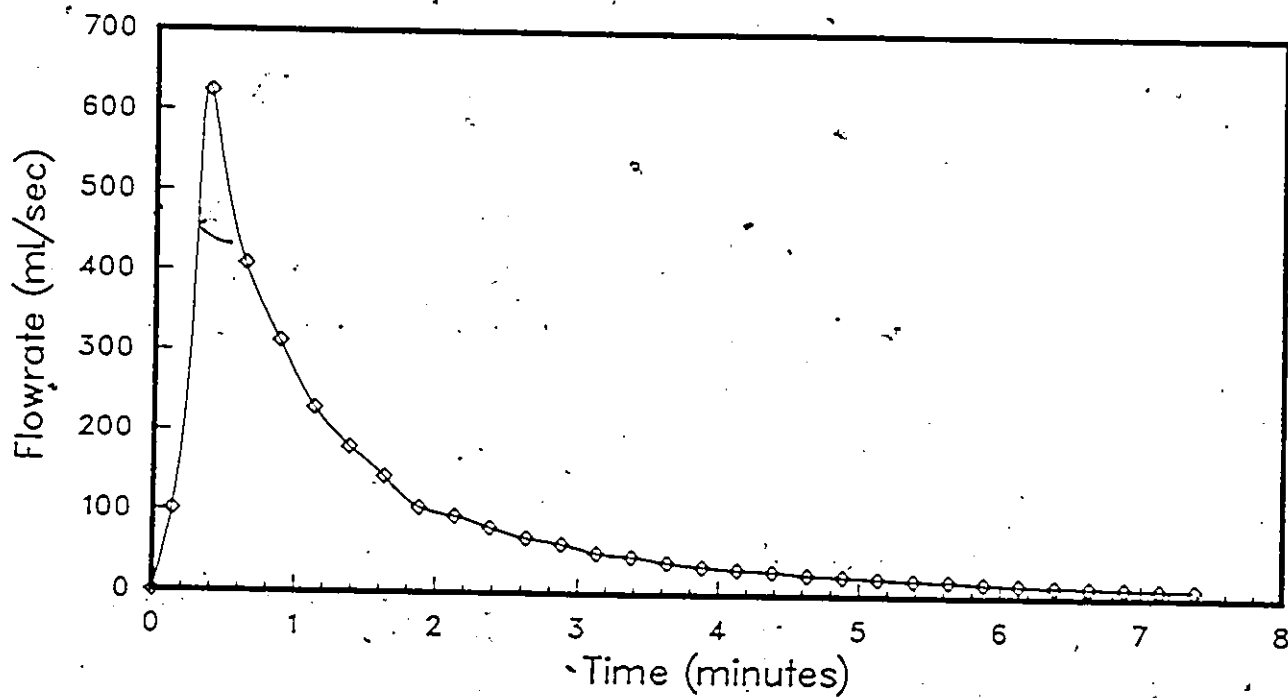
4.2.2.2 Outlet Flow Rate

As mentioned before, one of the functions of the interceptor tank is to attenuate surges and peak flows coming from dwellings. If a large volume of wastewater is released suddenly into the tank, it will be released into the small-bore sewers over a longer period of time.

The determination of the outlet flow rate was carried out several times as follows. A known volume in the supply reservoir was flushed into the tank. At different times, samples from the outlet were collected during a fixed period of time. Knowing the volumes and times required for collection, the flow rates were calculated. The flow rates are plotted versus time for volumes of 30 and 45 L in the supply reservoir in Figs. (4.4a) and (4.4b), respectively. Only a few seconds were required to flush the volumes from the supply reservoir. However, as it can be seen from the previous figures, it required more than seven minutes to release completely the flushed volumes into the small-bore sewers.

4.2.3 Measurement of the Short Circuiting

Short circuiting can be defined as the direct passage of fluid particles or other solid particles straight from the supply reservoir into the outlet pipe. Theoretically, the fluid particles flushed from the supply reservoir are expected to stay in the tank for a period of 24 hours if the

Figure 4.4a: Flowrate Hydrograph $V=30$ L.Figure 4.4b: Flowrate Hydrograph $V=45$ L.

hydraulic detention time is one day. However, in reality this statement is not true. To measure the short circuiting experimentally, tracer studies were performed.

4.2.3.1 Calibration of the Fluorometer

The fluorometer was calibrated in the laboratory to obtain the relationship between dial reading and dye concentration. For this purpose different solutions with different known concentrations of dye were prepared and then the corresponding fluorometer readings were recorded. The results were plotted in Fig. (4.5).

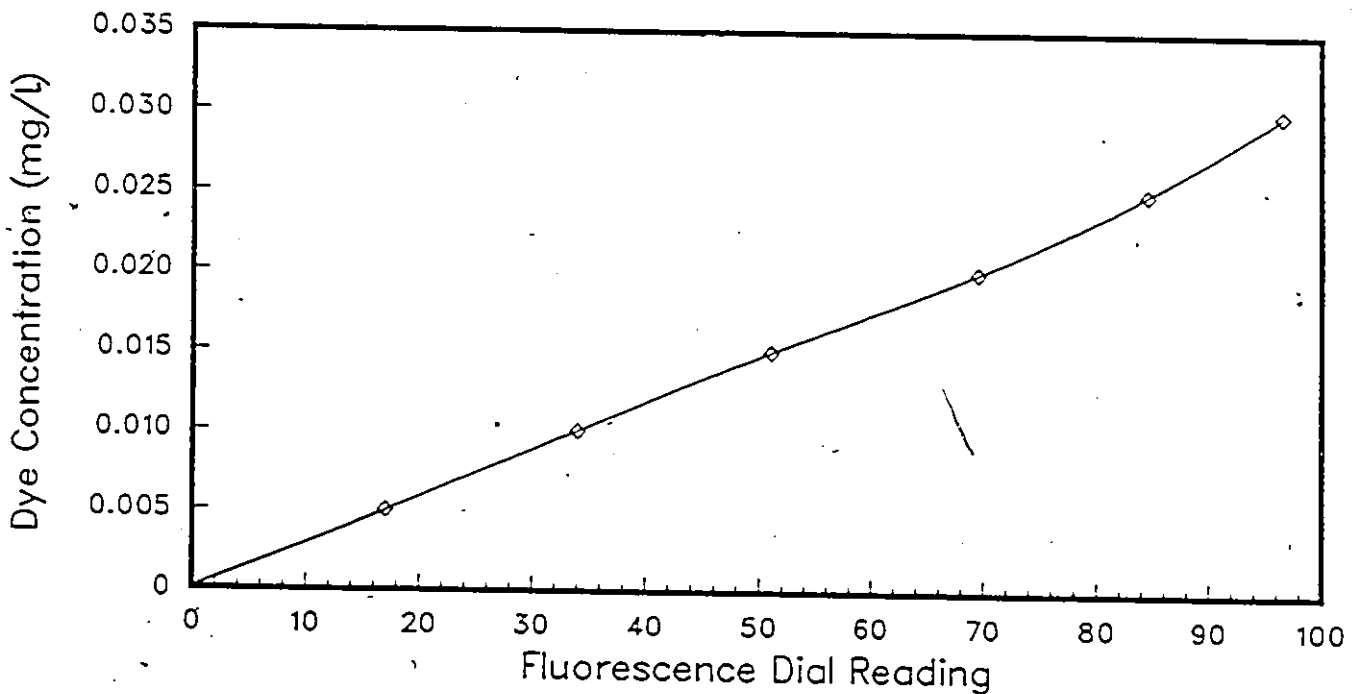


Figure 4.5 : Fluorometer Calibration Curve

4.2.3.2 Determination of the Amount of Short Circuiting

Short circuiting was measured by placing a known amount of dye in the supply reservoir containing a known volume of water. The water was then flushed into the tank. At the outlet, a fluorometer connected to a plotter recorded the fluorometer dial readings and the time. The dial readings were converted into concentrations using the fluorometer calibration curve. Knowing the outlet flow rate and the dye concentration leaving the tank, the total amount of dye escaping the tank was calculated. The experiment was carried out twice, once with a volume of 30 L and once with a volume of 45 L. Larger flush volumes were chosen because they would create more mixing and thus be more likely to cause loss of some influent immediately into the effluent. A solution of 100 mL containing 10^5 μg (0.1 g) of dye was placed in the supply reservoir. The whole volume was then flushed into the tank. The plots generated from the fluorometer are shown in Figs. (4.6a) and (4.6b) for 30 and 45 L, respectively. The total amounts of dye escaping the interceptor tank were estimated from Figs. (4.6a) and (4.6b), using the fluorometer calibration curve (Fig. (4.5)), and the flow rate hydrographs (Figs. (4.4a) and (4.4b)). The amount of short circuiting was estimated to be 500 and 450 μg (less than 1%) for flushing volumes of 30 and 45 L, respectively.

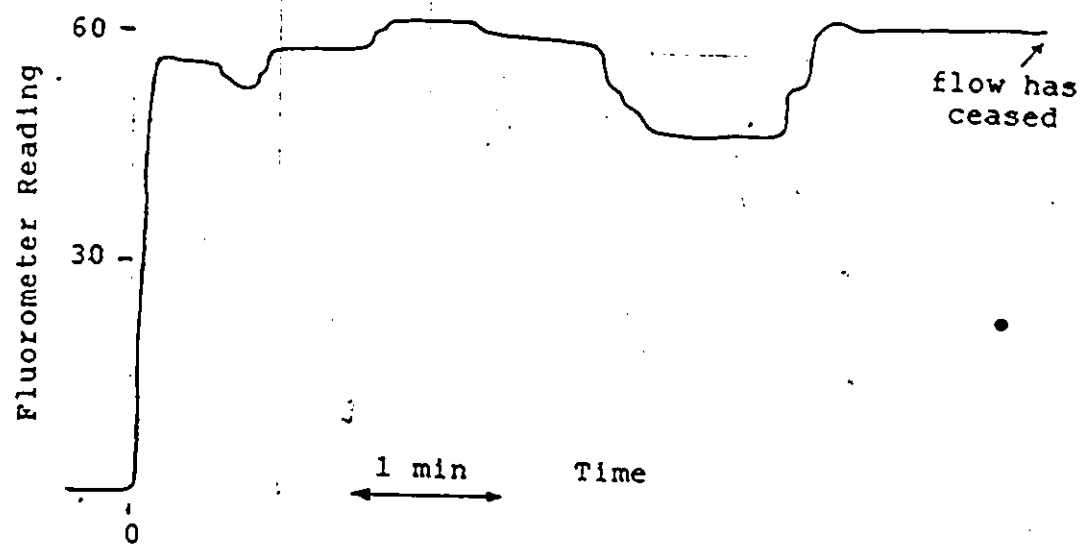


Figure 4.6a : Dye Outlet Concentration $V = 30$ L

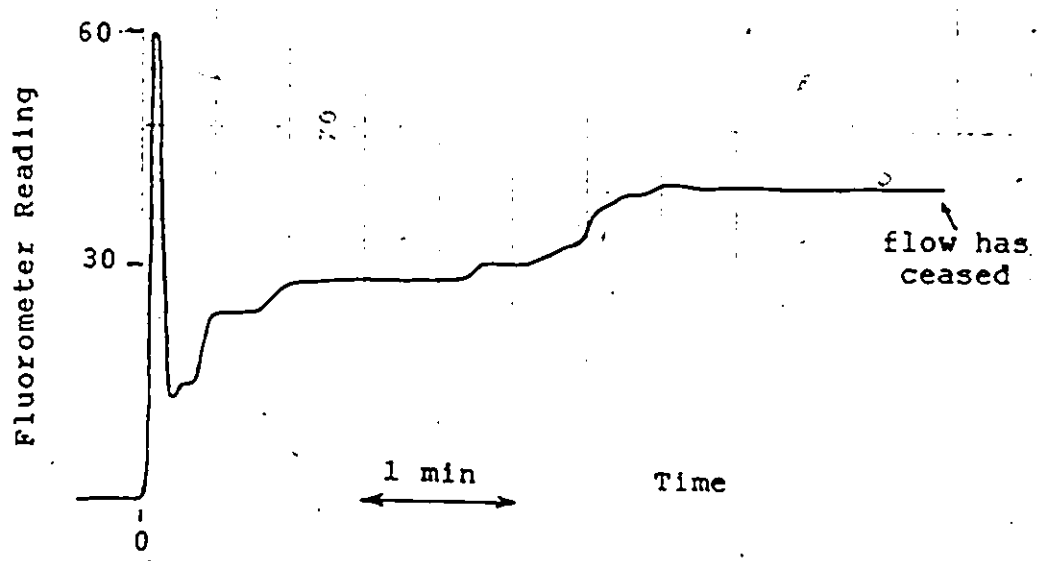


Figure 4.6b : Dye Outlet Concentration $V = 45$ L

4.3 EXPERIMENTAL DESIGN

As discussed in Section (4.1.1), four variables at three levels each (refer to Table (4.1)) were tested experimentally. The total number of possible combinations of factors (or variables) at all levels is 81 (4 factors at 3 levels corresponds to 3^4 combinations). It is not necessary, however, to carry out studies of all combinations. A fractional factorial design was used to select and perform only 27 combinations (i.e., one third fractional replication was used). The experimental design allows interactions among all variables to be assessed (refer to Section (4.3.2)) at a considerable savings of time and materials. The choice of the combinations to be tested and their notations are discussed in the following section.

4.3.1 Fractional Factorial Design

The purpose of this section is to present a brief discussion of the fractional factorial design and to apply it to this experiment.

When only a sub-group (or a fraction) of all possible combinations in a complete factorial design is selected, it is obvious that some information is lost. However, it is desirable to select a sub-group in such a way that important pieces of information are retained even though some unimportant pieces of information are lost.

Fractional factorial experimental designs are well documented (Bacon, 1980; and 1970; Box and Behnken, 1960) for different designs involving 2, 3, ..., 10 factors at 2 and 3 levels. The fractional factorial design of 4 factors at 3 levels was obtained from Box and Behnken (1960). Before presenting the design some notation should be explained.

Instead of using the actual values of the operating variables, greater numerical accuracy and simplification may be achieved by using coded values. Because each variable has only three equally spaced values, the three coded values were selected as -1, 0, and 1, corresponding to the low, middle, and high values, respectively. The coded variables are represented by X_1 , X_2 , X_3 , and X_4 corresponding to S, V, D, and L respectively. The values in Table (4.1) were transformed into coded values shown in Table (4.3).

Table 4.3 : Coded Variables

X_1	-1	0	1
X_2	-1	0	1
X_3	-1	0	1
X_4	-1	0	1

The fractional factorial design for the 4 operating variables is presented in Table (4.4a).

The first column in the above design (i.e., $X_1 = \pm 1$, $X_2 = \pm 1$, $X_3 = 0$, $X_4 = 0$) represents four experimental runs as shown in Table (4.4b).

Table 4.4a : Fractional Factorial Design

X_1	± 1	0	± 1	0	± 1	0	0
X_2	± 1	0	0	± 1	0	± 1	0
X_3	0	± 1	0	± 1	± 1	0	0
X_4	0	± 1	± 1	0	0	± 1	0
# of runs	4	4	4	4	4	4	3

Table 4.4b : Sample of the Four Possible Runs

X_1	-1	1	-1	1
X_2	-1	-1	1	1
X_3	0	0	0	0
X_4	0	0	0	0

The sixth column represents three runs at the center point of the design (i.e., each variable is assigned its middle value). The total number of runs to be tested experimentally is then 27.

4.3.2 Choice of the Mathematical Model

To describe and predict the outlet (e.g., effluent) SS for any combination of variables, a mathematical model could be obtained from the fitted experimental data. Often, this mathematical model can be either a first or second-degree polynomial model. For designs involving more than two levels for each variable, it is most appropriate to assume a sec-

second-degree polynomial model. The following model was proposed to describe the SS concentration in the interceptor tank effluent. This model was written in terms of the coded variables.

$$\begin{aligned}
 C_{\text{SS}} = & B_0 + B_1X_1 + B_2X_2 + B_3X_3 + B_4X_4 + \\
 & B_{11}X_1^2 + B_{22}X_2^2 + B_{33}X_3^2 + B_{44}X_4^2 + \\
 & B_{12}X_1X_2 + B_{13}X_1X_3 + B_{14}X_1X_4 + B_{23}X_2X_3 + \\
 & B_{24}X_2X_4 + B_{34}X_3X_4
 \end{aligned} \tag{4.1}$$

where C_{SS} = effluent SS Concentration
 X_1 = coded variable for the slope S
 X_2 = coded variable for the volume V
 X_3 = coded variable for the distance D
 X_4 = coded variable for the length L

The experimental data are fitted to the above model from which the 15 constants are estimated using the least squares method. After the constants (also known as coefficients) have been determined, the model may be used to predict the outlet (effluent) SS for the combinations of variables which have not been performed experimentally.

4.4 EXPERIMENTAL PROCEDURE

The following sections describe in detail the various procedures involved in carrying out the research.

4.4.1 Preparation of the Filters

The filters used in this study had 11.9cm diameters and were able to retain solid particles having sizes greater than 1.5 μm . Before they were used, the filters were cleaned with distilled water and dried at a temperature of 100-105

°C for at least one hour. After the drying was completed the filters and the aluminum dishes (for holding the filters) were placed in a dessicator for at least 20 min for cooling. Moisture in the dessicator was kept low by using calcium sulfate crystals. The initial weights of the filters and the aluminum dishes were then measured using an electronic balance having a precision of 0.1 mg. The filters could then be used for filtering the samples.

4.4.2 Preparation of the OM

The OM used to simulate domestic wastewater was placed in water the night before an experiment in buckets, after which it was well mixed and then dumped into the interceptor tank. An increment of 20 kg of OM that occupied almost 11% of the tank volume was added to the tank for each set of runs (to go from one sludge level to another, 20 kg of OM is required). To stop any biological activity that might occur in the tank, a few mL of bleach were periodically added to the OM.

4.4.3 Procedure

The 27 runs (each run corresponds to one combination of variables) selected previously in Section (4.3.1) were performed at 7 sludge levels presented in Table (4.2). For convenience the 27 runs were grouped into three sets. From Table (4.4) the three sets were rearranged according to the

bottom slope of the tank (S). This is presented in Tables (4.5a-c):

Table 4.5a : Experimental Runs (S = flat)

V (L)	D (m)	L (m)
30	0.5	0.3
30	0.1	0.3
15	0.3	0.3
45	0.3	0.3
30	0.3	0.1
30	0.3	0.5

Table 4.5b : Experimental Runs (S = 1:2)

V (L)	D (m)	L (m)
30	0.1	0.1
15	0.1	0.3
45	0.1	0.3
30	0.1	0.5
15	0.3	0.5
45	0.3	0.5
15	0.3	0.1
45	0.3	0.1
30	0.3	0.3
15	0.5	0.3
45	0.5	0.3
30	0.5	0.1
30	0.5	0.5
30	0.3	0.3
30	0.3	0.3

Table 4.5c : Experimental Runs (S = 1:1)

V (L)	D (m)	L (m)
30	0.5	0.3
30	0.1	0.3
15	0.3	0.3
45	0.3	0.3
30	0.3	0.1
30	0.3	0.5

The general procedure consisted of placing a known quantity of OM in the interceptor tank, and then a known volume of clear water from the supply reservoir was flushed. The flow from the outlet pipe was collected in a 50 L container. The effluent was then well mixed and two sample solutions of about 300 mL each were taken. After about 30 minutes the following run for different V, D, and L was performed. The same procedure was repeated for each run.

Clear water was used in the supply reservoir. It was shown in Section (4.2.3.2) that the amount of short circuiting can be neglected and therefore, influent does not escape from the tank until a later flush. Therefore, only solids already in the tank will escape and there was no need to have solids in the influent, and no provision of placing OM in the supply reservoir (to simulate the actual flush from a toilet, sink, etc.) was taken into account. It was assumed that solids sent out in the effluent are only those which were in the tank, not those coming from the supply reservoir.

Thirty minutes elapsed between successive runs. This interval was based on Fig. (4.2), from which it was determined that 90% or more solids have settled after 30 min.

After the first set of runs (i.e., Table (4.5a)) was completed at all seven sludge levels, the tank was drained and a conical shape having a slope $S = 1:2$ was placed in the bottom of the tank. The same procedure with the 15 runs presented in Table (4.5b) was repeated as before. Finally the same procedure was done with $S = 1:1$.

4.4.3.1 Settling Column Tests:

The quality of the effluent was also tested for some selected combination of variables at selected sludge levels. The quality was tested in terms of settling characteristics of the particles washed out due to the turbulence created by the flushing action of water from the supply reservoir. The settling column test of the whole effluent was performed following the same procedure described in Section (4.2.1). Table (4.6) presents the selected combinations of variables and the selected sludge levels for which the effluent was tested.

To check the settling characteristics of the tank OM, the slope $S = \text{flat}$ and $S = 1:1$ at random sludge levels were selected. From each of the selected S and sludge level, a quantity of OM was taken from the tank and then diluted in the settling column already filled with water. The same procedure as described in Section (4.2.1) was followed. Five checks as shown in Table (4.7) were performed on the tank OM.

Table 4.6 : Selected Settling Tests of the Effluents

S	V (L)	L (m)	D (m)	sludge level (%)
flat	30	0.3	0.1	21
flat	30	0.3	0.1	43
flat	30	0.3	0.1	64
1:2	30	0.3	0.3	21
1:2	30	0.3	0.3	43
1:2	30	0.3	0.3	64
1:1	30	0.3	0.1	21
1:1	30	0.3	0.1	43
1:1	30	0.3	0.1	64

Table 4.7 : Selected Settling Tests of the Tank OM

S	sludge level (%)
flat	43
flat	64
1:1	21
1:1	43
1:1	64

CHAPTER 5

RESULTS AND DISCUSSION

As discussed in Chapter 4, this study was divided into two main parts: the first part was the determination of SS in the effluent flow for different combinations of variables. The second part was the testing of the settling characteristics of the effluent and the tank OM for some selected runs. The experimental data are presented in Table (5.1) for the first part, and the related graphs are presented in Appendix A. The data for the second part are tabulated in Appendix C, and the related figures are presented in Appendix D.

The data and the information generated by this study were entered into the computer and stored in a spreadsheet program. The extensive numerical data generated are summarized in tables in the following sections.

5.1 RESULTS AND ANALYSIS OF THE EFFLUENT SS

For the 27 runs performed experimentally the SS in the effluent flow were found to vary from one combination of variables to another, and from one sludge level to another. The reported values for the effluent SS were found to be as low as 30 mg/L, other values were as high as 4600 mg/L or even more. For particular arrangements of the inlet and the out-

let locations, an increase in sludge level in the tank did not necessarily lead to an increase in the effluent SS.

A summary of the experimental data for the 27 combinations of variables at all sludge levels is presented in Table (5.1). As mentioned in Chapter 4, the coded variables (i.e., X_1 , X_2 , X_3 , X_4 , corresponding to S, V, D, and L, respectively) have been used in Table (5.1). The level of sludge was expressed in percentage of the total volume of the tank. For each sludge level the corresponding SS in the tank is given in Table (4.2). The values presented in Table (5.1) were based on the average SS of two samples.

5.1.1 Determination of the Regression Coefficients

As discussed in Section (4.3.2), the selected mathematical (empirical) model was assumed to be a second-degree polynomial. The purpose of the present work was to find an expression of the effluent SS in terms of the selected variables.

To be able to predict the SS in the effluent, the 15 constants (coefficients) were determined by the method of least squares. A sample procedure is outlined in Appendix G for an 11 % sludge level in the tank. For each sludge level the coefficients were estimated, and hence a separate mathematical model was obtained. The 15 coefficients for the 7 sludge levels are summarized in Table (5.2). The unit of the coefficients in Table (5.2) is (mg/L).

Table 5.1 : Summary of the Experimental Data for C_{SS} (mg/L)

Coded Variables				Sludge level (%)						
X_1	X_2	X_3	X_4	11	21	32	43	53	64	75
-1	0	1	0	103	147	316	425	551	594	743
-1	0	-1	0	206	212	425	601	560	708	1025
-1	-1	0	0	26	158	273	247	445	497	379
-1	1	0	0	83	284	465	525	771	773	1085
-1	0	0	-1	26	73	179	349	576	547	690
-1	0	0	1	33	128	269	509	945	924	1826
1	0	1	0	242	243	288	835	933	642	809
1	0	-1	0	306	315	336	840	927	1004	966
1	-1	0	0	239	225	220	417	492	493	432
1	1	0	0	316	564	763	777	1273	1673	1796
1	0	0	-1	244	337	410	544	859	868	949
1	0	0	1	220	468	1003	1065	1434	2539	1736
0	0	-1	-1	216	371	415	454	446	516	651
0	-1	-1	0	130	311	389	453	472	303	362
0	1	-1	0	350	689	735	812	1246	1089	1462
0	0	-1	1	667	946	1170	1208	2045	2424	2145
0	-1	0	1	258	467	469	471	956	317	488
0	1	0	1	645	1196	1624	1869	3456	4646	4165
0	-1	0	-1	182	350	434	500	731	415	510
0	1	0	-1	345	613	738	804	1440	1423	1812
0	0	0	0	207	429	488	524	917	838	1099
0	-1	1	0	167	336	339	358	653	379	553
0	1	1	0	259	381	516	593	1443	1298	2065
0	0	1	-1	177	368	376	411	779	790	1065
0	0	1	1	185	356	378	437	1061	945	1797
0	0	0	0	235	385	459	535	825	926	1120
0	0	0	0	196	422	495	545	859	967	1131

5.1.2 Prediction of the Effluent SS

Once the coefficients of the assumed empirical model are estimated, the model represented by Eq. (4.1) can be used to predict any combination of variables.

Each group of 15 constants shown in Table (5.2) were substituted in Eq. (4.1) to give an empirical model for each

Table 5.2 : Regression Coefficients for the Empirical Model

Coeff- icients	Sludge level (%)						
	11	21	32	43	53	64	75
B ₀	212.7	412.0	480.7	534.7	867.0	910.3	1116.7
B ₁	90.8	95.8	91.1	151.8	172.3	269.7	78.3
B ₂	83.0	156.7	226.4	244.5	490.2	708.2	805.1
B ₃	-61.8	-84.4	-104.8	-109.1	-23.0	-121.3	35.1
B ₄	68.2	120.8	196.8	208.1	422.2	603.0	540.0
B ₁₁	-85.2	-208.2	-144.0	-21.6	-233.2	-127.7	-233.5
B ₂₂	34.3	93.3	108.2	64.9	218.3	135.3	100.7
B ₃₃	35.3	-19.0	-52.0	15.0	-64.7	-199.0	-82.6
B ₄₄	59.0	105.7	170.7	164.3	386.8	516.3	441.3
B ₁₂	5.0	53.3	87.8	20.5	113.3	226.0	164.5
B ₁₃	9.8	-1.8	15.3	42.8	3.8	-47.0	31.3
B ₁₄	-7.8	19.0	125.8	90.3	51.5	323.5	-87.3
B ₂₃	-32.0	-83.3	-42.3	-31.0	4.0	33.3	103.0
B ₂₄	56.0	116.5	212.8	273.5	447.8	830.3	593.8
B ₃₄	-110.8	-146.8	-188.3	-182.0	-329.3	-438.3	-190.5

sludge level in the tank. To estimate the effluent SS concentration for a given combination of variables, only the coded values of the variables could be substituted in Eq. (4.1).

Using Table (5.2) and the model (4.1), the effluent SS concentrations were estimated for all 81 possible combinations of variables at all 7 sludge levels in the tank. The results are presented in Table (5.3).

In a few particular cases the model predicts negative values for the effluent SS concentration. All predicted negative values are associated with the combinations of variables involving $V = 15$ L (i.e., $X_2 = -1$). This means that the flushed volume $V = 15$ L creates little if any turbulence inside the tank. Those few negative values were ignored and replaced by zeros in Table (5.3).

Table 5.3 : Predicted Effluent SS Concentration (mg/L)

Coded Variables				Sludge level (%)						
X ₁	X ₂	X ₃	X ₄	11	21	32	43	53	64	75
-1	-1	-1	-1	0	51	365	476	404	703	499
-1	-1	-1	0	68	77	241	338	269	75	282
-1	-1	-1	1	258	315	458	529	908	478	947
-1	-1	0	-1	32	217	528	522	767	1233	673
-1	-1	0	0	0	97	215	202	303	166	265
-1	-1	0	1	72	188	244	211	613	132	740
-1	-1	1	-1	139	346	586	598	1001	1365	682
-1	-1	1	0	0	78	85	96	207	0	83
-1	-1	1	1	0	23	0	0	188	0	368
-1	0	-1	-1	16	28	225	393	110	187	342
-1	0	-1	0	144	171	314	528	424	388	718
-1	0	-1	1	389	525	743	992	1510	1622	1978
-1	0	0	-1	20	111	345	408	478	750	619
-1	0	0	0	37	107	246	361	462	513	805
-1	0	0	1	172	314	487	643	1219	1309	1873
-1	0	1	-1	94	156	361	453	715	915	731
-1	0	1	0	0	5	74	224	370	240	726
-1	0	1	1	25	66	127	325	798	597	1604
-1	1	-1	-1	104	191	301	439	254	0	387
-1	1	-1	0	288	451	603	848	1015	973	1357
-1	1	-1	1	590	921	1245	1586	2549	3037	3210
-1	1	0	-1	76	191	379	423	625	537	766
-1	1	0	0	149	304	492	650	1057	1130	1546
-1	1	0	1	34	628	947	1206	2262	2756	3208
-1	1	1	-1	119	153	353	437	867	735	981
-1	1	1	0	81	119	278	482	969	890	1570
-1	1	1	1	161	296	544	856	1845	2078	3042

Table 5.3 : (Continued)

Coded Variables				Sludge level (%)						
X ₁	X ₂	X ₃	X ₄	11	21	32	43	53	64	75
0	-1	-1	-1	165	286	371	496	641	598	702
0	-1	-1	0	229	331	373	448	557	293	398
0	-1	-1	1	411	588	716	729	1248	1020	976
0	-1	0	-1	211	450	549	585	1008	1081	907
0	-1	0	0	164	349	362	355	595	337	412
0	-1	0	1	235	459	517	454	956	626	800
0	-1	1	-1	327	577	623	704	1245	1166	947
0	-1	1	0	169	328	248	292	503	0	262
0	-1	1	1	130	292	214	209	535	0	459
0	0	-1	-1	190	316	319	433	461	308	710
0	0	-1	0	310	477	533	659	825	833	999
0	0	-1	1	548	851	1089	1213	1964	2390	2171
0	0	0	-1	204	397	455	491	832	824	1018
0	0	0	0	213	412	481	535	867	910	1117
0	0	0	1	340	638	848	907	1676	2030	2098
0	0	1	-1	288	440	288	440	486	579	1073
0	0	1	0	186	309	324	441	779	590	1069
0	0	1	1	203	388	503	631	1259	1271	1860
0	1	-1	-1	283	532	483	500	717	288	919
0	1	-1	0	459	811	910	999	1530	1643	1802
0	1	-1	1	753	1300	1679	1827	3116	4031	3567
0	1	0	-1	265	530	577	527	1092	837	1330
0	1	0	0	330	662	815	844	1576	1754	2022
0	1	0	1	51	1005	1396	1490	2832	3703	3597
0	1	1	-1	317	491	566	584	1338	988	1576
0	1	1	0	271	475	616	719	1492	1467	2078
0	1	1	1	344	672	1008	1183	2419	2978	3462

Table 5.3 : (Continued)

Coded Variables				Sludge level (%)						
X ₁	X ₂	X ₃	X ₄	11	21	32	43	53	64	75
1	-1	-1	-1	164	102	90	473	411	238	439
1	-1	-1	0	220	166	217	515	380	256	47
1	-1	-1	1	394	442	686	886	1122	1307	538
1	-1	0	-1	219	265	283	604	782	673	675
1	-1	0	0	165	182	222	465	421	253	93
1	-1	0	1	228	311	502	654	834	866	393
1	-1	1	-1	345	389	372	766	1023	711	746
1	-1	1	0	180	160	122	445	333	0	0
1	-1	1	1	133	142	215	452	417	27	83
1	0	-1	-1	194	185	125	430	345	173	611
1	0	-1	0	306	366	465	746	761	1022	813
1	0	-1	1	536	758	1147	1391	1950	2903	1897
1	0	0	-1	217	265	276	531	719	642	950
1	0	0	0	218	299	428	665	806	1052	962
1	0	0	1	338	544	921	1128	1667	2495	1856
1	0	1	-1	311	306	323	662	965	713	1124
1	0	1	0	202	193	286	614	722	685	945
1	0	1	1	210	292	591	894	1253	1690	1649
1	1	-1	-1	292	455	377	518	715	379	984
1	1	-1	0	460	752	930	1107	1578	2058	1780
1	1	-1	1	746	1261	1824	2025	3216	4769	3458
1	1	0	-1	283	451	486	587	1093	881	1427
1	1	0	0	341	602	850	995	1628	2122	2032
1	1	0	1	516	964	1556	1731	2936	4395	3520
1	1	1	-1	345	410	490	687	1343	986	1704
1	1	1	0	292	413	666	913	1548	1788	2119
1	1	1	1	357	629	1184	1467	2527	3623	3416

5.1.3 Comparison Between Measured and Predicted SS

The measured effluent SS for the designed combinations of variables, and their corresponding predicted values (i.e., using the empirical model) are plotted in the same figure

for each combination of variables for comparison purposes. The figures are presented in Appendix B. Comparing the measured and the predicted values in Appendix B it can be seen that in most cases the agreement between the measured and the predicted values is acceptable.

For comparison purposes, a line of best fit was drawn for each of the measured and the predicted values for each combination of variables as shown in Appendix B.

5.1.4 Sensitivity Analysis

By varying one variable and keeping all other variables constant, the effect of the variable on the effluent SS could be analyzed. This could be done using the predicted values presented earlier in Table (5.3) or simply by comparing the figures obtained from the experimental work, presented in Appendix A. The latter choice is selected for the following discussion.

5.1.4.1 Effect of the Bottom Slope (S)

By comparing Figs. (A.1-6) for the bottom slope $S = \text{flat}$, and their corresponding Figs. (A.7-10) for $S = 1:1$, it can be seen that the bottom slope affects the effluent SS especially at low sludge levels. For instance, the effluent SS was almost doubled when the slope S was changed from flat to 1:1.

5.1.4.2 Effect of the Flushed Volume (V)

The flushed volume $V = 45$ L created the highest turbulence in the tank, and hence the concentration of SS in the effluent was very high. However, $V = 15$ L created in most cases little turbulence if any, and hence the concentration of SS sent into the effluent pipe was very low. This is clearly shown by comparing Figs. (A.3, A.9, A.15, A.17, A.19, A.21) with Figs. (A.4, A.10, A.16, A.18, A.20, A.22), respectively.

5.1.4.3 Effect of the Inlet Location (D)

The turbulence created by varying the inlet location was not found to have a significant effect. It was found that as the inlet was moved closer to the bottom of the tank, and the tank was filled with sludge, the disturbance of the deposited solids became negligible. This is probably due to the weight of the material above the inlet. The sludge seems to dissipate the turbulence and absorb most of the kinetic energy. The above point is illustrated by comparing Figs. (A.1, A.7, A.13, A.15, A.16) with Figs. (A.2, A.8, A.23, A.21, A.22), respectively.

5.1.4.4 Effect of the Outlet Location (L)

The outlet location was found to be the most critical of all studied variables. By comparing the effluent SS concentration for $L = 0.1$ m and for $L = 0.5$ m, it is clearly seen

that the highest effluent SS concentration obtained in the experimental study was for the combinations of variables involving $L = 0.5$ m. The figures that can be compared are Figs. (A.5, A.11, A.13, A.17, A.18, A.23) with Figs. (A.6, A.12, A.14, A.19, A.20, A.24), respectively.

5.1.5 Relation Between Effluent and Tank SS

In an actual interceptor tank different quantities of wastewater are flushed. Although volumes of wastewater flushed into the tank can be greater than 15 L, they are not as frequent as volume of 15 L, as it can be seen from Table (2.2). Based on this fact, only the volume of 15 L is used in the analysis that follows. However, the same procedure presented in the following sections can be applied for the volumes of 30 and 45 L.

The values of the effluent SS concentrations for all combinations involving $V = 15$ L (i.e., $X_2 = -1$) presented earlier in Table (5.3) were fitted to different mathematical expressions for the effluent and tank SS concentrations. Linear, logarithmic, exponential, and power regressions were tested on the plotted points. Based on the correlation coefficient (R), the power regression was found to describe best the relation between effluent and tank SS concentrations. The power regression has the following form:

$$C_{\text{eff}} = \alpha C_T^\beta \quad (5.1)$$

where α and β are the regression constants
 C_{eff} = effluent SS concentration in mg/L
 C_T = tank nominal SS concentration in g/L

Typical curves for $V = 15, 30, \text{ and } 45 \text{ L}$ are shown in Fig. (5.1). The coefficients α , β , and R for the power regression are presented in Table (5.4) for all combinations of variables involving $V = 15 \text{ L}$.

Table 5.4 : Regression Coefficients for the Power Curve

Coded Variables				α	β	R
X_1	X_2	X_3	X_4			
-1	-1	-1	-1	0.001	3.17	0.92
-1	-1	-1	0	16.58	0.60	0.57
-1	-1	-1	1	48.96	0.62	0.87
-1	-1	0	-1	0.73	1.67	0.94
-1	-1	0	0	0.007	2.52	0.85
-1	-1	0	1	9.38	0.86	0.72
-1	-1	1	-1	12.44	1.01	0.92
-1	-1	1	0	0.35	1.16	0.35
-1	-1	1	1	0.01	1.68	0.43
0	-1	-1	-1	24.57	0.76	0.99
0	-1	-1	0	132.44	0.27	0.63
0	-1	-1	1	113.62	0.51	0.93
0	-1	0	-1	29.09	0.81	0.96
0	-1	0	0	67.28	0.44	0.78
0	-1	0	1	57.16	0.59	0.88
0	-1	1	-1	67.73	0.63	0.93
0	-1	1	0	3587	-0.89	0.28
0	-1	1	1	1230	-0.61	0.19
1	-1	-1	-1	20.67	0.64	0.62
1	-1	-1	0	429.30	-0.19	0.17
1	-1	-1	1	117.76	0.48	0.70
1	-1	0	-1	30.67	0.71	0.91
1	-1	0	0	155.57	0.10	0.12
1	-1	0	1	58.24	0.57	0.77
1	-1	1	-1	81.13	0.52	0.82
1	-1	1	0	204345	-2.23	0.57
1	-1	1	1	381.17	-0.25	0.17

The expression (5.1) describing the relation between C_{MZZ} and C_T will be used in the following section, and it will be

combined with the mass balance equation to determine how the effluent and the tank SS concentrations vary with time.

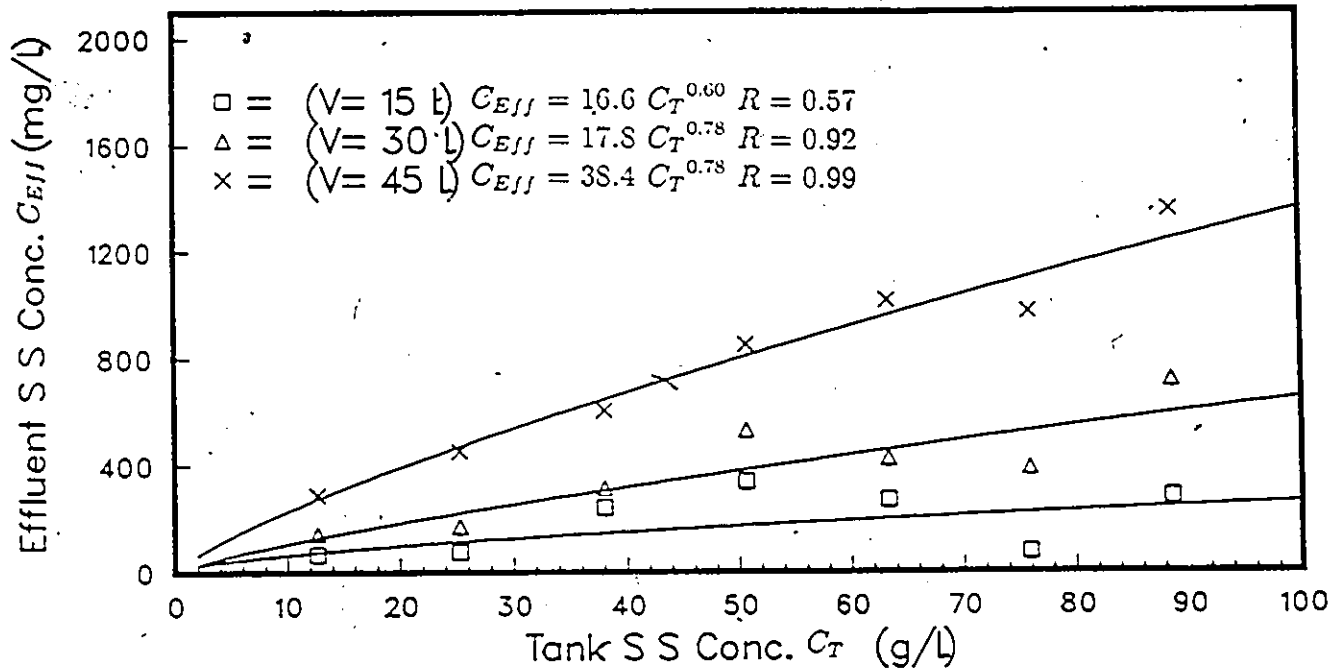


Figure 5.1: Plot of the predicted Values
of the Effluent S S Conc.
($S = \text{flat}$, $D = 0.1$ m, $L = 0.3$ m)

5.1.6 Determination of the Rate of Accumulation

5.1.6.1 Assumption of the Inlet SS

The concentration of SS entering the interceptor tank differs from place to place and time to time. It is very difficult to define a constant value of inlet SS concentration to be a general case. The assumption that the inlet SS concentration is constant only applies to the average value throughout the designed lifetime of the tank. In the analysis that follows, different inlet SS concentrations were as-

sumed. Values of 200, 400, and 600 mg/L were selected (Metcalf & Eddy, 1979).

5.1.6.2 Assumption of the Rate Constant K

The rate of digestion, r , can be defined in this study as the rate of loss of SS within the system. SS can be digested by microorganisms and transformed into gases, such as methane and hydrogen sulfide or be transformed into dissolved solids.

The rate constant, K , in Eq. (3.39) is very critical in determining the rate of accumulation of SS in the tank. A proper selection of K based on experimental studies gives a realistic accumulation of SS in the tank. Since the value of K is not available in the literature, it is assumed to be zero.

Ignoring the amount of suspended solids that will be transformed into soluble solids or gases by anaerobic digestion (i.e., assuming $K=0$) is not expected to have a significant influence on the relative ranking of desludging intervals for each tank geometry. The amount of solids that is transformed is proportional to the amount of solids in the tank. The fraction of solids transformed per day will be the same for each tank unless there is a differential rate of washout of degradable and nonbiodegradable solids. There is no reason to assume that degradable suspended solids will leave the tank at a rate different from nondegradable suspended solids. Therefore, the decrease in suspended solids

due to anaerobic digestion will lead to an equal percentage increase in solids accumulation time for two tank geometries compared for the same number of days. The relative ranking of the two geometries vis-a-vis time to reach a given effluent solids concentration will not change.

5.1.6.3 Application of the Mass Balance Equation

As discussed in Chapter 3 the mass balance equation can be expressed in terms of C_{eff} and C_T only. Although the inlet SS to the tank could vary significantly, for convenience it is assumed to be constant.

As discussed in Section (5.1.5) the relation between C_{eff} and C_T was found to have the form expressed by Eq. (5.1). Substituting Eq. (5.1) into Eq. (3.39), and replacing C by C_T , yields :

$$\frac{C_{in}}{1000} - \frac{\alpha C_T^n}{1000} = \frac{dC_T}{dt} \quad (5.2)$$

C_{in} is expressed in mg/L, C_T in g/L, and t in days (d). The number 1000 was used in this equation to make it dimensionally constant. Since the tank was designed on a 24-hour hydraulic retention time, t_a was replaced by 1.

The differential equation represented by Eq. (5.2) was solved numerically since an exact solution cannot be obtained. The Runge-Kutta method was selected for this purpose. The program listing and a sample computer output is presented in Appendix F. The tank and the effluent SS Concentrations were obtained for different periods of time.

Table 5.5a : Time Required to Reach C_{MSS} (Days)
(case when $C_{in} = 200$ mg/L)

Coded Variables				C_{MSS} criterion (mg/L)						
X_1	X_2	X_3	X_4	20	40	60	80	100	120	140
-1	-1	-1	-1	115	150	175	200	225	245	275
-1	-1	-1	0	10	25	55	95	150	230	350
-1	-1	-1	1	-	5	10	15	25	35	55
-1	-1	0	-1	40	60	80	100	125	145	175
-1	-1	0	0	120	165	200	235	270	305	350
-1	-1	0	1	15	30	50	80	110	155	205
-1	-1	1	-1	10	20	30	40	55	70	95
-1	-1	1	0	175	335	500	680	895	1145	1460
-1	-1	1	1	400	630	840	1050	1280	1535	1845
0	-1	-1	-1	5	10	20	30	45	65	95
0	-1	-1	0	-	-	-	-	-	5	15
0	-1	-1	1	-	-	-	-	5	10	15
0	-1	0	-1	5	10	15	25	35	45	65
0	-1	0	0	-	-	5	10	20	35	55
0	-1	0	1	-	-	5	10	20	30	45
0	-1	1	-1	-	-	5	10	15	20	30
0	-1	1	0	-	-	-	-	-	-	-
0	-1	1	1	-	-	-	-	-	-	-
1	-1	-1	-1	5	15	35	55	90	135	195
1	-1	-1	0	-	-	-	-	-	-	-
1	-1	-1	1	-	-	-	-	5	10	15
1	-1	0	-1	-	10	15	25	40	55	80
1	-1	0	0	-	-	-	-	-	-	5
1	-1	0	1	-	-	5	10	20	30	45
1	-1	1	-1	-	-	-	5	10	20	30
1	-1	1	0	-	-	-	-	-	-	-
1	-1	1	1	-	-	-	-	-	-	-

Sample plots of tank SS concentration versus time, and effluent SS concentration versus time, for different inlet SS concentrations, are shown in Figs. (5.2-7), respectively.

Table 5.5b : Time Required to Reach C_{xxx} (Days)
(case when $C_{in} = 400$ mg/L)

Coded Variables				C_{xxx} criterion (mg/L)						
X_1	X_2	X_3	X_4	20	40	60	80	100	120	140
-1	-1	-1	-1	60	75	85	95	105	110	115
-1	-1	-1	0	5	10	25	40	60	85	115
-1	-1	-1	1	-	-	-	5	10	15	20
-1	-1	0	-1	20	30	40	45	55	60	70
-1	-1	0	0	60	80	95	110	120	135	145
-1	-1	0	1	5	15	25	35	45	60	75
-1	-1	1	-1	5	10	15	20	25	30	35
-1	-1	1	0	85	155	230	300	375	455	535
-1	-1	1	1	195	305	390	475	555	635	715
0	-1	-1	-1	-	5	10	15	20	25	30
0	-1	-1	0	-	-	-	-	-	-	5
0	-1	-1	1	-	-	-	-	-	-	5
0	-1	0	-1	-	-	5	10	15	20	25
0	-1	0	0	-	-	-	5	10	15	20
0	-1	0	1	-	-	-	-	5	10	15
0	-1	1	-1	-	-	-	-	-	5	10
0	-1	1	0	-	-	-	-	-	-	-
0	-1	1	1	-	-	-	-	-	-	-
1	-1	-1	-1	5	10	15	25	35	50	65
1	-1	-1	0	-	-	-	-	-	-	-
1	-1	-1	1	-	-	-	-	-	-	5
1	-1	0	-1	-	-	5	10	15	20	25
1	-1	0	0	-	-	-	-	-	-	-
1	-1	0	1	-	-	-	-	5	10	15
1	-1	1	-1	-	-	-	-	-	5	10
1	-1	1	0	-	-	-	-	-	-	-
1	-1	1	1	-	-	-	-	-	-	-

This procedure was repeated for all combinations of variables presented in Table (5.4). For different C_{xxx} criteria the time required was obtained and presented in Tables (5.5a-c) for all three cases of inlet SS concentrations.

Table 5.5c : Time Required to Reach C_{max} (Days)
(case when $C_{in} = 600$ mg/L)

Coded Variables				C_{max} criterion (mg/L)						
X_1	X_2	X_3	X_4	20	40	60	80	100	120	140
-1	-1	-1	-1	40	50	55	60	65	70	75
-1	-1	-1	0	5	10	15	25	40	50	70
-1	-1	-1	1	-	-	-	-	-	5	10
-1	-1	0	-1	15	20	25	30	35	40	45
-1	-1	0	0	40	55	65	70	80	85	90
-1	-1	0	1	5	10	15	20	30	35	45
-1	-1	1	-1	-	-	5	10	15	20	25
-1	-1	1	0	55	105	150	195	240	285	330
-1	-1	1	1	130	200	255	310	355	405	450
0	-1	-1	-1	-	-	-	5	10	15	20
0	-1	-1	0	-	-	-	-	-	-	-
0	-1	-1	1	-	-	-	-	-	-	-
0	-1	0	-1	-	-	-	-	5	10	15
0	-1	0	0	-	-	-	-	-	5	10
0	-1	0	1	-	-	-	-	-	5	10
0	-1	1	-1	-	-	-	-	-	-	5
0	-1	1	0	-	-	-	-	-	-	-
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1	-1	-1	0	-	-	-	-	-	-	-
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1	-1	1	-1	-	-	-	-	-	-	5
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1	-1	1	1	-	-	-	-	-	-	-

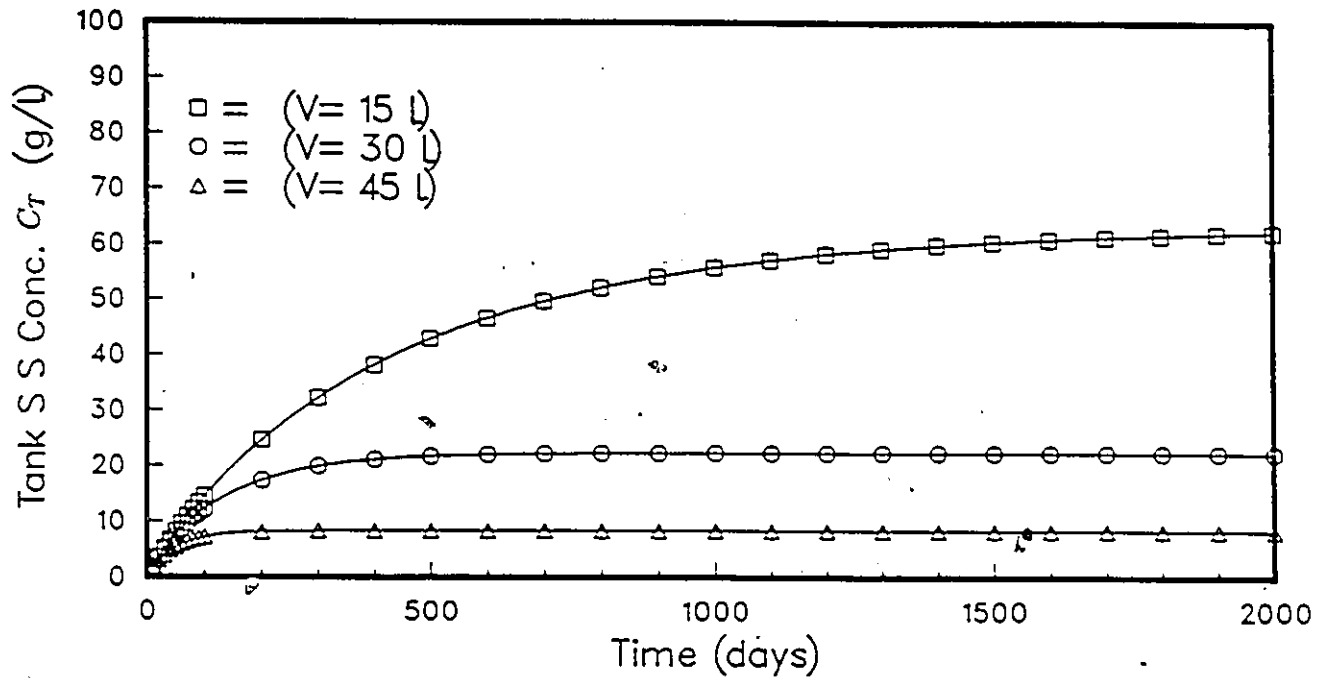


Figure 5.2 : Tank S S Conc. vs Time.
 ($K = 0.0$ 1/d , $c_{in} = 200$ mg/l)
 ($S = \text{flat}$, $D = 0.1$ m, $L = 0.3$ m)

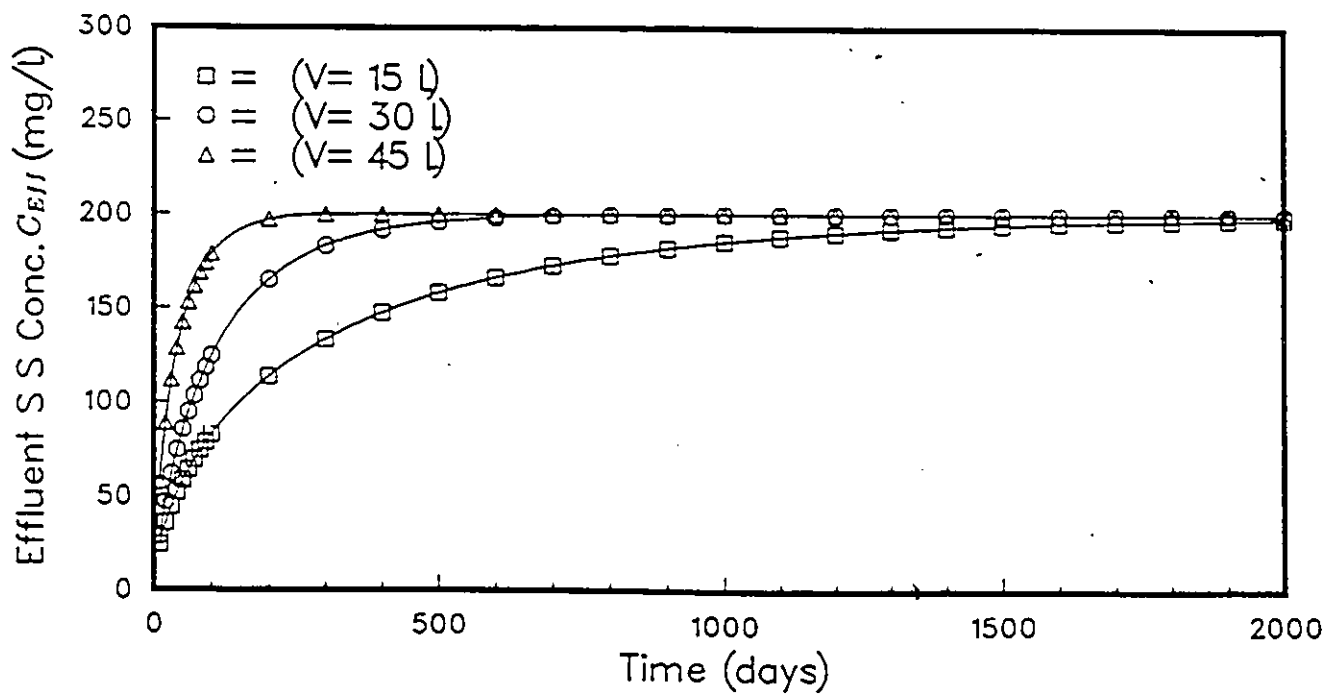


Figure 5.3 : Effluent S S Conc. vs Time.
 ($K = 0.0$ 1/d , $c_{in} = 200$ mg/l)
 ($S = \text{flat}$, $D = 0.1$ m, $L = 0.3$ m)

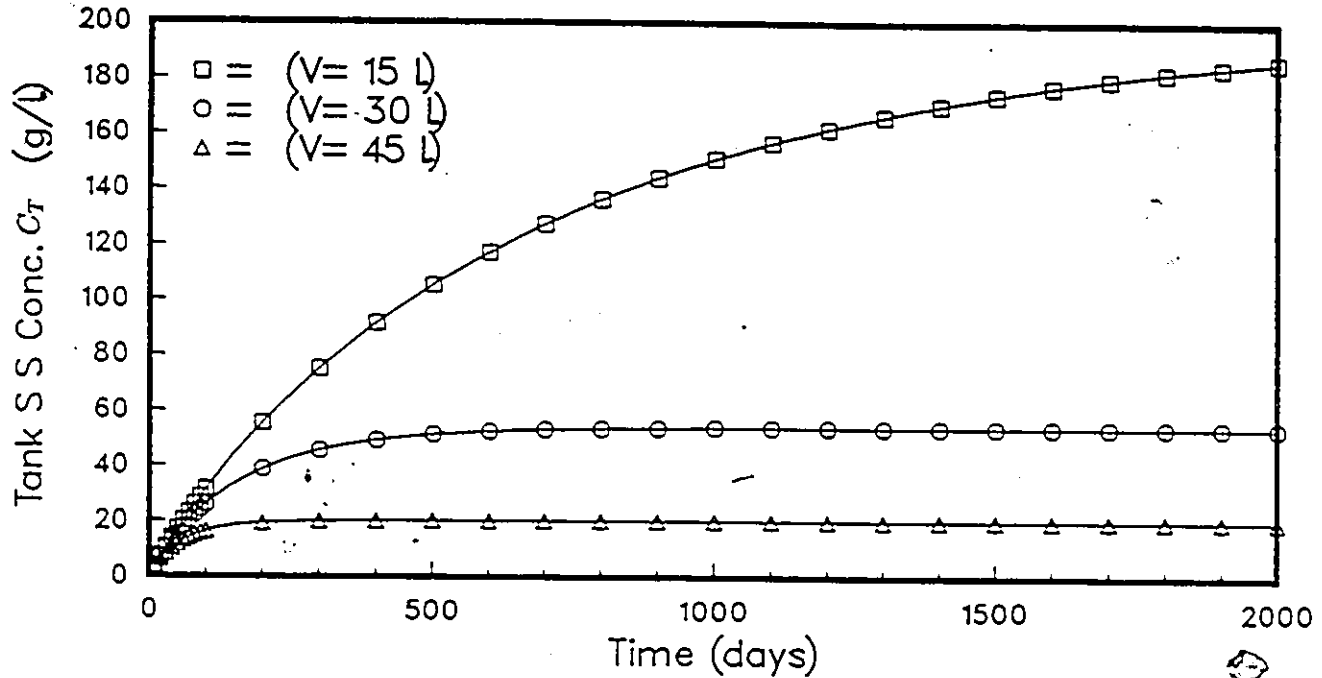


Figure 5.4 : Tank S S Conc. vs Time.

($K = 0.0$ 1/d, $C_{in} = 400$ mg/l)
 (S= flat, D= 0.1' m, L= 0.3 m)

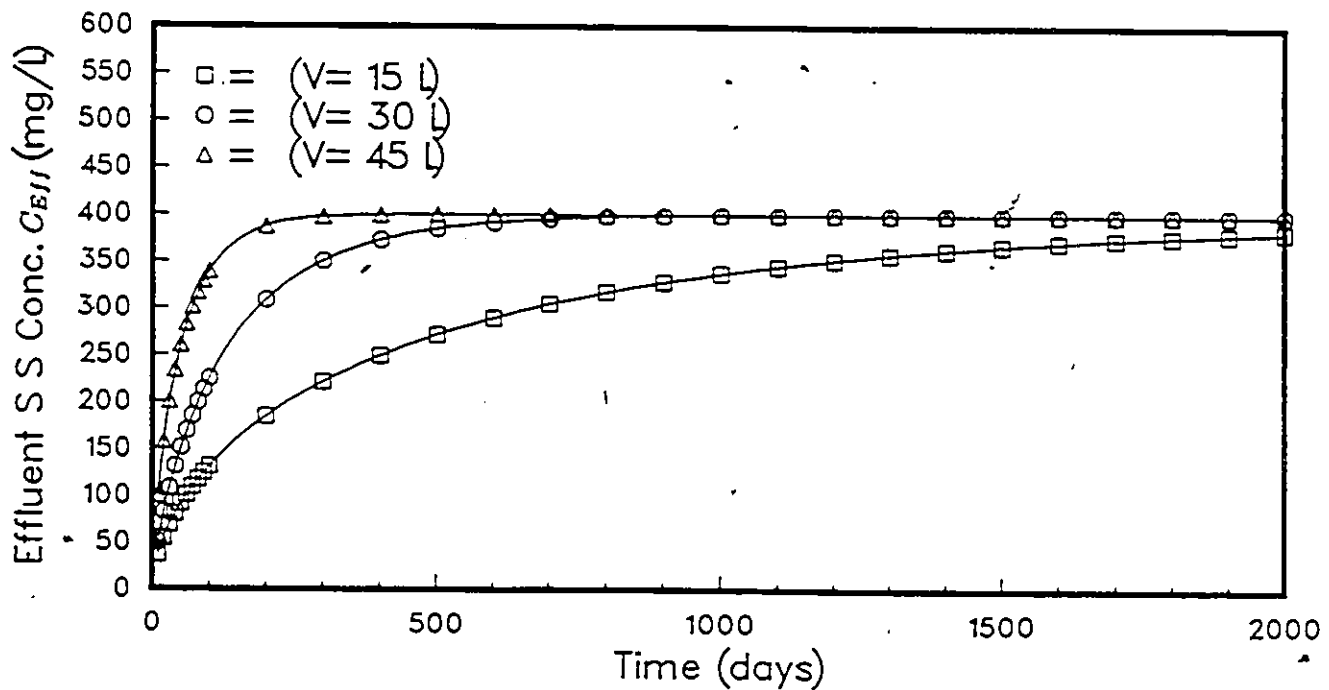


Figure 5.5 . Effluent S S Conc. vs Time.

($K = 0.0$ 1/d, $C_{in} = 400$ mg/l)
 (S= flat, D= 0.1 m, L= 0.3 m)

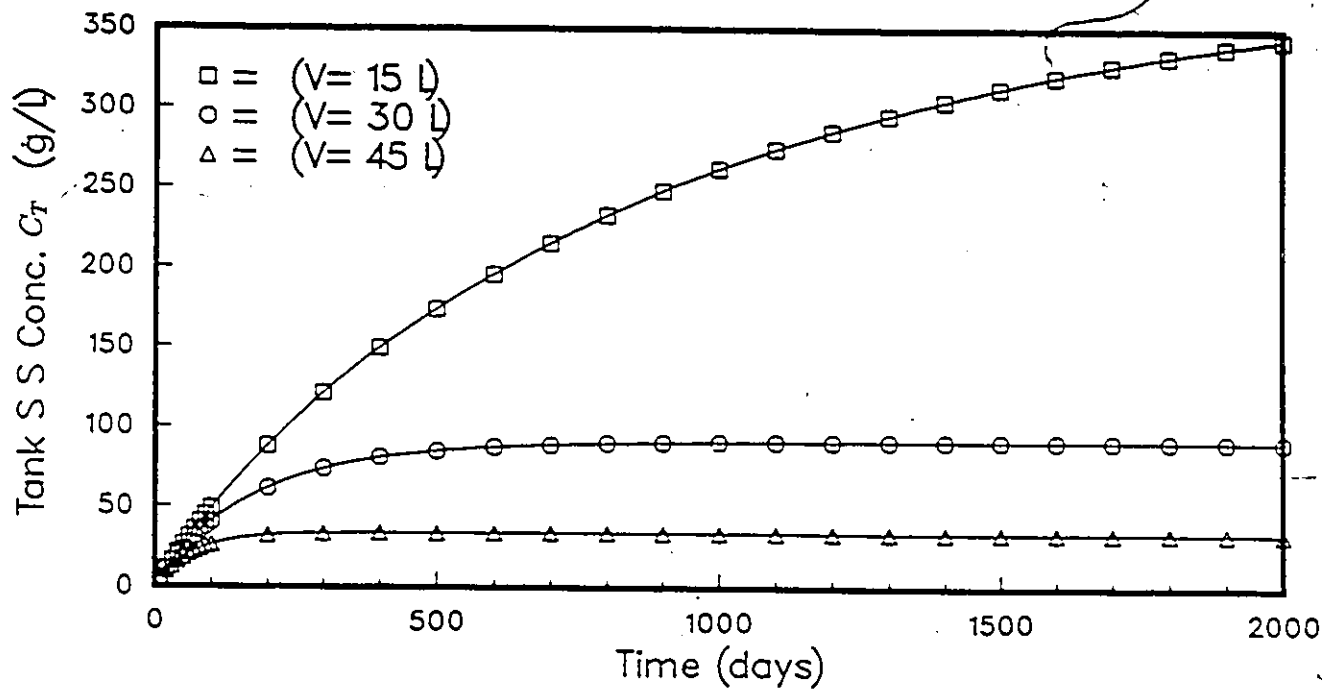


Figure 5.6 : Tank S S Conc. vs Time.
($K = 0.0$ 1/d , $C_{in} = 600$ mg/l)
($S = \text{flat}$, $D = 0.1$ m, $L = 0.3$ m)

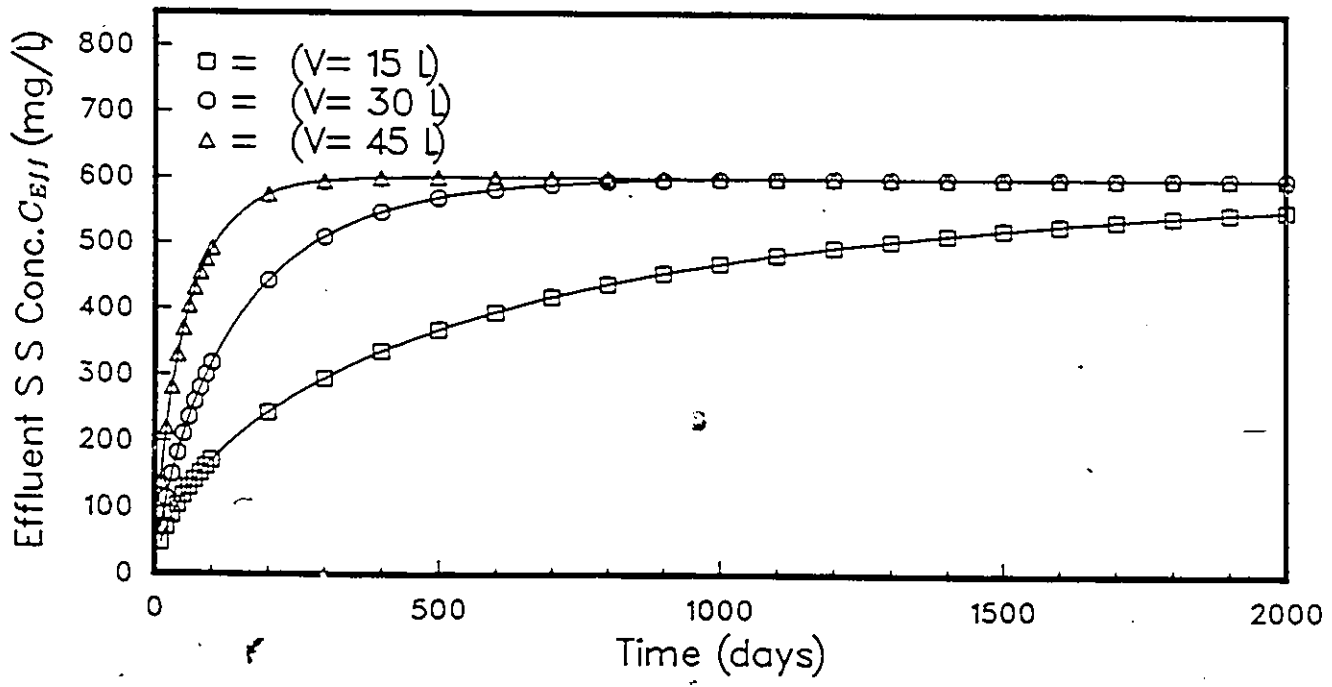


Figure 5.7 : Effluent S S Conc. vs Time.
($K = 0.0$ 1/d , $C_{in} = 600$ mg/l)
($S = \text{flat}$, $D = 0.1$ m, $L = 0.3$ m)

5.1.6.4 Selection of a Convenient Combination of Variables for a Practical Design Case

In developing countries and in many isolated communities in developed countries, the septic tank is still used with the subsurface absorption field as a means of treating domestic wastewater. In many communities, however, the subsurface absorption field has been replaced by small-bore sewers. Since the sewers have the ability to transport colloidal and fine solid particles, some inexpensive modifications to the septic tank could be incorporated. For instance, the locations of the inlet and the outlet pipes could be modified. The inlet pipe may be placed in the center within a given distance from the bottom of the tank. For new constructed homes in isolated communities, a bottom slope could be included in the design of the interceptor tank as well. This has the advantage of accumulating the sludge in the center and increases the effluent SS concentration at the beginning period of the tank operation.

The selection of a convenient combination of variables for a practical design case must be based on the information of the maximum loading capacity of small-bore sewers. This information, unfortunately, is not available. Based on the longest period of time required to reach C_{max} design criterion, combinations of variables $X_1 = -1$, $X_2 = 1$, $X_3 = 0$ (i.e., $S = \text{flat}$, $D = 0.5 \text{ m}$, $L = 0.3 \text{ m}$) or, $X_1 = -1$, $X_2 = 1$, $X_3 = 1$ (i.e., $S = \text{flat}$, $D = 0.5 \text{ m}$, $L = 0.5 \text{ m}$) can be selected from Tables (5.5a-c).

5.2 SETTLING ANALYSIS OF THE EFFLUENT AND THE TANK OM

The settling column as described in Section (4.2.1) and shown in Fig. (3.2) was used to test the settling characteristics of the effluent and the tank OM. The figures of the settling column tests are presented in Appendix D on which the following discussion is based.

5.2.1 Settling Characteristics of the Tank Effluent

The purpose of these tests was to study the quality of the effluent in terms of the settling characteristics of the solids. It was found that the solids in the effluent are only fine solid particles for low sludge levels in the tank. However, as the sludge level in the tank increased heavier particles were carried out in the effluent since the settling characteristics deteriorated. This is clearly seen when comparing, for instance, Figs. (D.4, D.5, and D.6) for the sludge levels 21%, 43%, and 64%, respectively.

The settling characteristics of the solids in the effluent are very important in extending the designed lifetime of the small-bore sewers. The settling of fine solid particles on the invert of the sewer pipes is less likely to occur. This has many advantages such as minimizing the chance of pipe clogging, and reducing the frequency and hence the cost of maintenance.


5.2.2 Settling Characteristics of the Tank OM

The purpose of these tests was to check the quality of the tank OM. To insure that the OM has approximately the same settling characteristics for all experimental runs, the settling column test was also performed on a random basis.

The results of the five tests performed on the tank OM are shown in Figs. (D.10-14). The settling curves of the tank OM shown in part (b) of these figures are very close to each other. The conclusion that can be drawn is that the settling characteristics of the tank OM can be considered to be reasonably the same for all experimental runs.

CHAPTER 6

CONCLUSIONS



From this study the following conclusions can be made:

1. The bottom slope of the tank was found to affect the effluent SS mainly at low sludge levels. By making the bottom slope steeper, the effluent SS almost doubled.
2. High flushed volumes into the tank created a lot of turbulence and hence the effluent SS concentration was very high. The volume 15 L was found to create little turbulence inside the tank and hence the effluent SS was very low.
3. The inlet location was not found to be critical, especially at high sludge levels. The disturbance in the tank decreased as the inlet was moved closer to the bottom and the tank was filled with sludge.
4. The outlet location significantly affected the effluent SS. As the outlet was moved closer to the center of the tank, the effluent SS concentration increased dramatically.
5. The power regression was found to best describe the relationship between the effluent and the tank SS Concentration.
6. For the longest sludge accumulation time, the inlet and the outlet locations can be selected to be 0.5 m and 0.3 m, or 0.5 m and 0.5 m, respectively. This assumes that the

small-bore sewer can handle the SS concentration that will exit the tank.

7. The settling velocities of the particles in the effluent were very low compared with the settling velocities of those in the tank. It took longer periods of time for the particles in the effluent to settle. This means that only fine and colloidal solid particles were exiting the tank. Larger and heavier particles were left in the tank. This minimizes the settling of solid particles in the sewers, and extends their designed lifetime.

CHAPTER 7

RECOMMENDATIONS FOR FUTURE RESEARCH

The experimental work described earlier may be extended in the future and more information could be gathered, concerning the performance of the interceptor tank under conditions other than described here in this project. For instance, tanks with larger and/or smaller volumes may be tested. It is also advisable to carry out an experimental research to test the maximum loading capacity of small-bore sewers. Specifically if the maximum allowable concentration of SS in the system can be determined, then detailed design criteria for interceptor tanks can be established. Different pipe sizes may be tested for this purpose.

There is also the possibility of using computer programs to analyze small-bore sewer systems. Interceptor tank inputs resulting from this experimental work could be introduced into the program to determine the ability of small-bore sewer system to handle the combined load from a series of interceptor tanks.

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APPENDIX A

FIGURES OF THE EFFLUENT SS CONCENTRATION VS TANK SS

Note: Tank SS is a nominal concentration based on the total mass of solids in the total volume (See Table (4.2)).

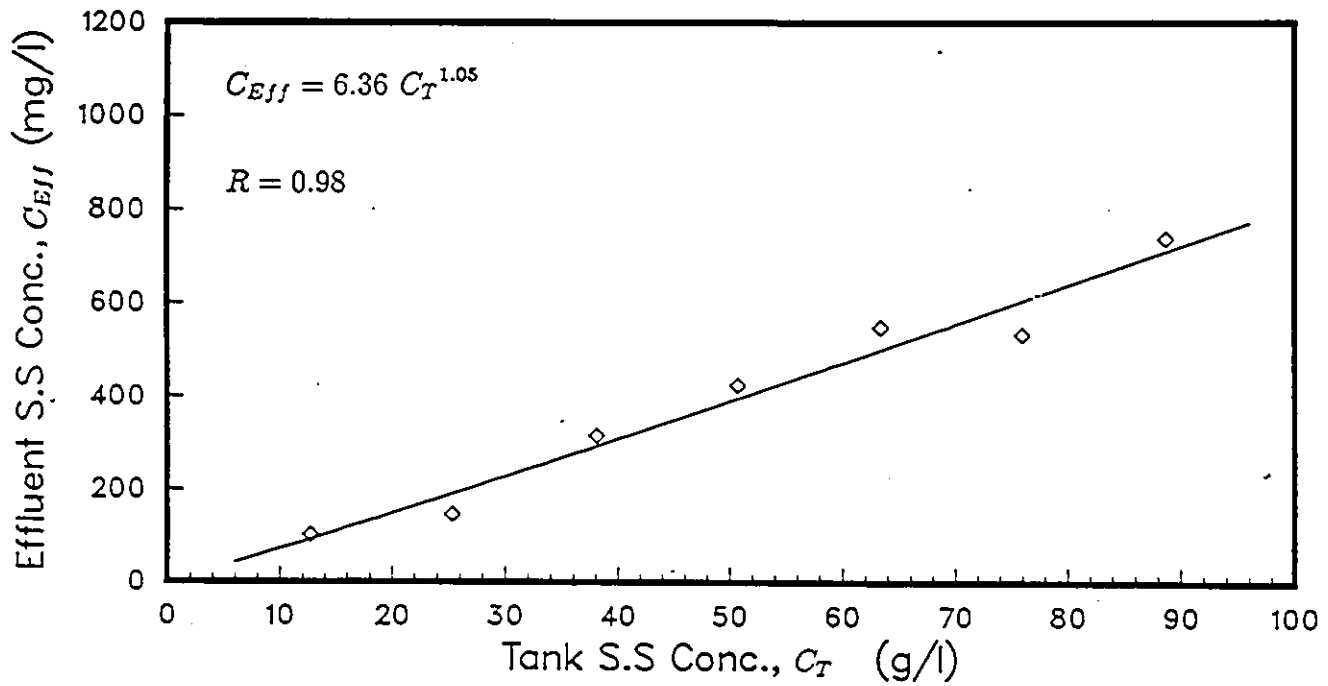


Figure A.1 : Effluent S.S. Conc. vs Tank S.S. Conc.
(S= flat, V= 30 l, D= 0.5 m, L= 0.3 m)

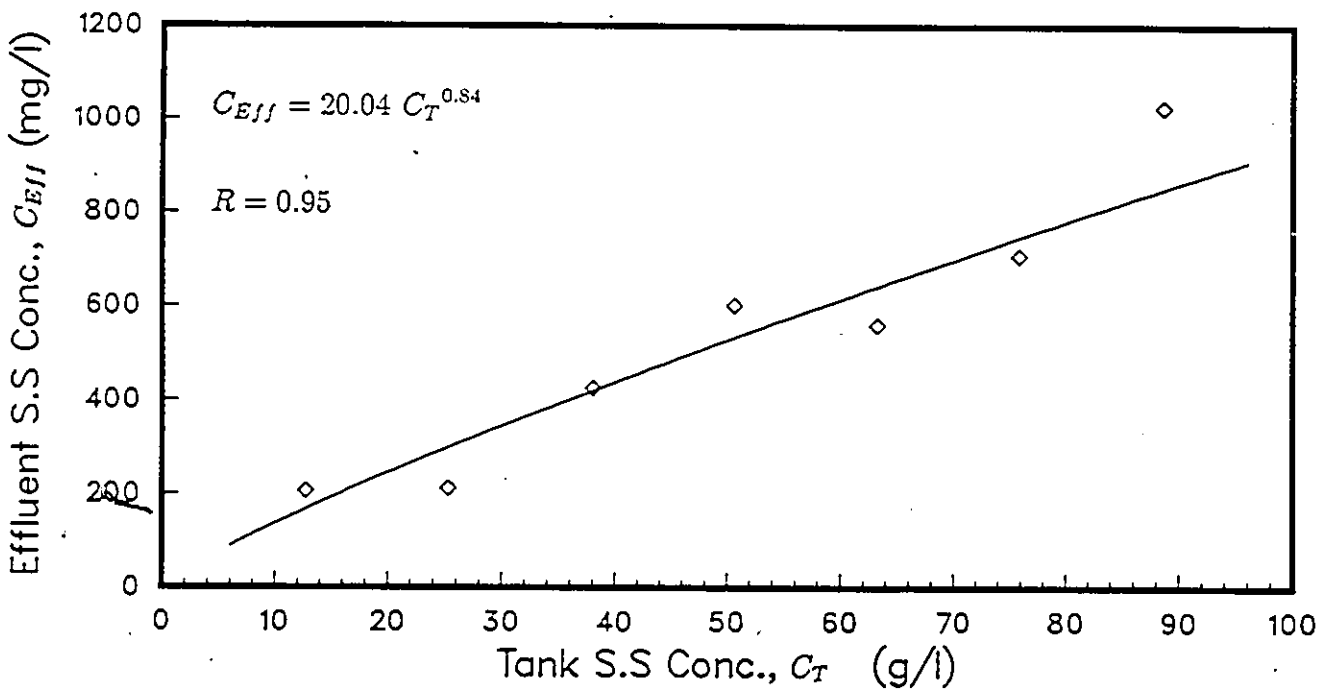


Figure A.2 : Effluent S.S. Conc. vs Tank S.S. Conc.
(S= flat, V= 30 l, D= 0.1 m, L= 0.3 m)

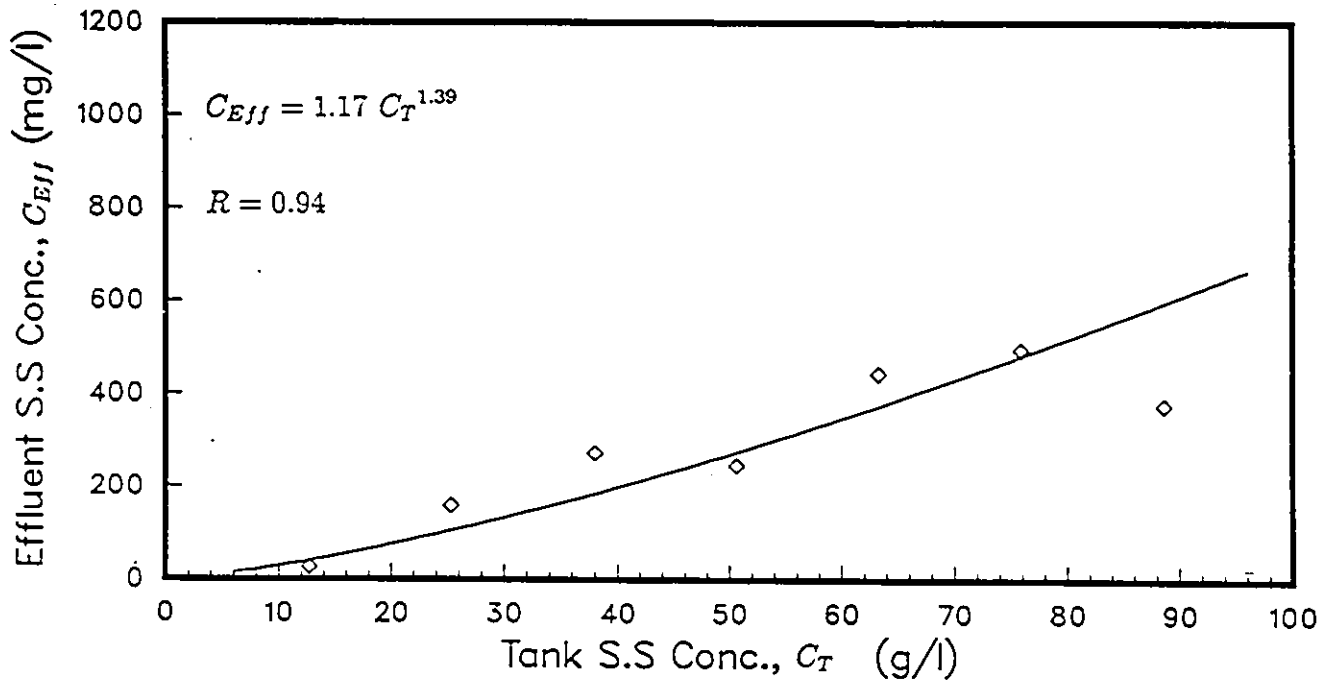


Figure A.3 : Effluent S.S. Conc. vs Tank S.S. Conc.
(S= flat, V= 15 l, D= 0.3 m, L= 0.3 m)

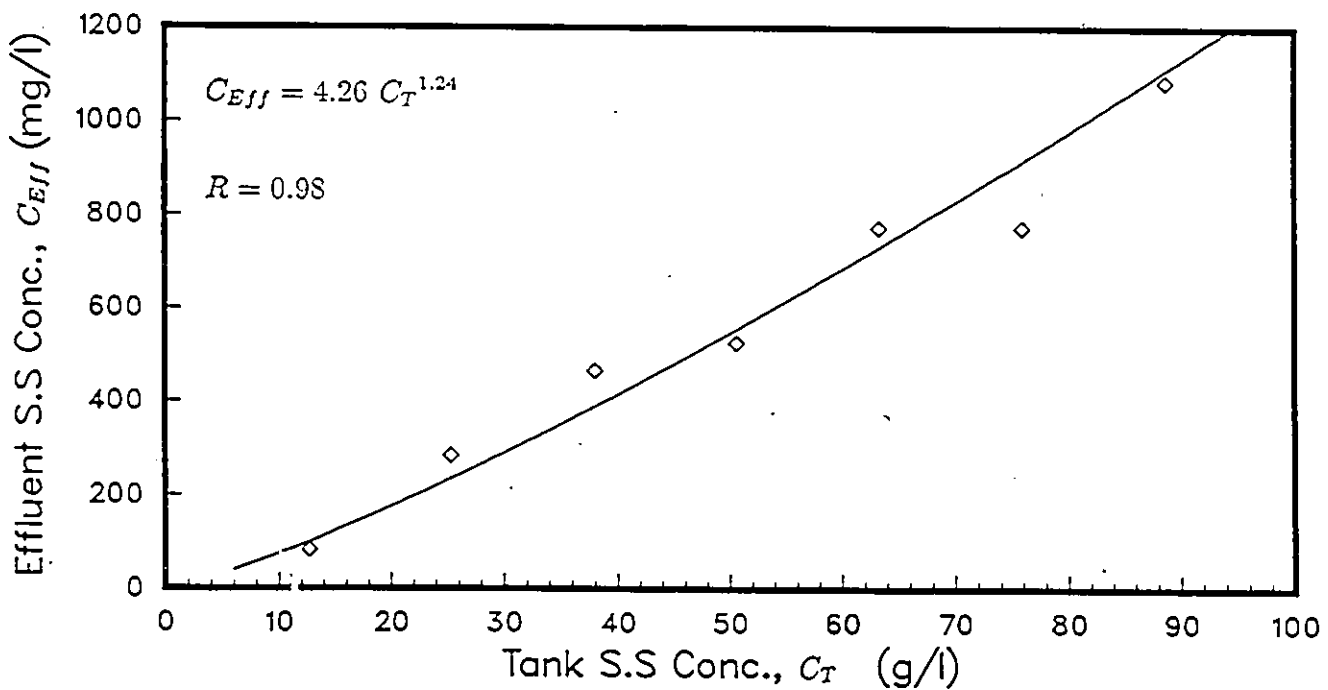


Figure A.4 : Effluent S.S. Conc. vs Tank S.S. Conc.
(S= flat, V= 45 l, D= 0.3 m, L= 0.3 m)

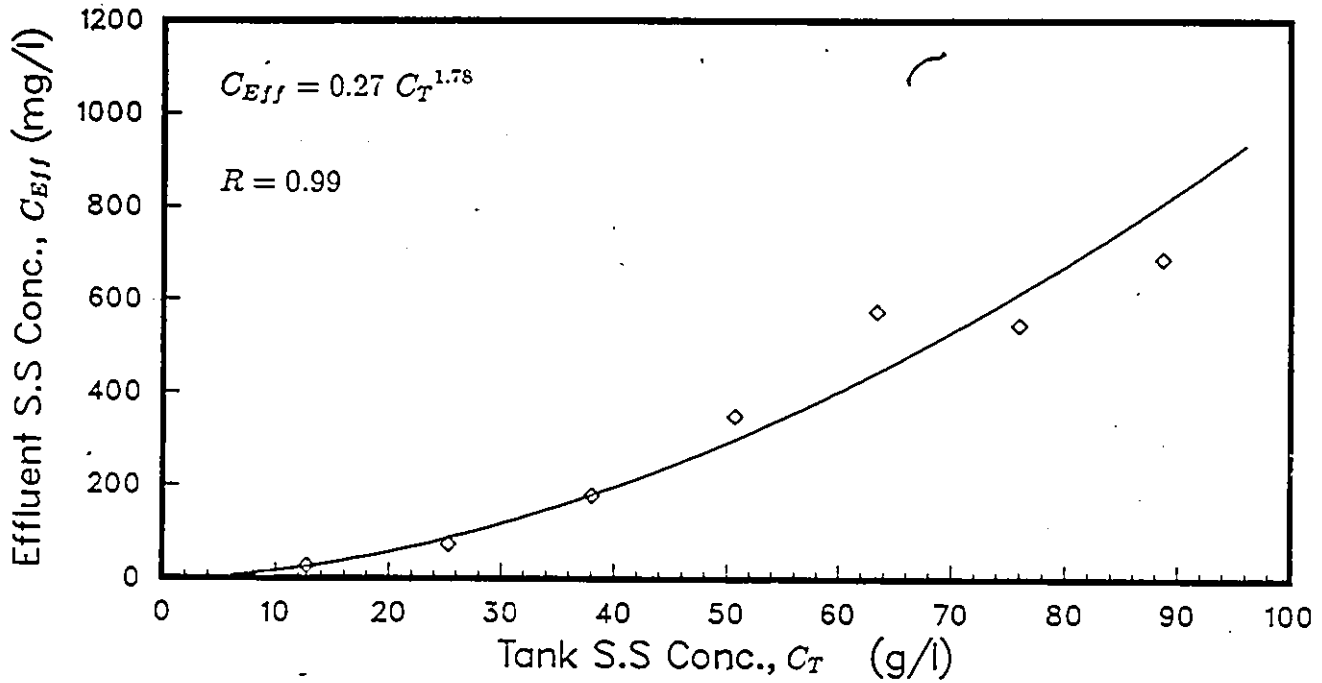


Figure A.5 : Effluent S.S. Conc. vs Tank S.S. Conc.
(S= flat, V= 30 l, D= 0.3 m, L= 0.1 m)

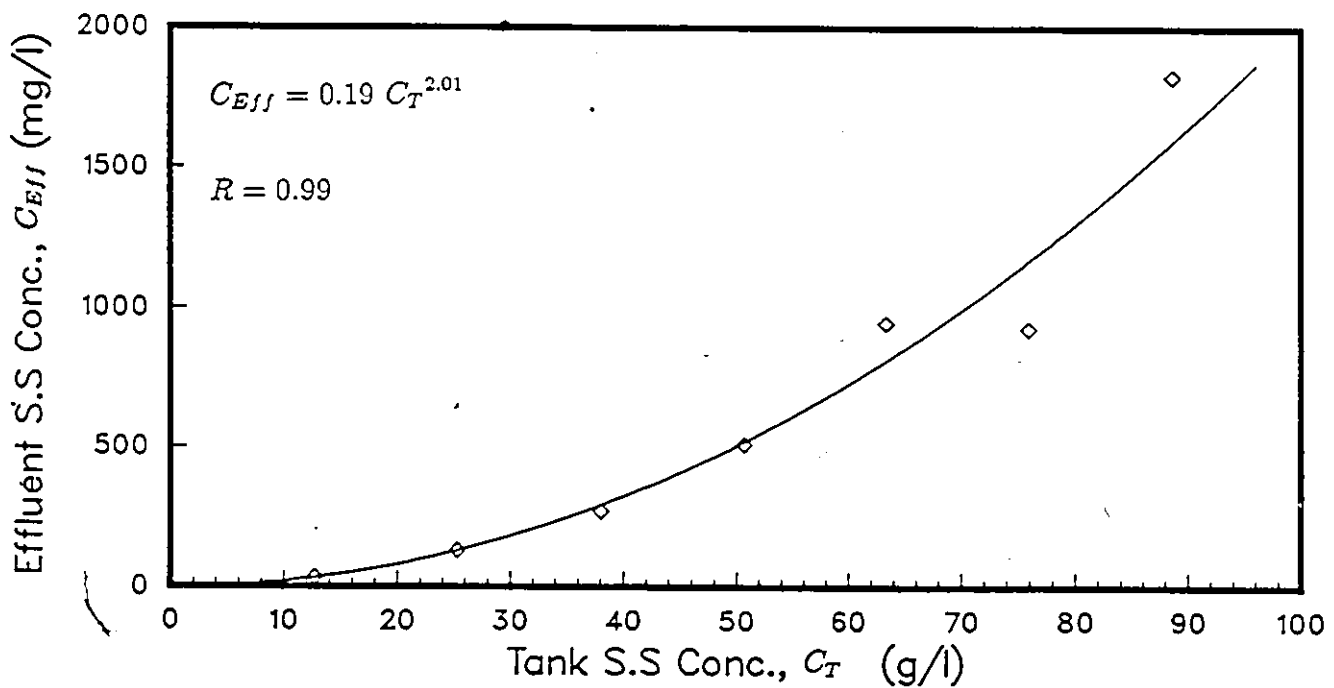


Figure A.6 : Effluent S.S. Conc. vs Tank S.S. Conc.
(S= flat, V= 30 l, D= 0.3 m, L= 0.5 m)

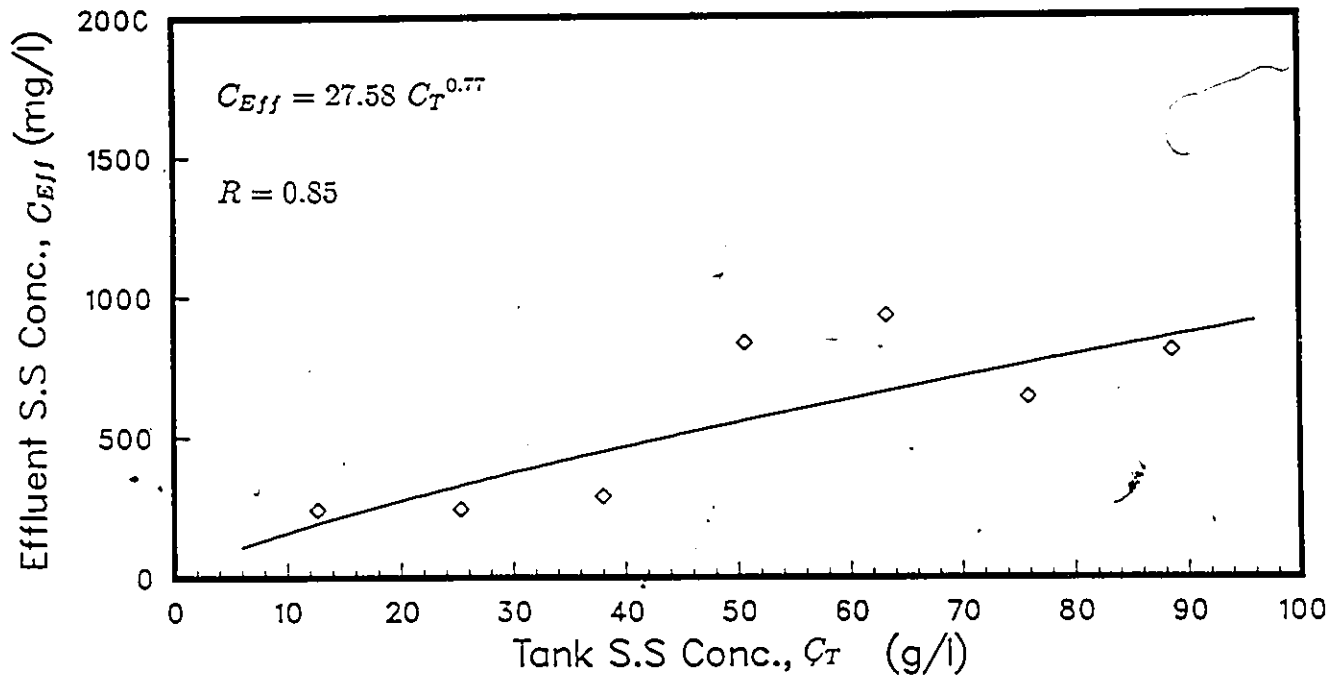


Figure A.7 : Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:1$, $V = 30$ l, $D = 0.5$ m, $L = 0.3$ m)

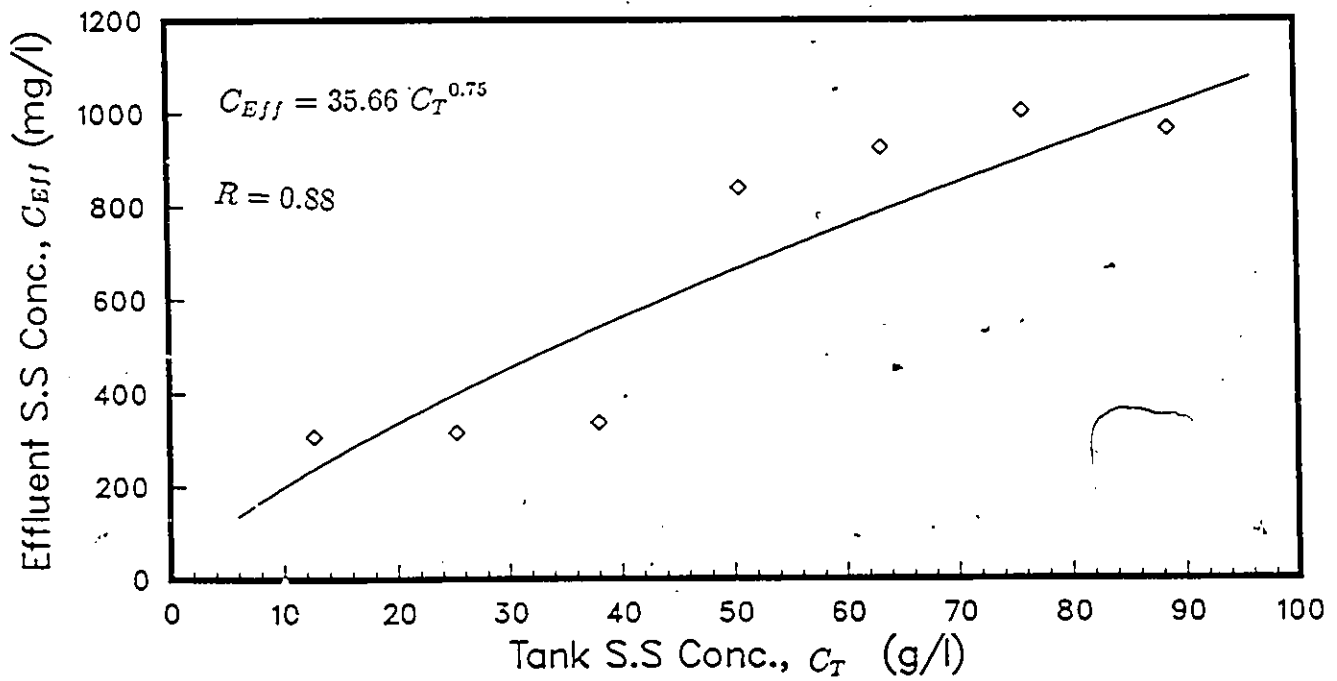


Figure A.8 : Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:1$, $V = 30$ l, $D = 0.1$ m, $L = 0.3$ m)

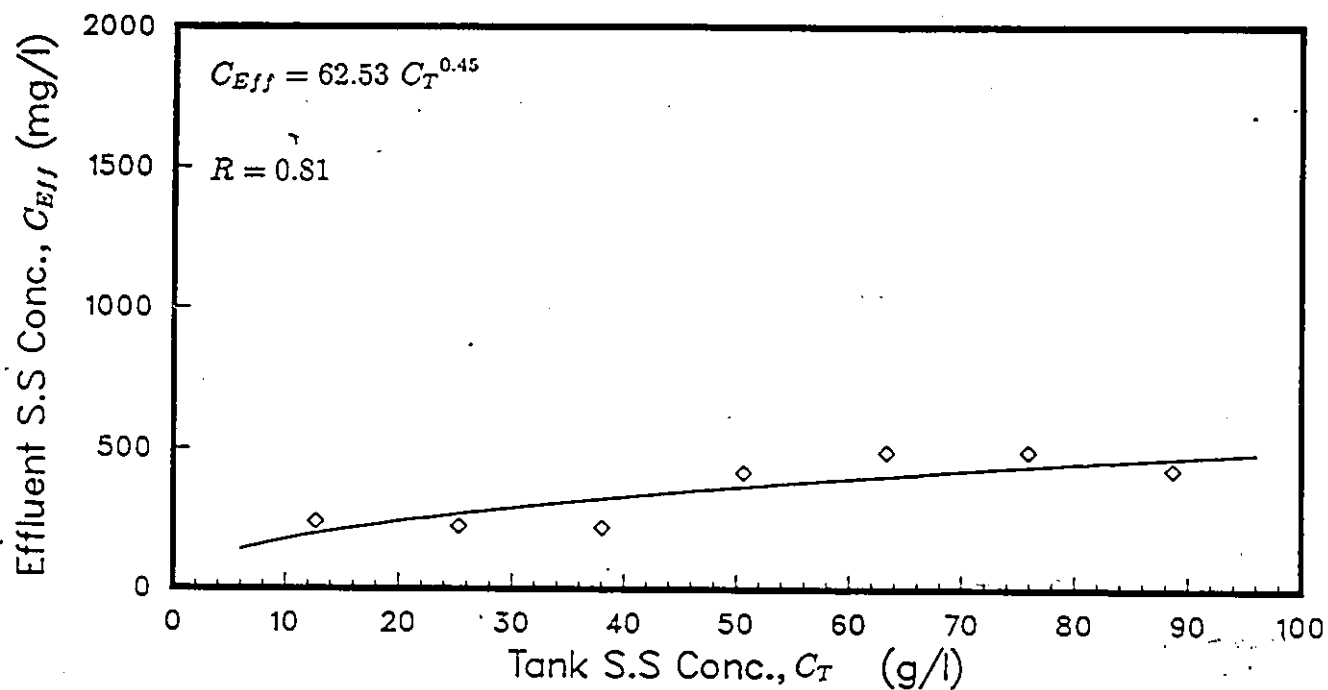


Figure A.9 : Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:1$, $V = 15$ l, $D = 0.3$ m, $L = 0.3$ m)

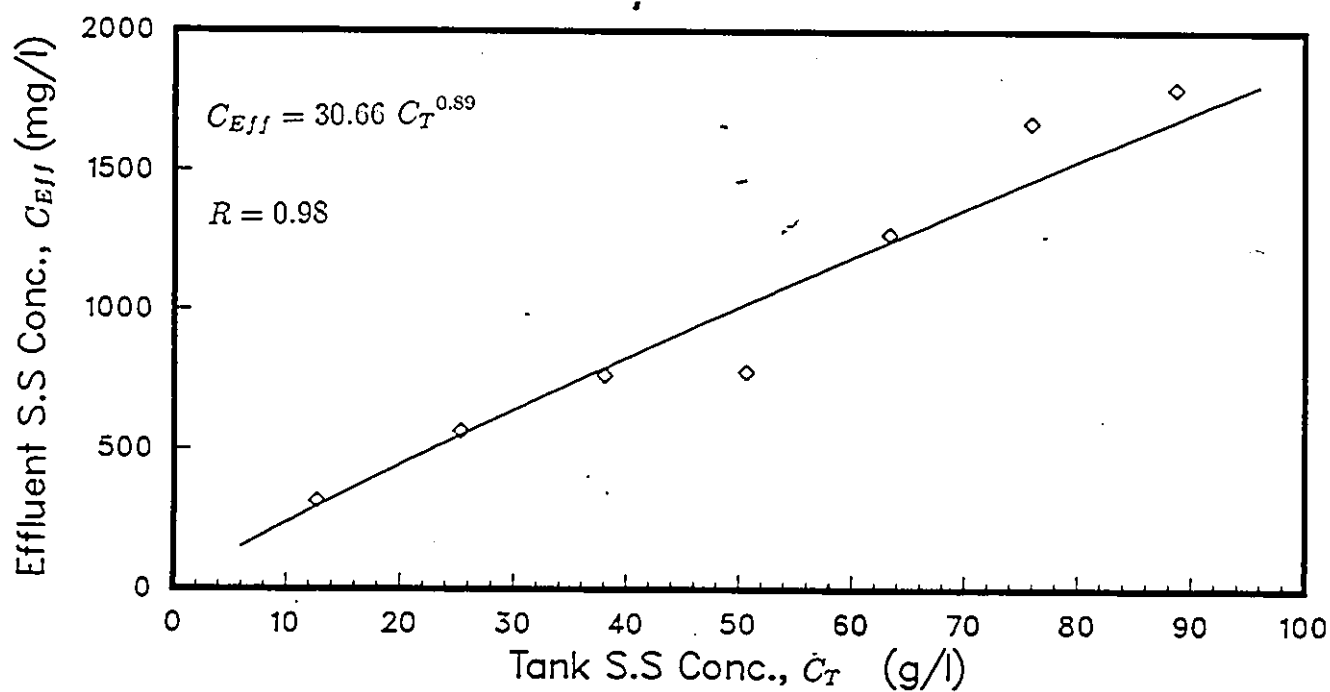


Figure A.10: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:1$, $V = 45$ l, $D = 0.3$ m, $L = 0.3$ m)

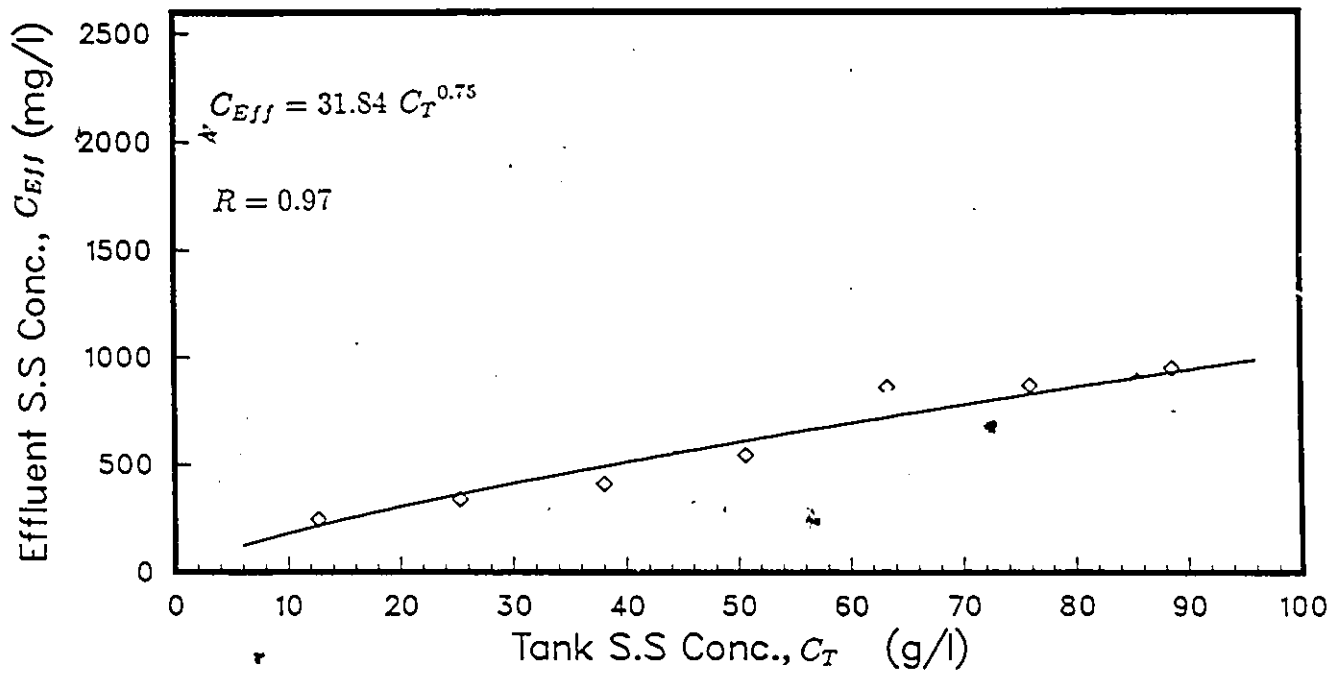


Figure A.11: Effluent S.S. Conc. vs Tank S.S. Conc.
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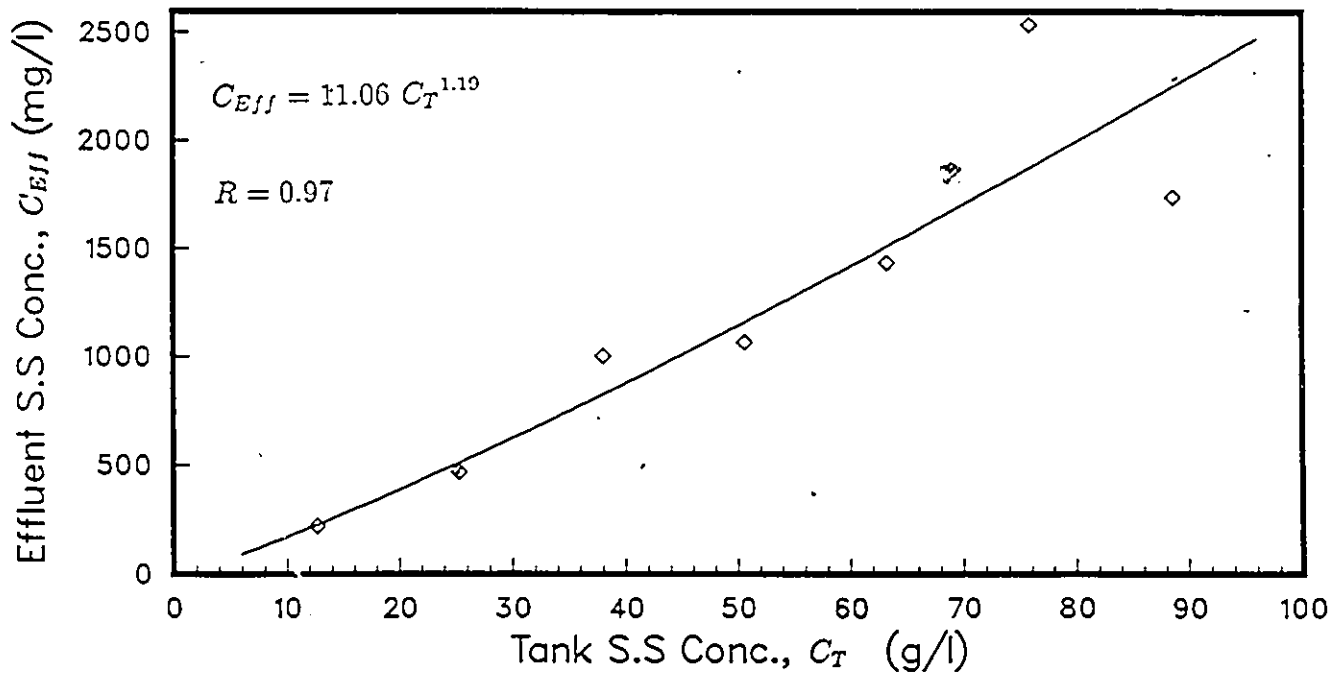


Figure A.12: Effluent S.S. Conc. vs Tank S.S. Conc.
 (S= 1:1 , V= 30 l, D= 0.3 m, L= 0.5 m)

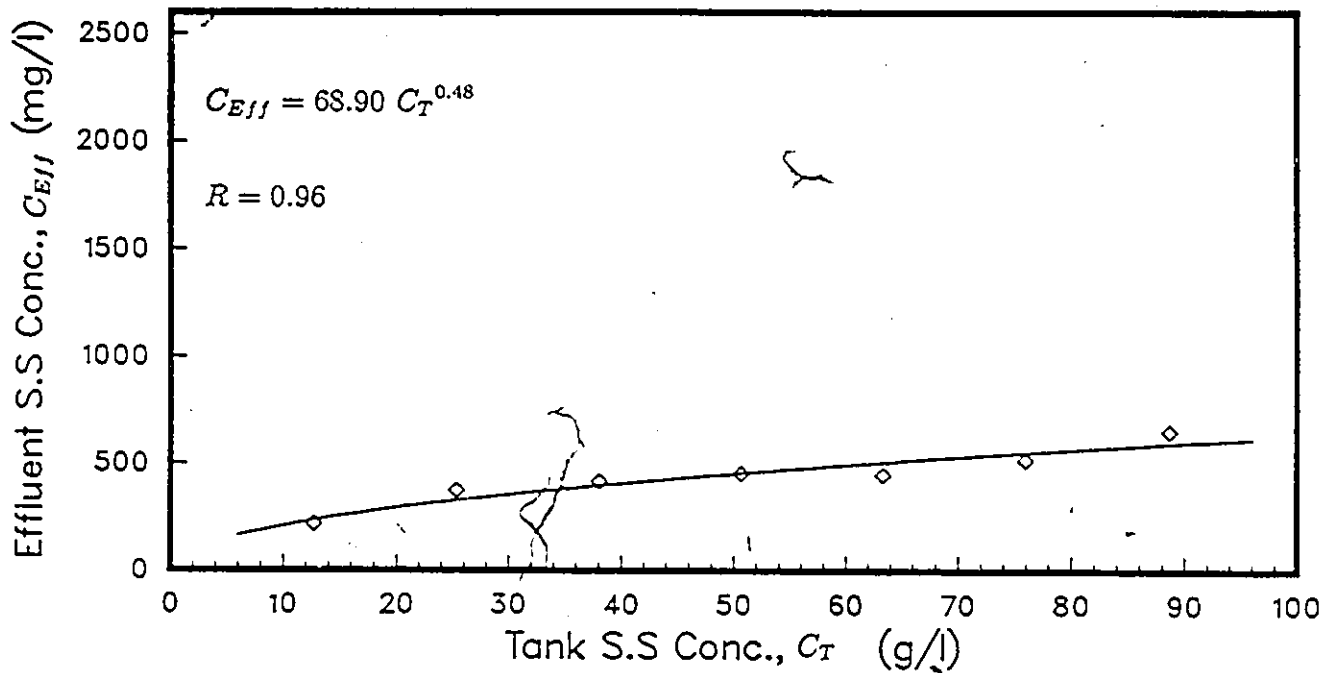


Figure A.13: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 30$ l, $D = 0.1$ m, $L = 0.1$ m)

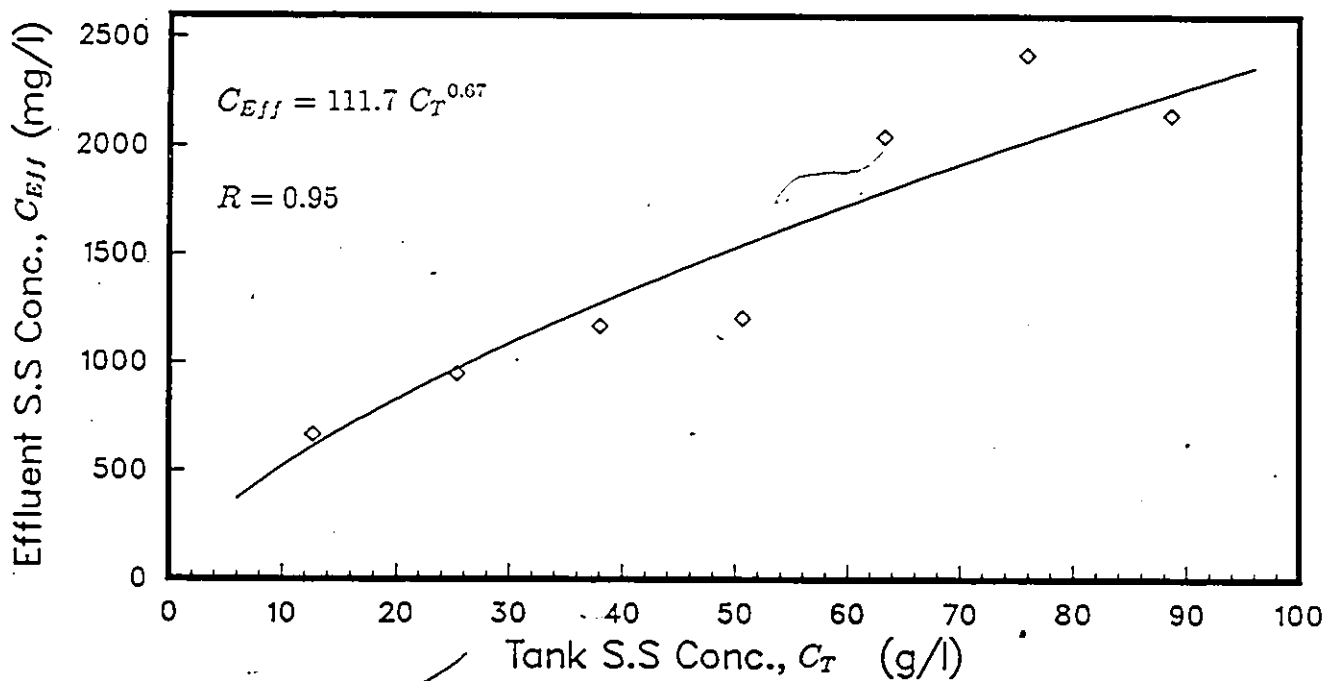


Figure A.14: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 30$ l, $D = 0.1$ m, $L = 0.5$ m)

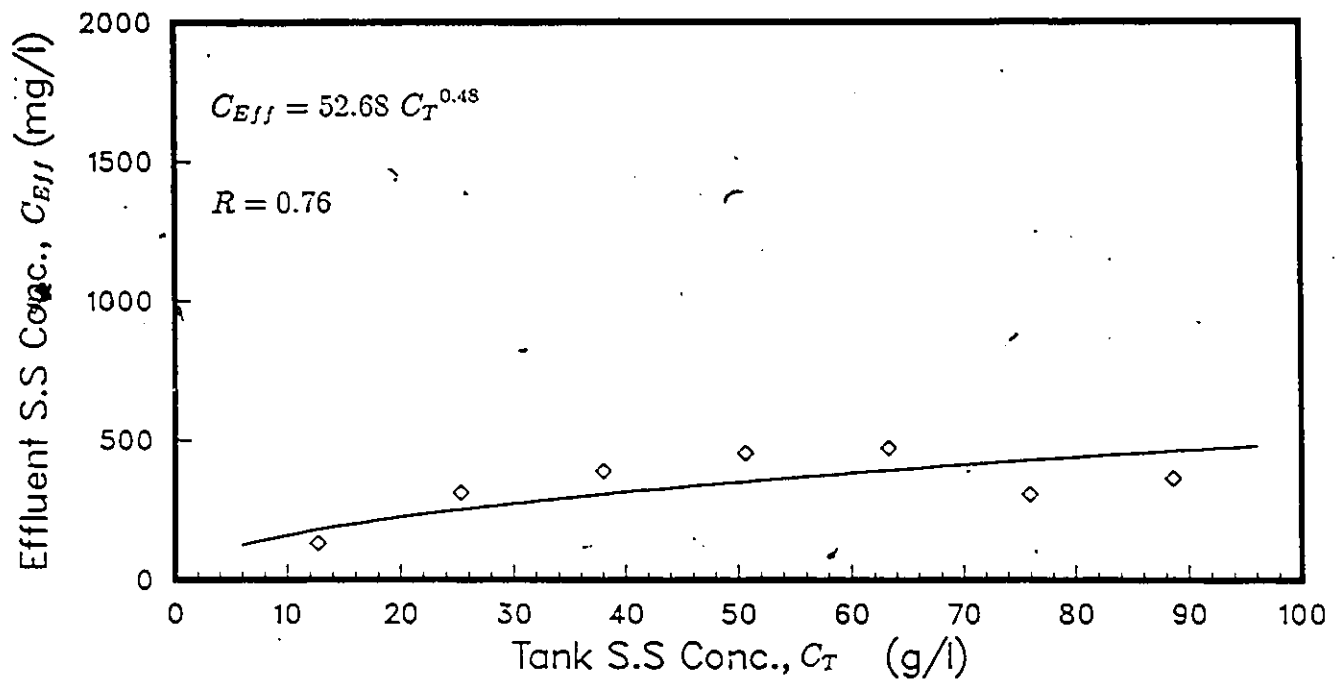


Figure A.15: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 15$ l, $D = 0.1$ m, $L = 0.3$ m)

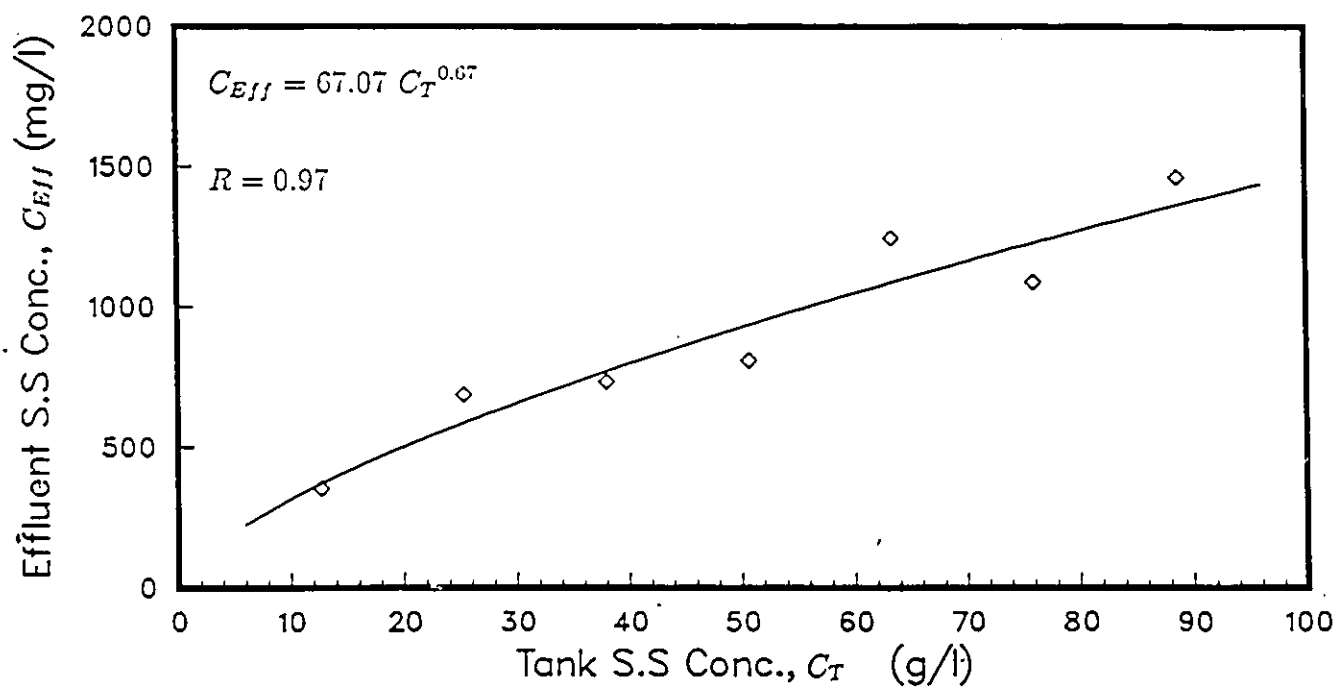


Figure A.16: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 45$ l, $D = 0.1$ m, $L = 0.3$ m)

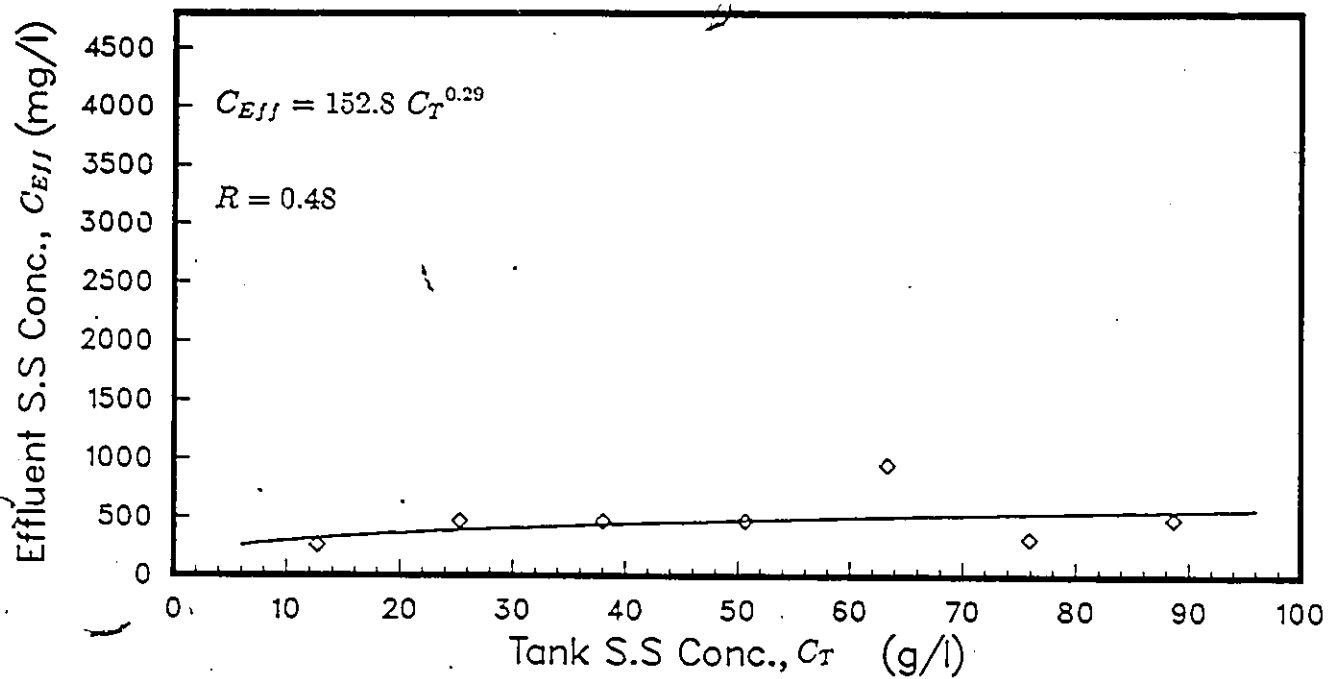


Figure A.17: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 15$ l, $D = 0.3$ m, $L = 0.5$ m)

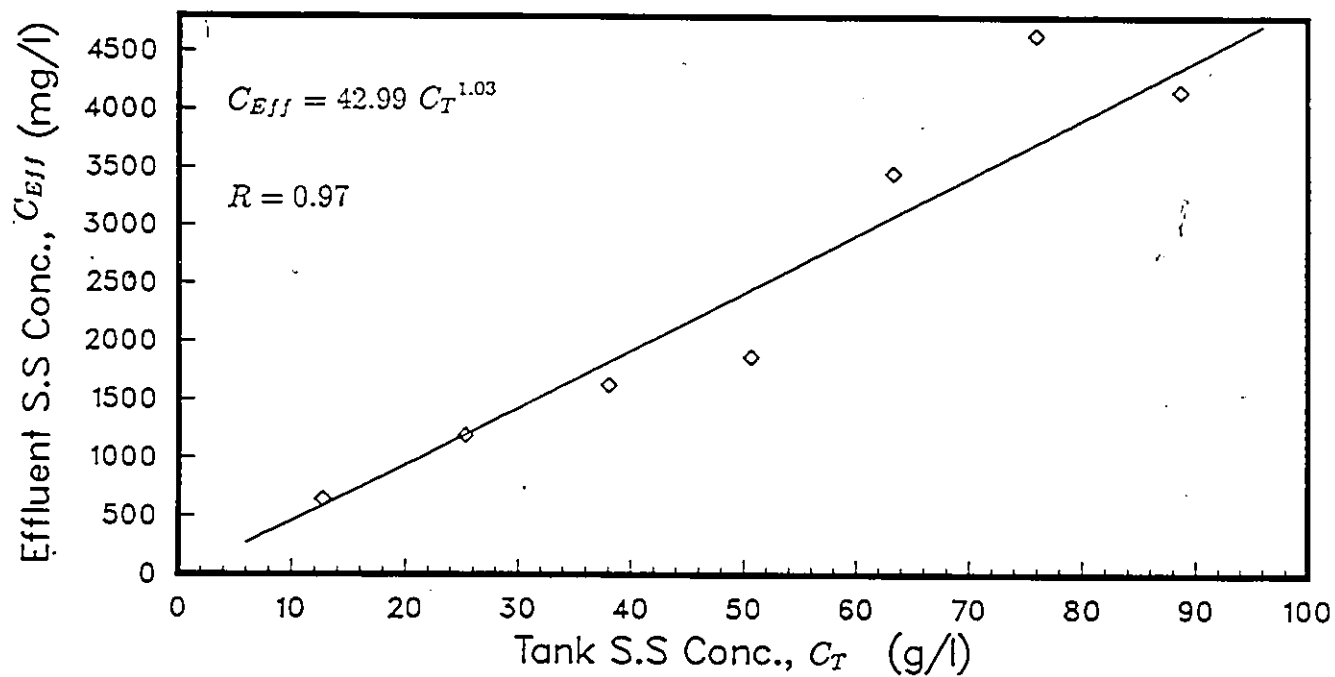


Figure A.18: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 45$ l, $D = 0.3$ m, $L = 0.5$ m)

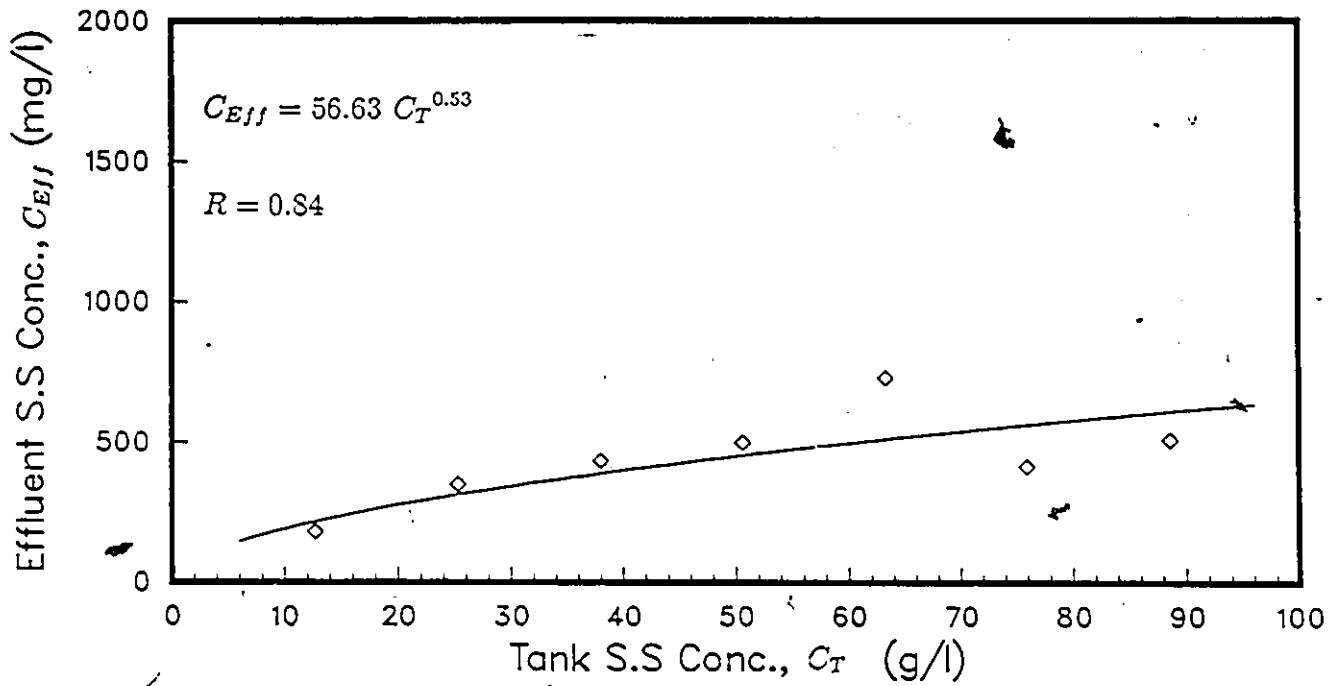


Figure A.19: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 15$ l, $D = 0.3$ m, $L = 0.1$ m)

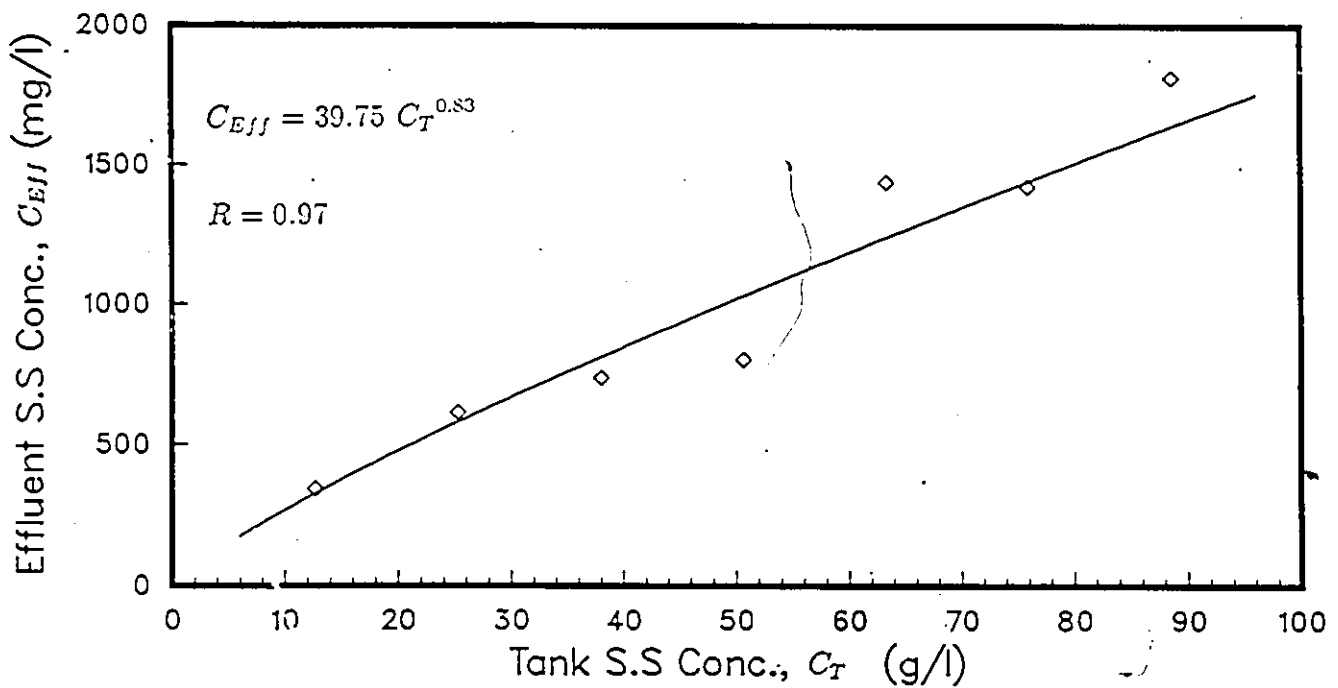


Figure A.20: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 45$ l, $D = 0.3$ m, $L = 0.1$ m)

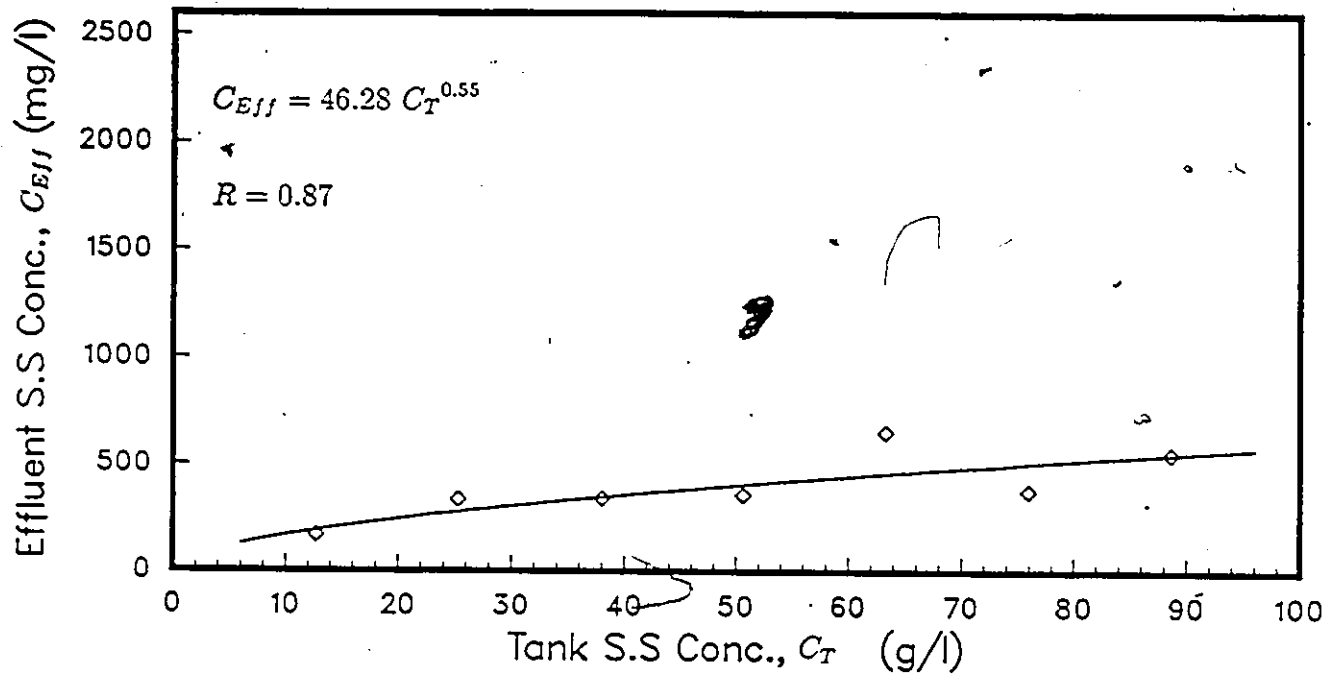


Figure A.21: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 15$ l, $D = 0.5$ m, $L = 0.3$ m)

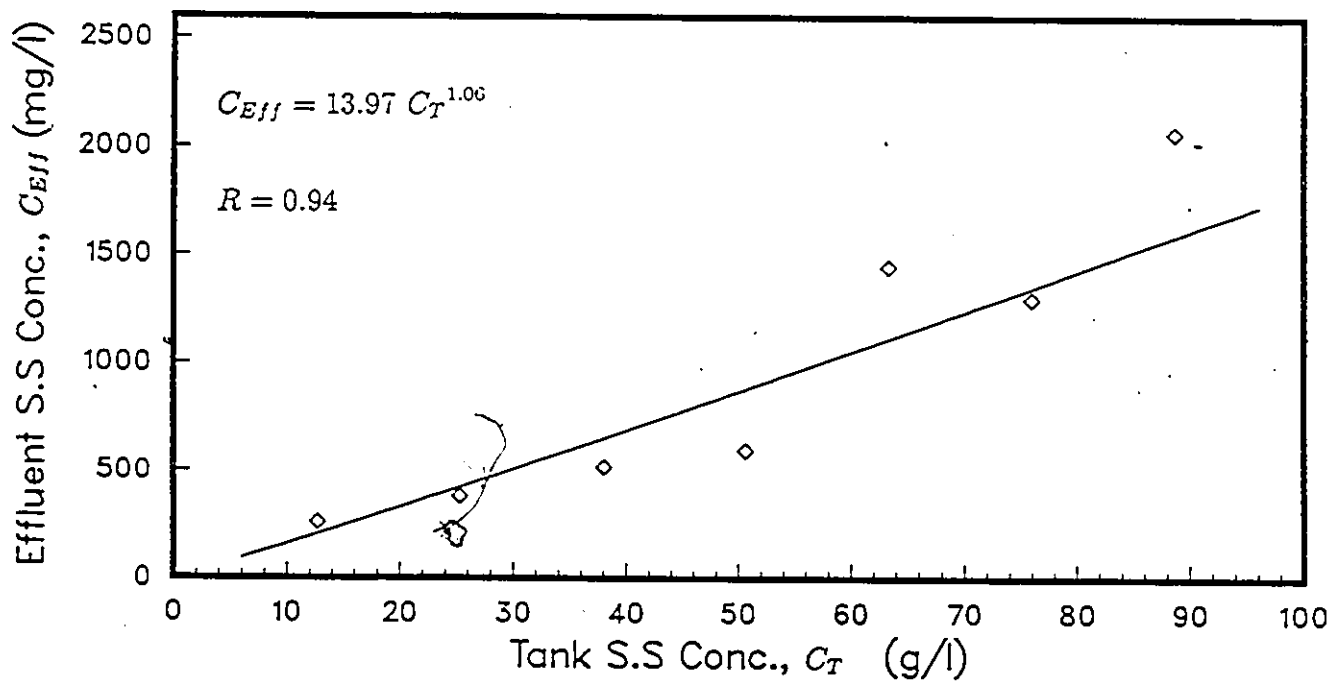


Figure A.22: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 45$ l, $D = 0.5$ m, $L = 0.3$ m)

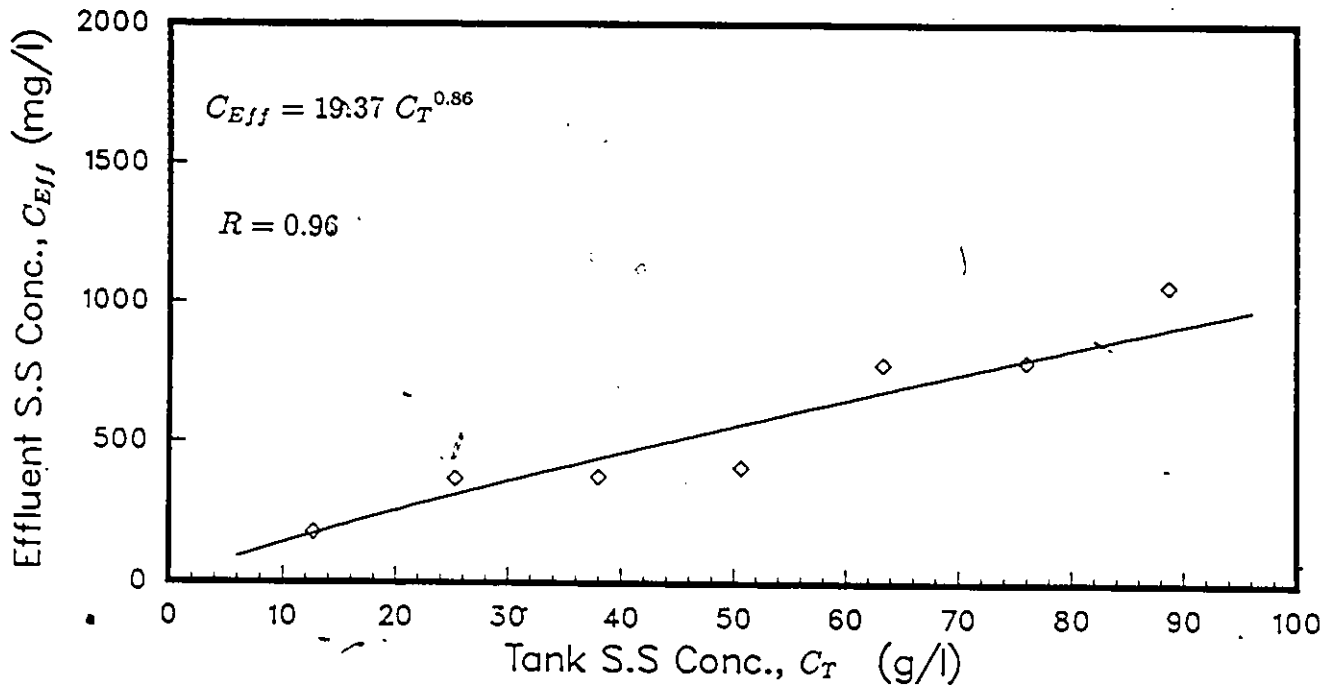


Figure A.23: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 30$ l, $D = 0.5$ m, $L = 0.1$ m)

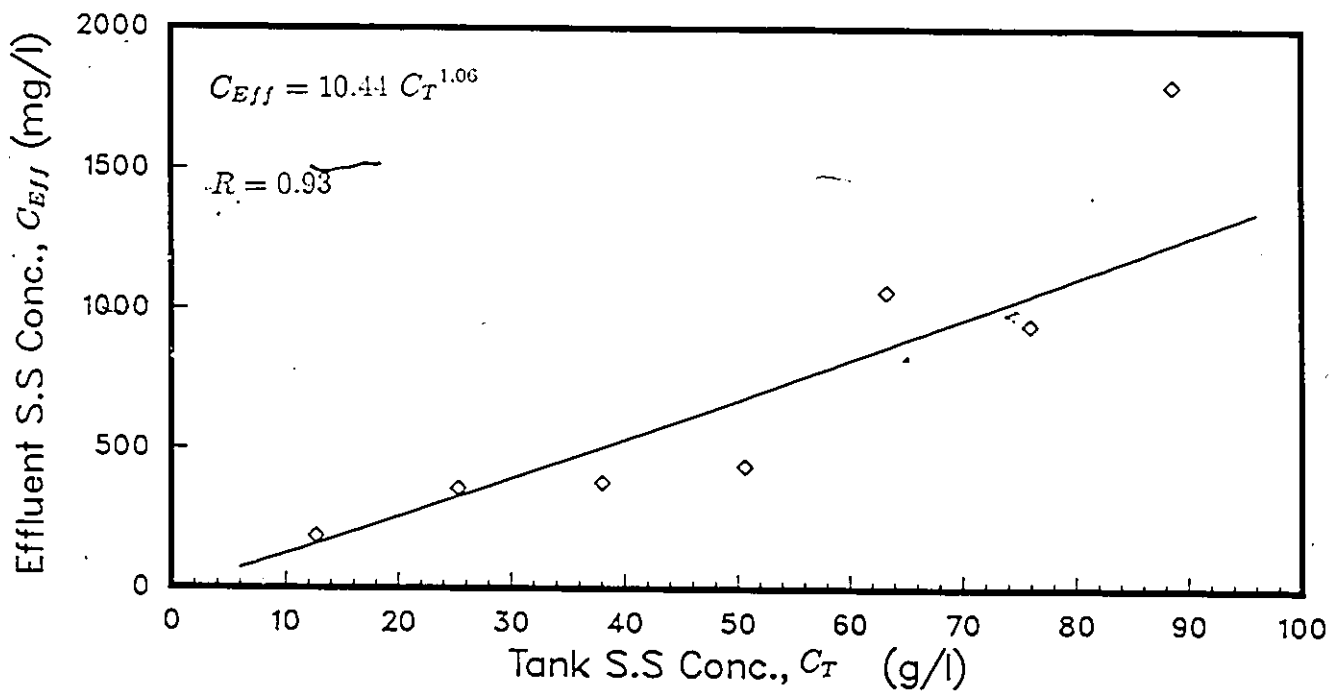


Figure A.24: Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 30$ l, $D = 0.5$ m, $L = 0.5$ m)

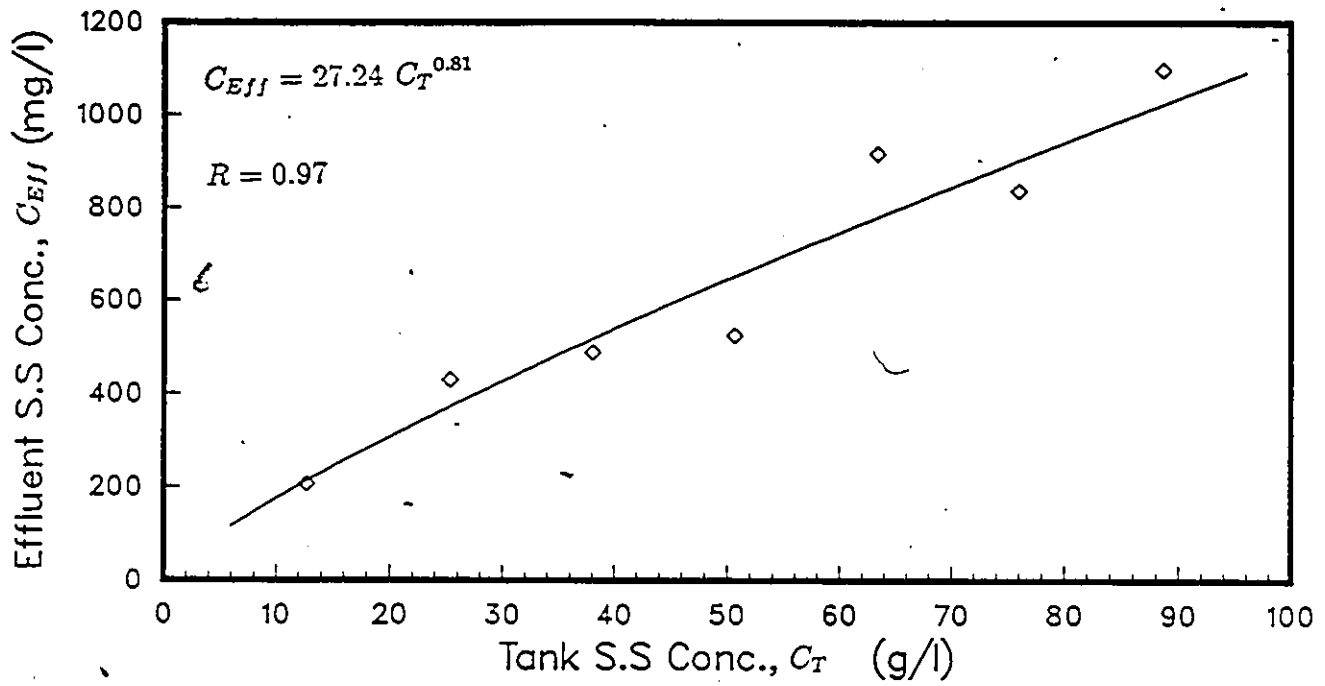


Figure A.25 : Effluent S.S. Conc. vs Tank S.S. Conc.
($S = 1:2$, $V = 30$ l, $D = 0.3$ m, $L = 0.3$ m)

APPENDIX B

COMPARISON BETWEEN MEASURED AND PREDICTED EFFLUENT SS

Note: Tank SS is a nominal concentration based on the total mass of solids in the total volume (See Table (4.2)).

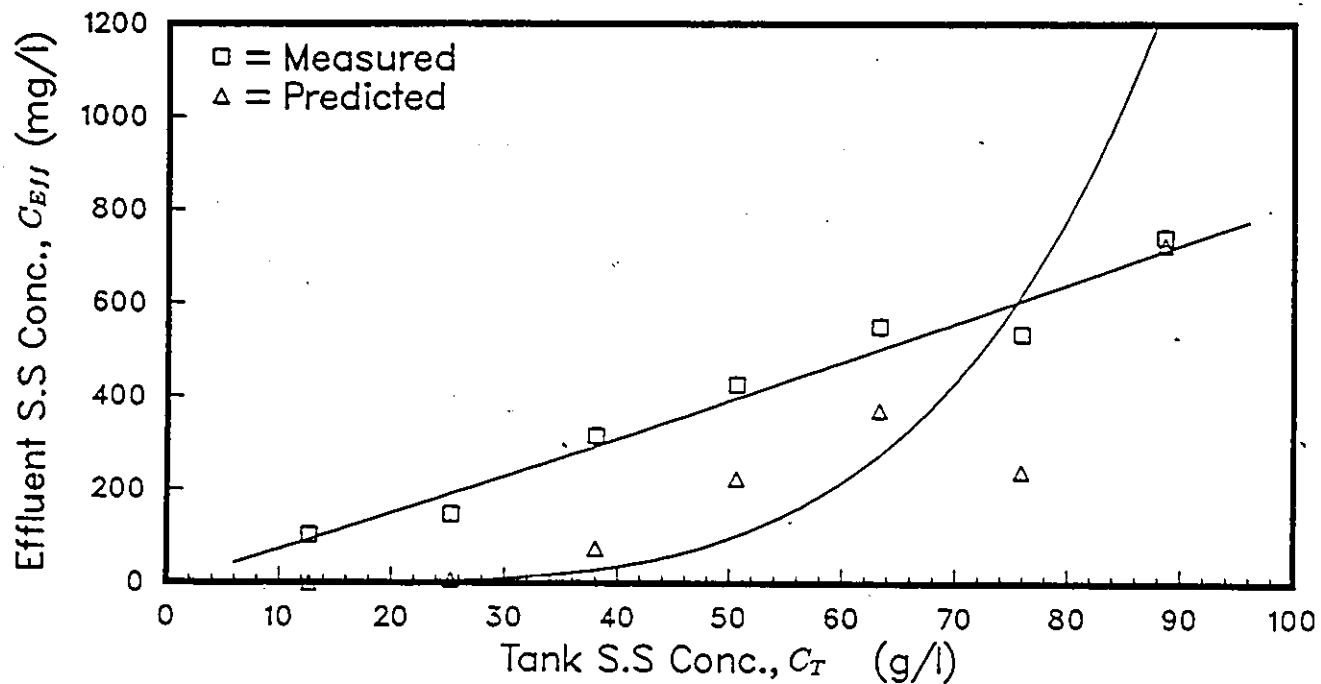


Figure B.1 : Plot of the Measured and the Predicted S.S. Conc.
(S = flat, V = 30 l, D = 0.5 m, L = 0.3 m)

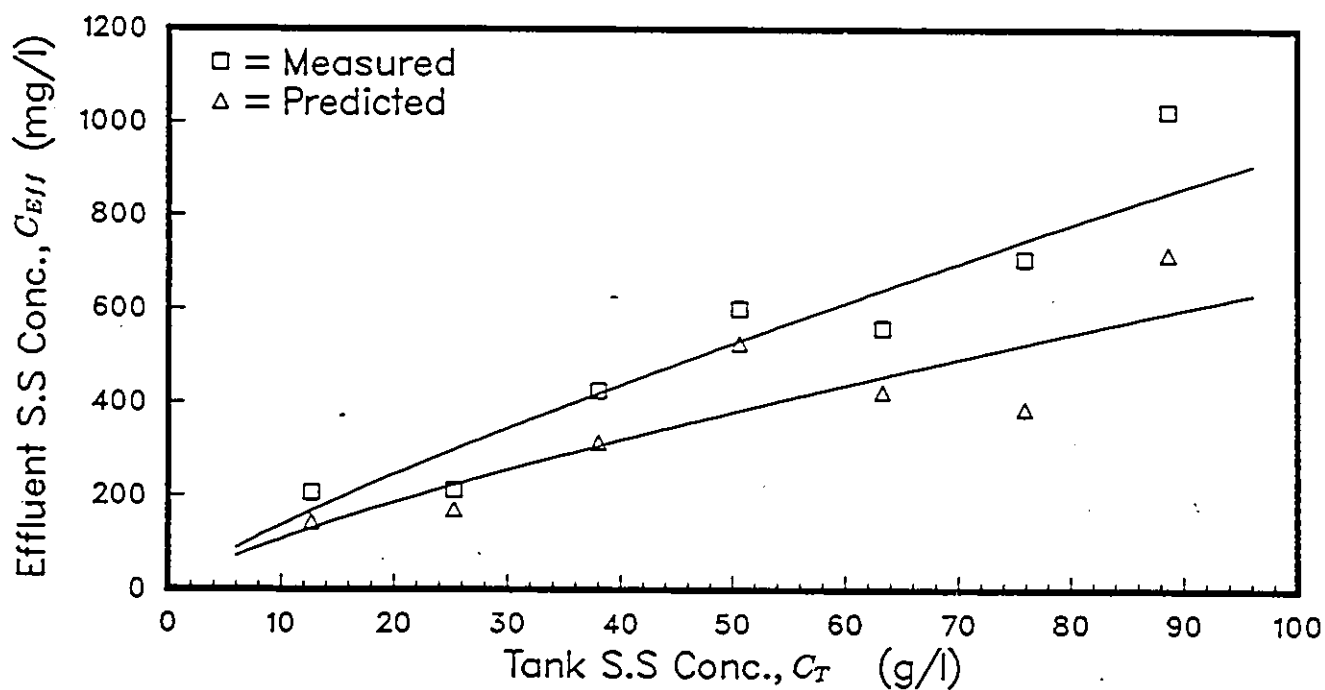


Figure B.2 : Plot of the Measured and the Predicted S.S. Conc.
(S = flat, V = 30 l, D = 0.1 m, L = 0.3 m)

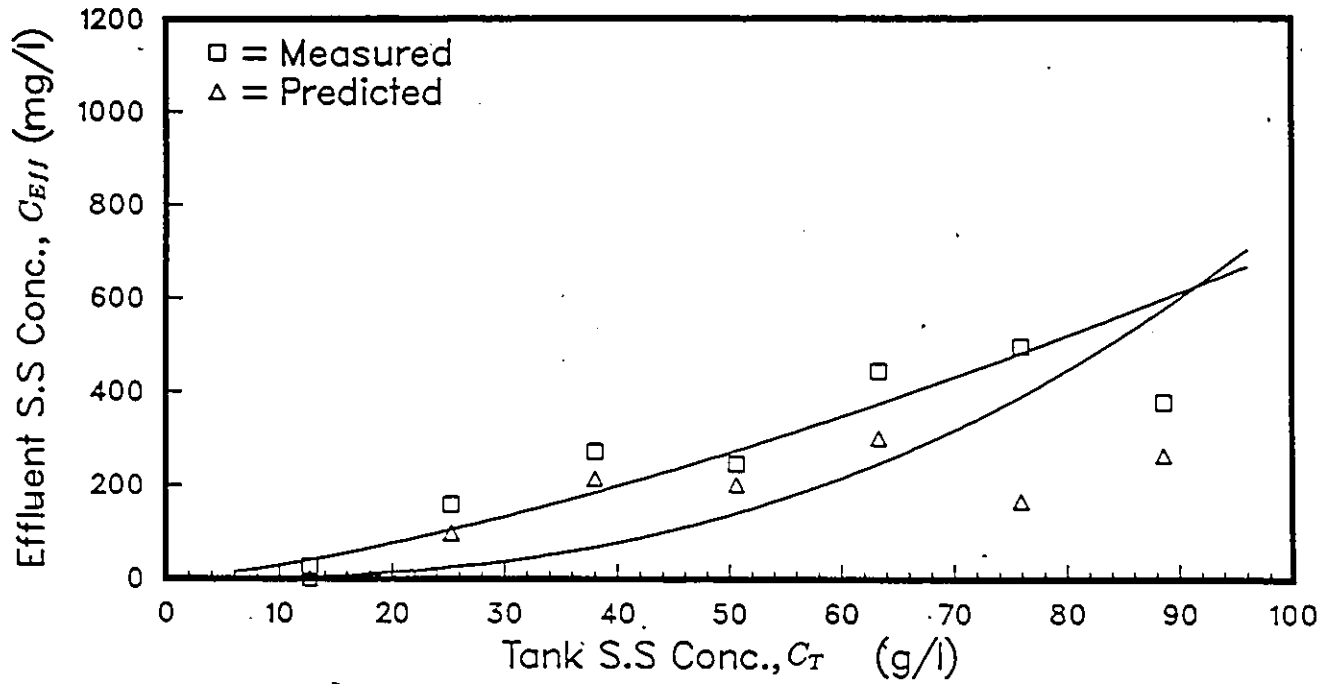


Figure B.3 : Plot of the Measured and the Predicted S.S. Conc.
(S = flat, V = 15 l, D = 0.3 m, L = 0.3 m)

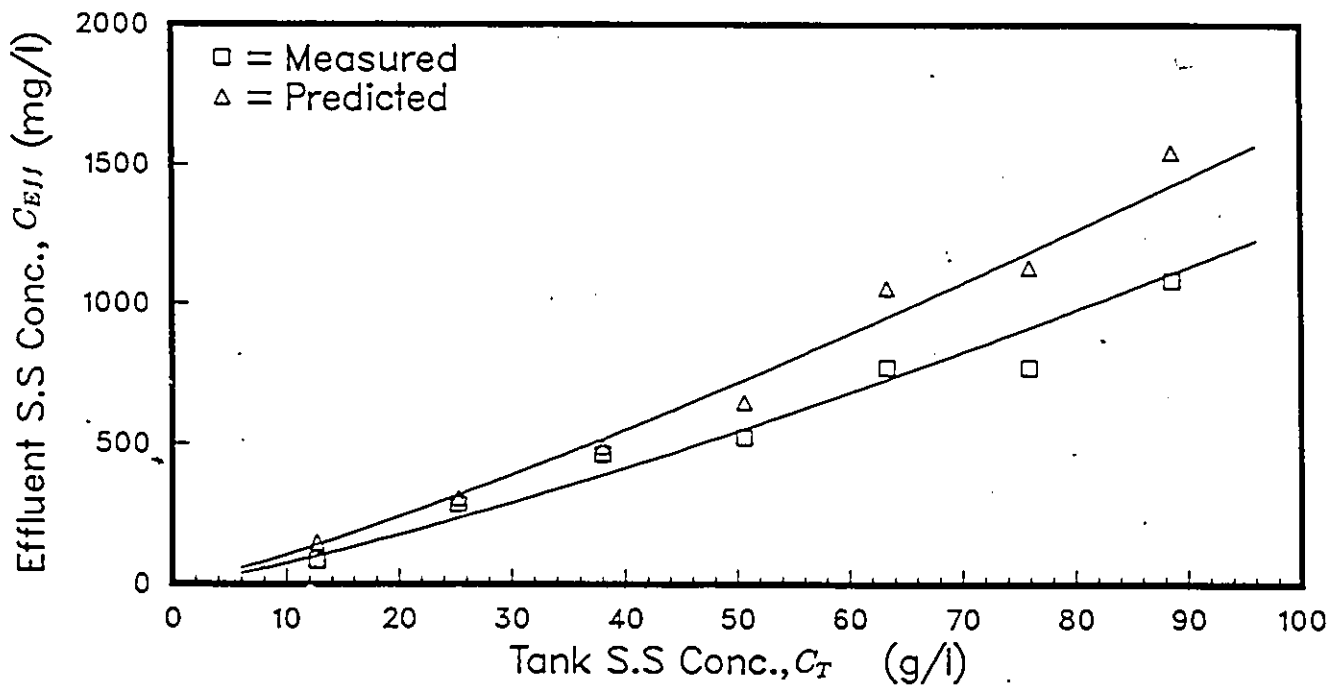


Figure B.4 : Plot of the Measured and the Predicted S.S. Conc.
(S = flat, V = 45 l, D = 0.3 m, L = 0.3 m)

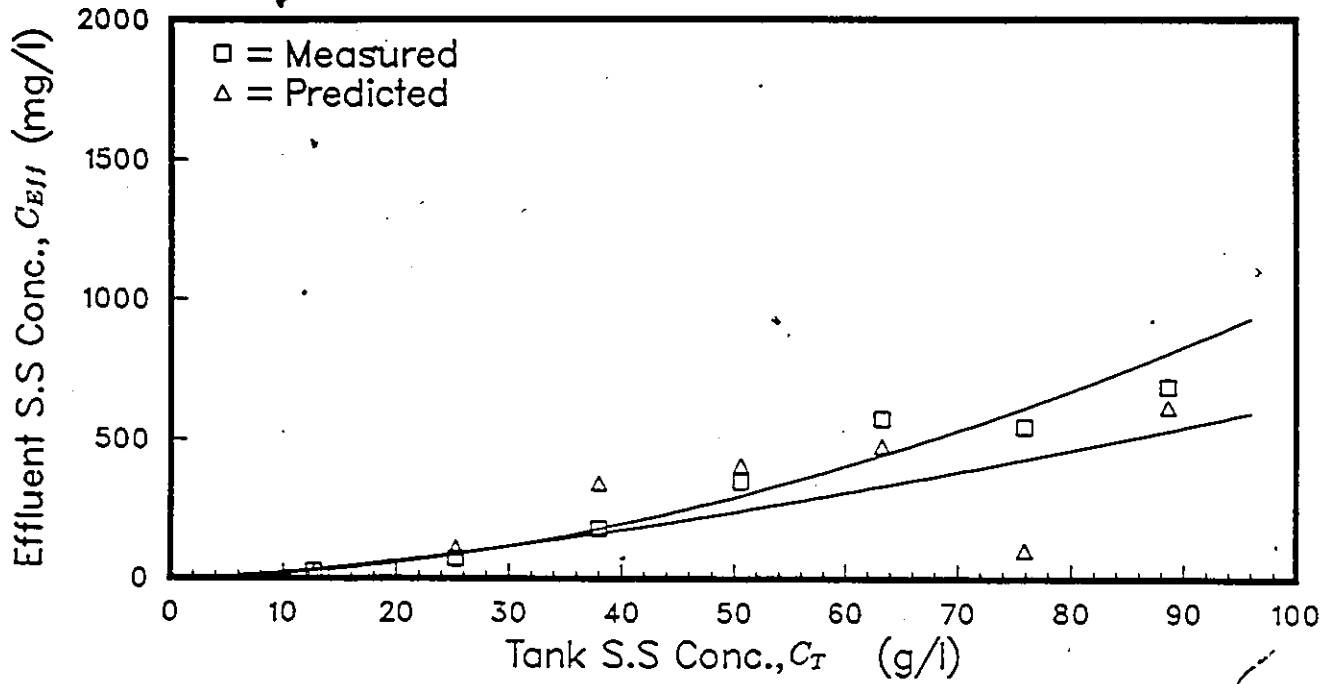


Figure B.5 : Plot of the Measured and the Predicted S.S. Conc. (S= flat, V= 30 l, D= 0.3 m, L= 0.1 m)

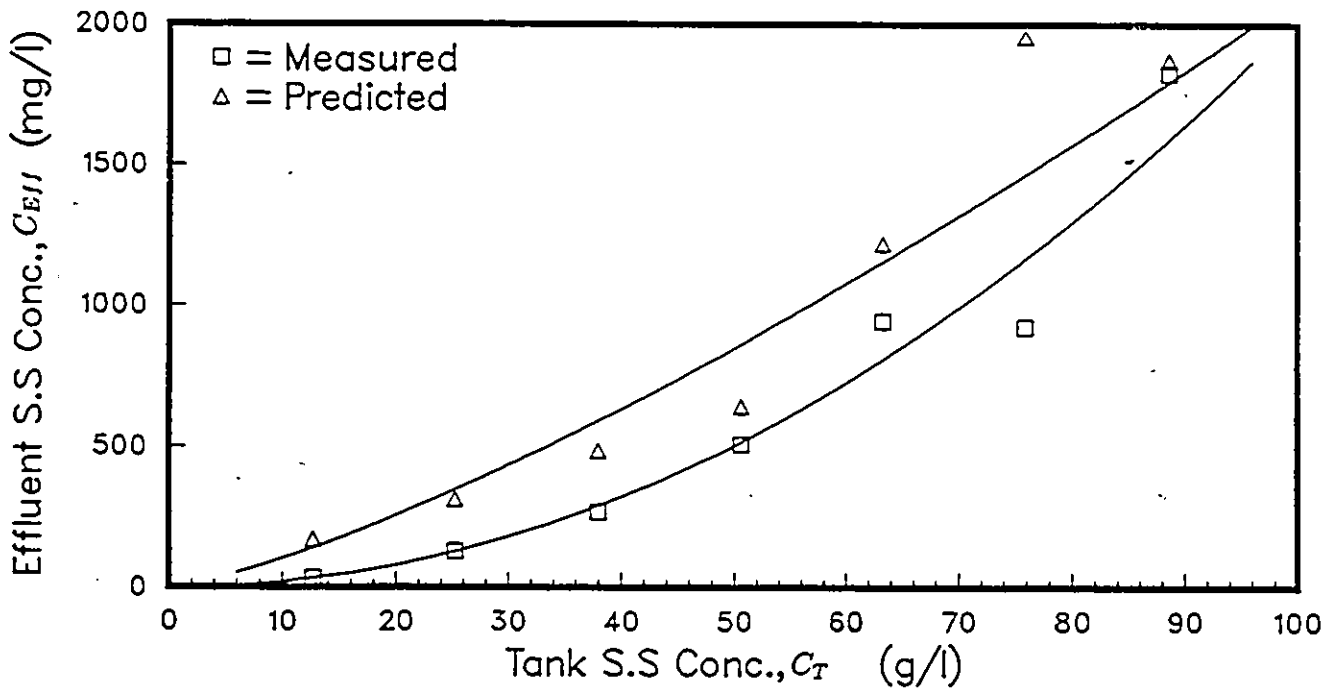


Figure B.6 : Plot of the Measured and the Predicted S.S. Conc. (S= flat, V= 30 l, D= 0.3 m, L= 0.5 m)

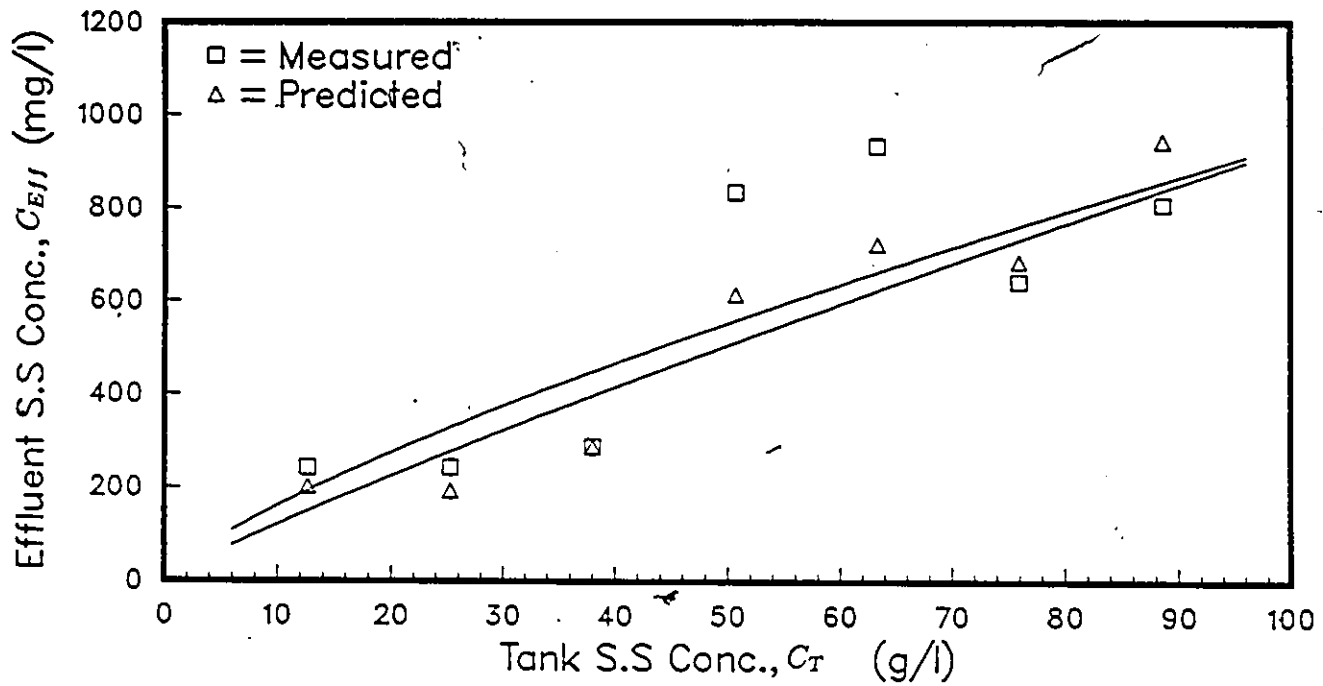


Figure B.7 : Plot of the Measured and the Predicted S.S Conc.
($S= 1:1$, $V= 30$ l, $D= 0.5$ m, $L= 0.3$ m)

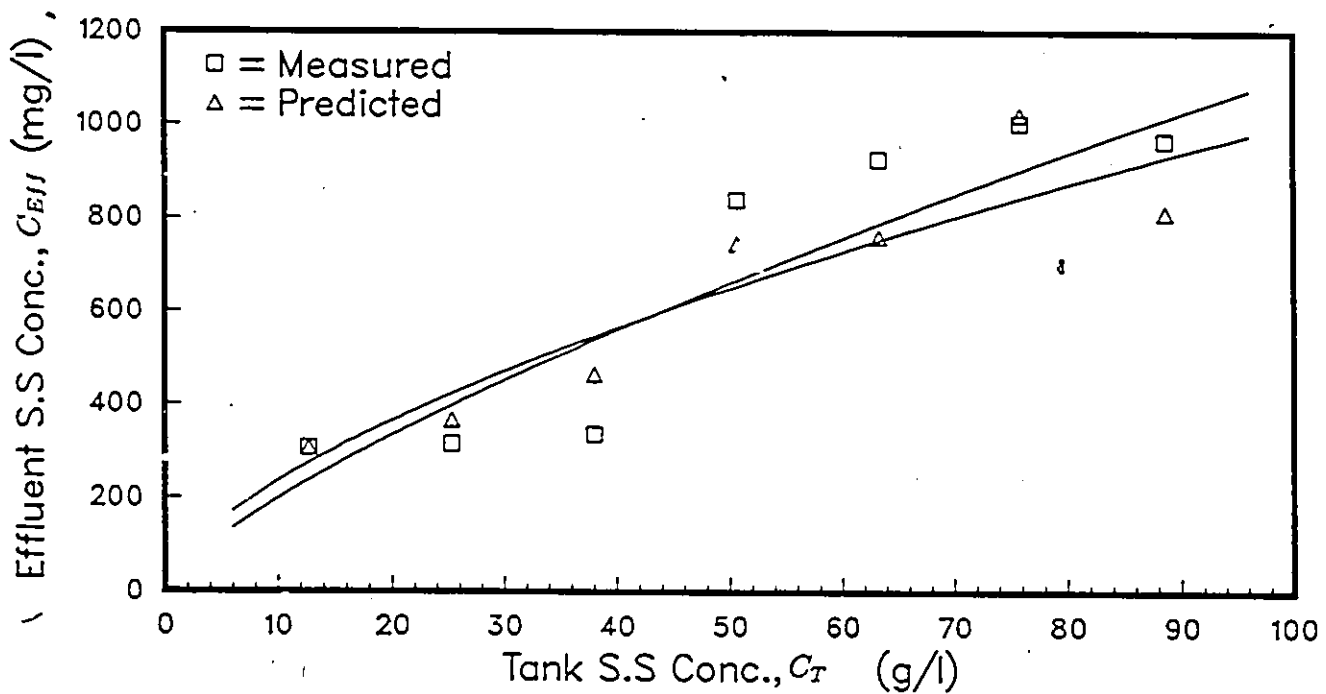


Figure B.8 : Plot of the Measured and the Predicted S.S Conc.
($S= 1:1$, $V= 30$ l, $D= 0.1$ m, $L= 0.3$ m)

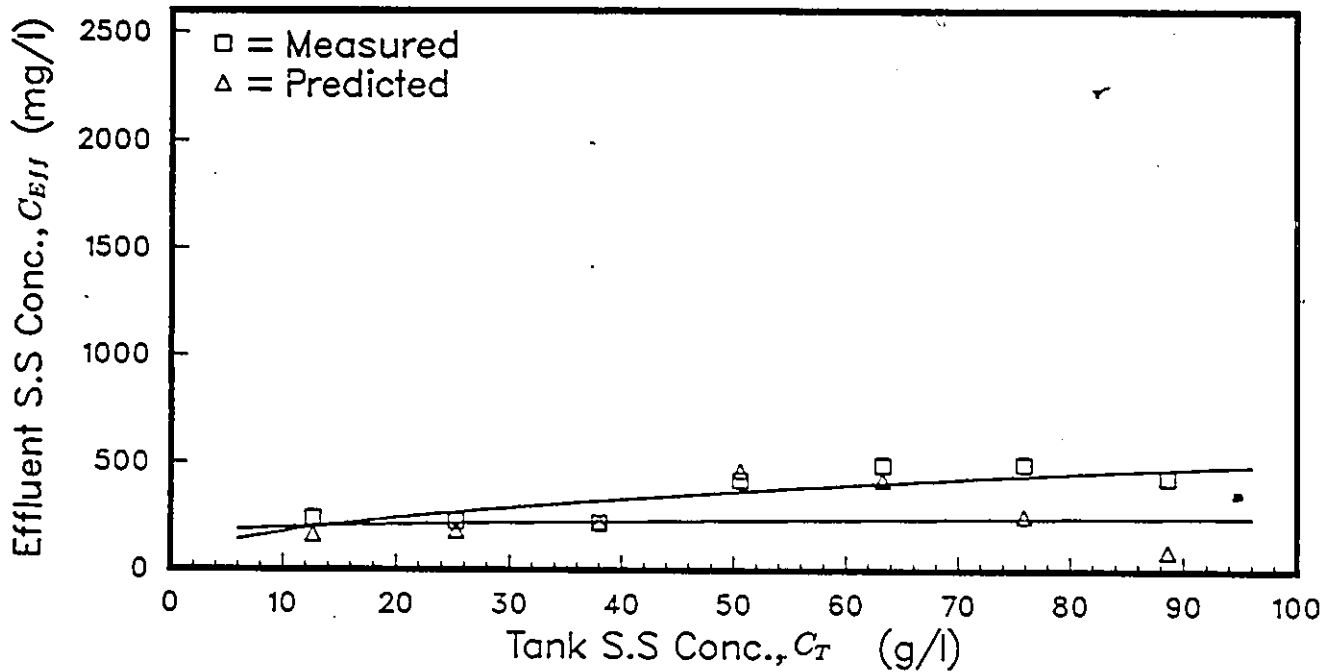


Figure B.9 : Plot of the Measured and the Predicted S.S Conc.
($S= 1:1$, $V= 15$ l, $D= 0.3$ m, $L= 0.3$ m)

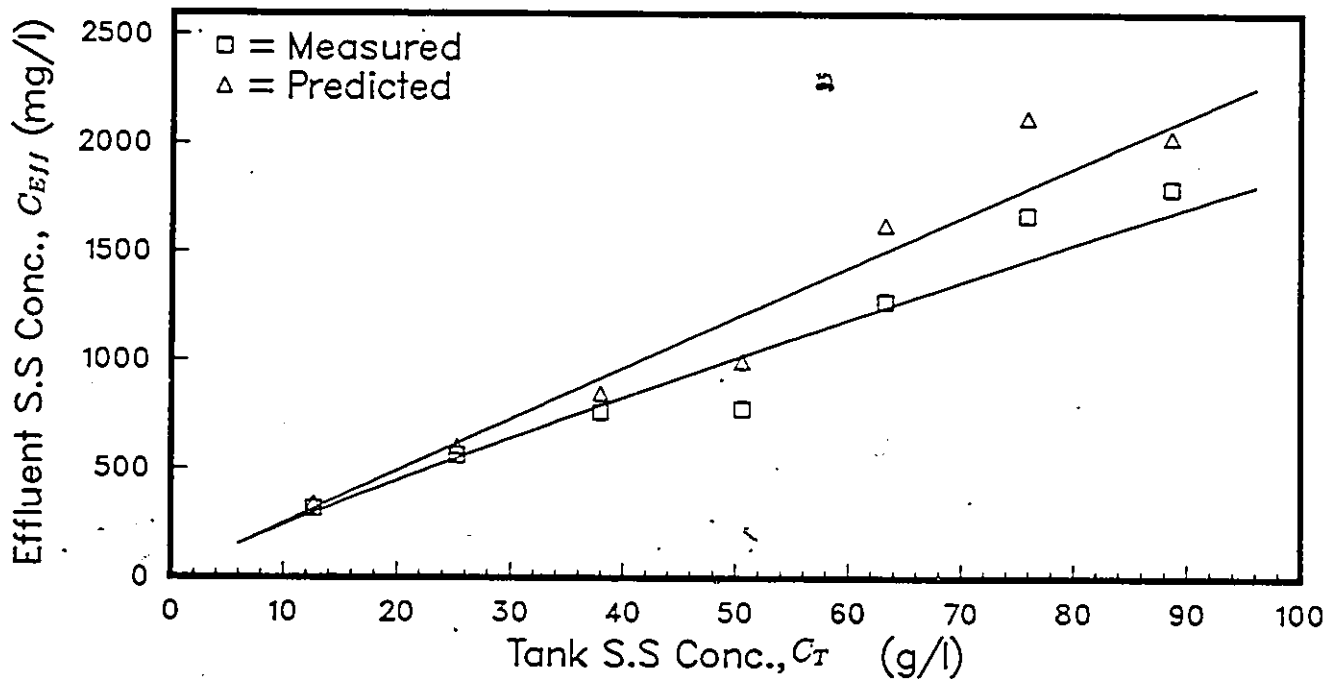


Figure B.10: Plot of the Measured and the Predicted S.S Conc.
($S= 1:1$, $V= 45$ l, $D= 0.3$ m, $L= 0.3$ m)

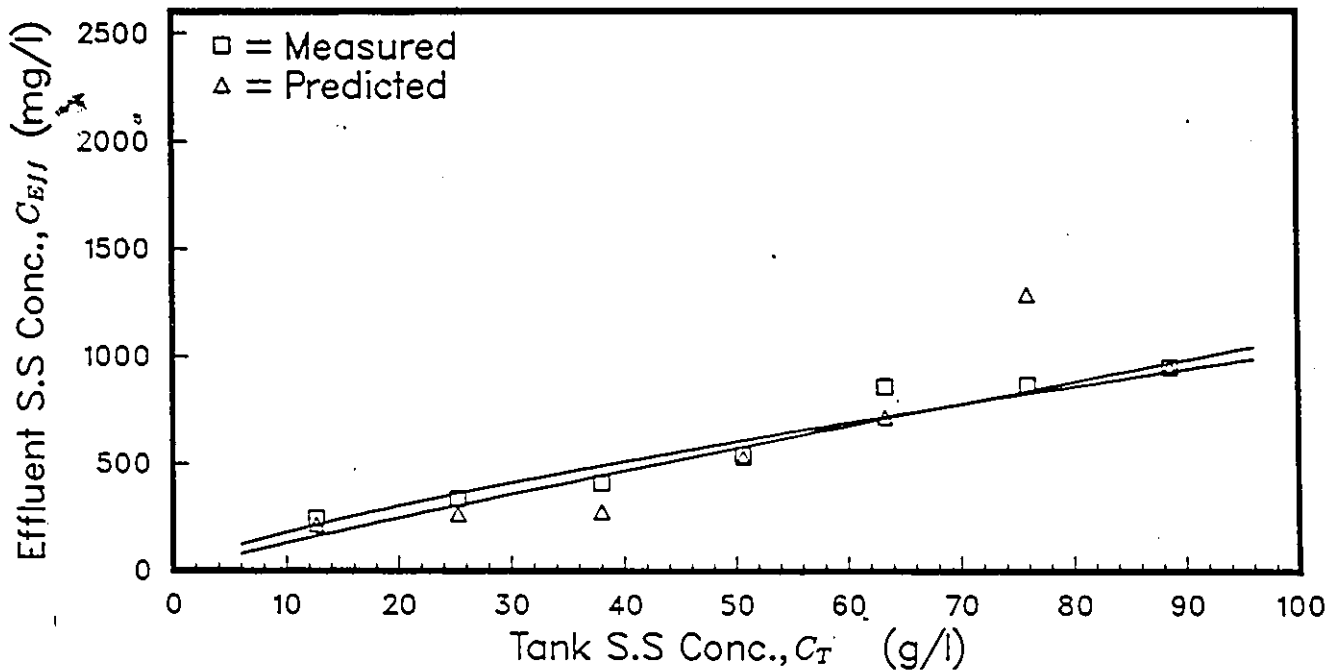


Figure B.11: Plot of the Measured and the Predicted S.S. Conc.
($S= 1:1$, $V= 30$ l, $D= 0.3$ m, $L= 0.1$ m)

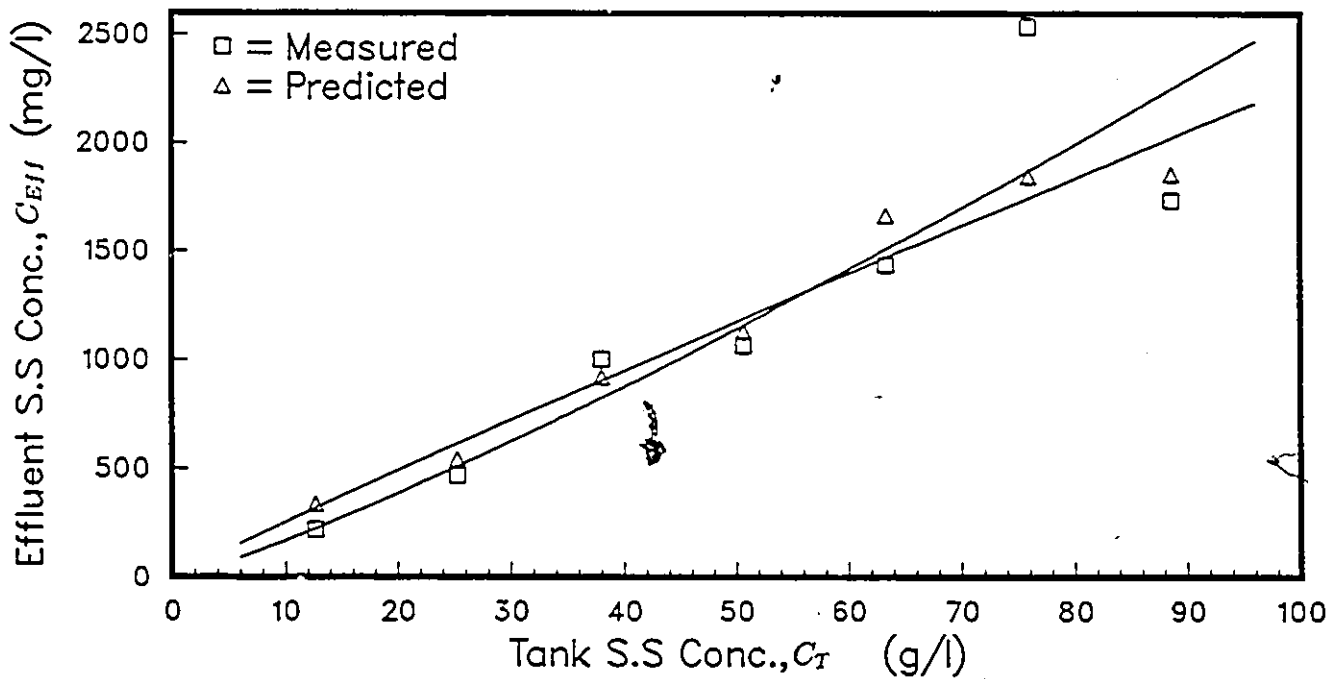


Figure B.12: Plot of the Measured and the Predicted S.S. Conc.
($S= 1:1$, $V= 30$ l, $D= 0.3$ m, $L= 0.5$ m)

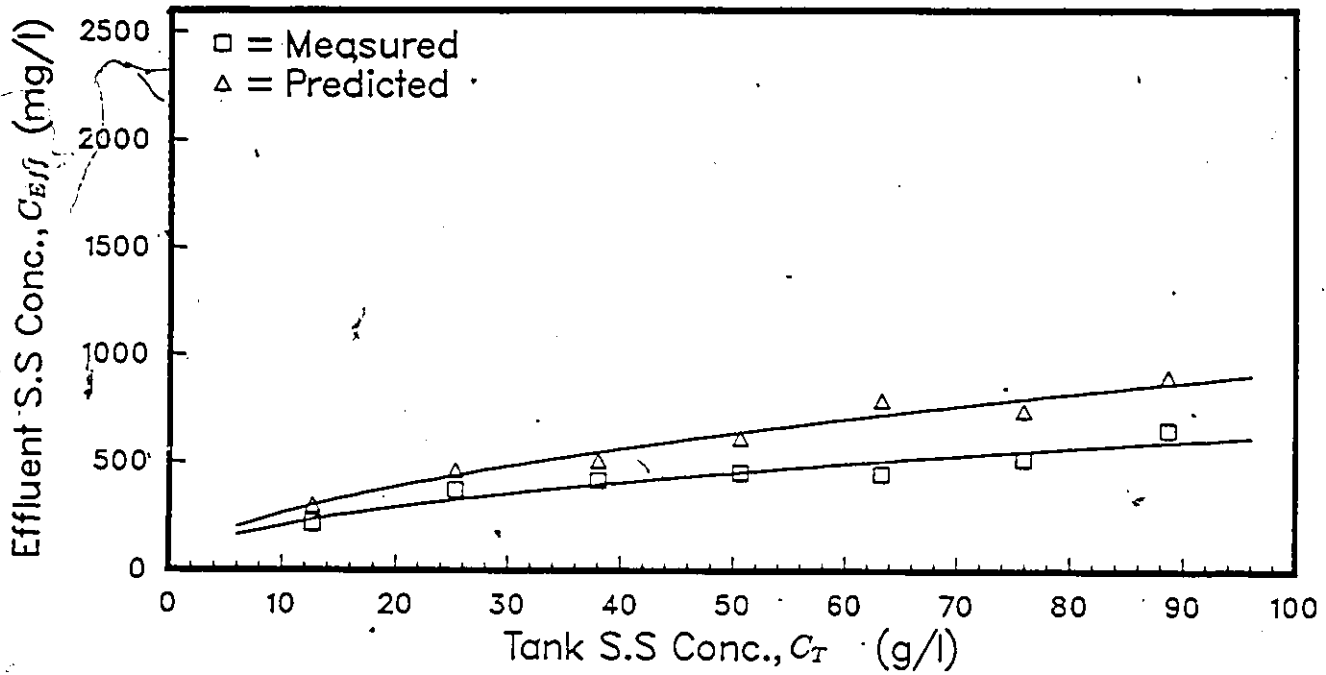


Figure B.13: Plot of the Measured and the Predicted S.S. Conc. ($S=1:2$, $V=30$ l, $D=0.1$ m, $L=0.1$ m)

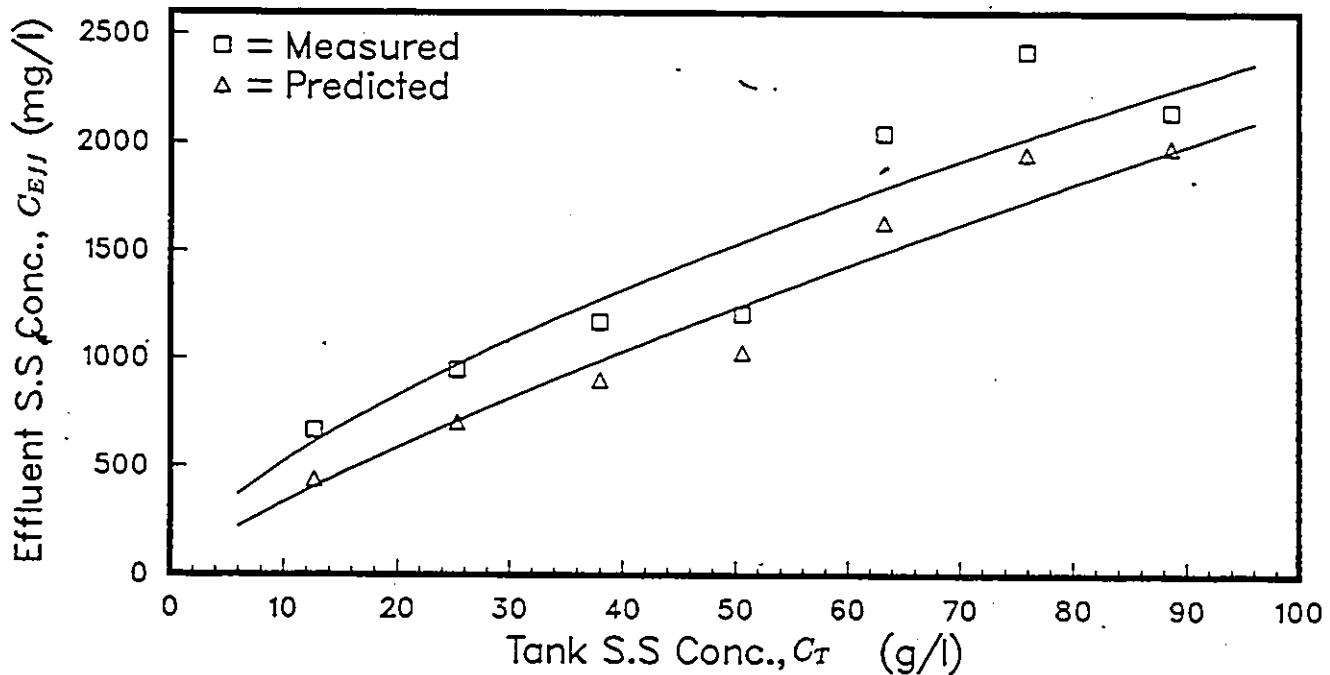


Figure B.14: Plot of the Measured and the Predicted S.S. Conc. ($S=1:2$, $V=30$ l, $D=0.1$ m, $L=0.5$ m)

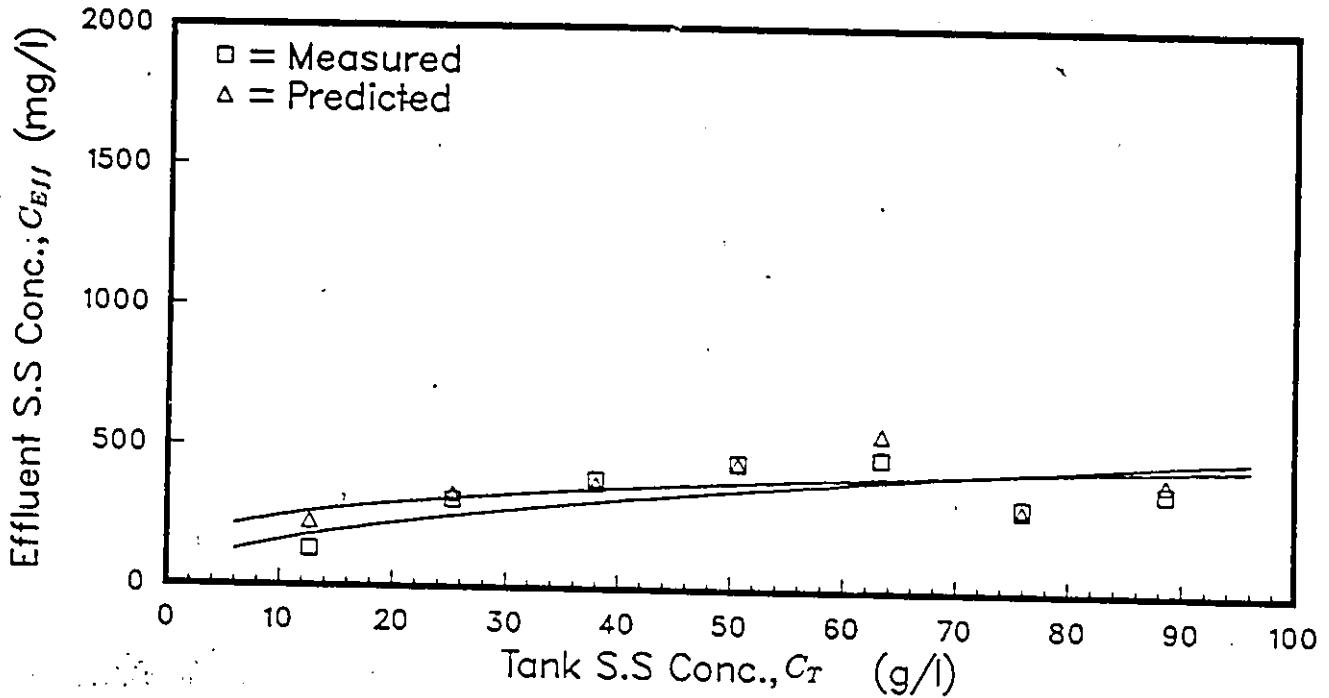


Figure B.15: Plot of the Measured and the Predicted S.S. Conc. ($S=1:2$, $V=15$ l, $D=0.1$ m, $L=0.3$ m)

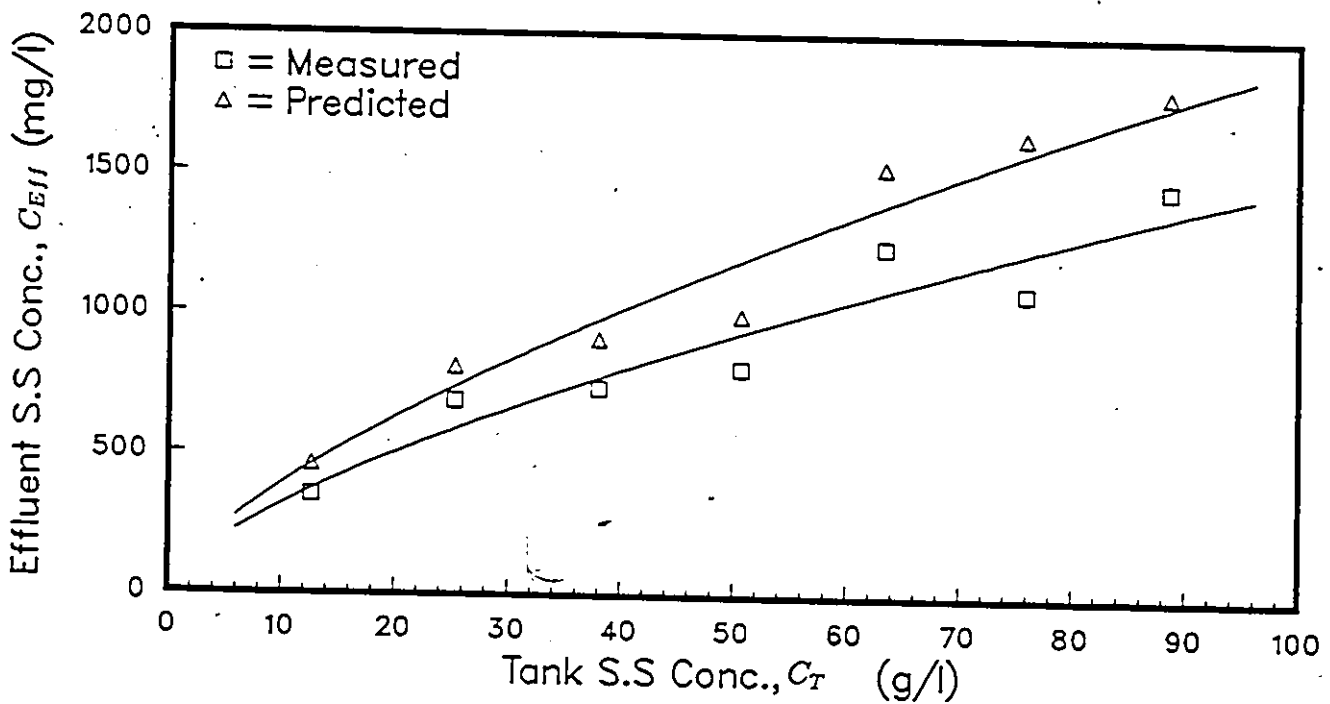


Figure B.16: Plot of the Measured and the Predicted S.S. Conc. ($S=1:2$, $V=45$ l, $D=0.1$ m, $L=0.3$ m)

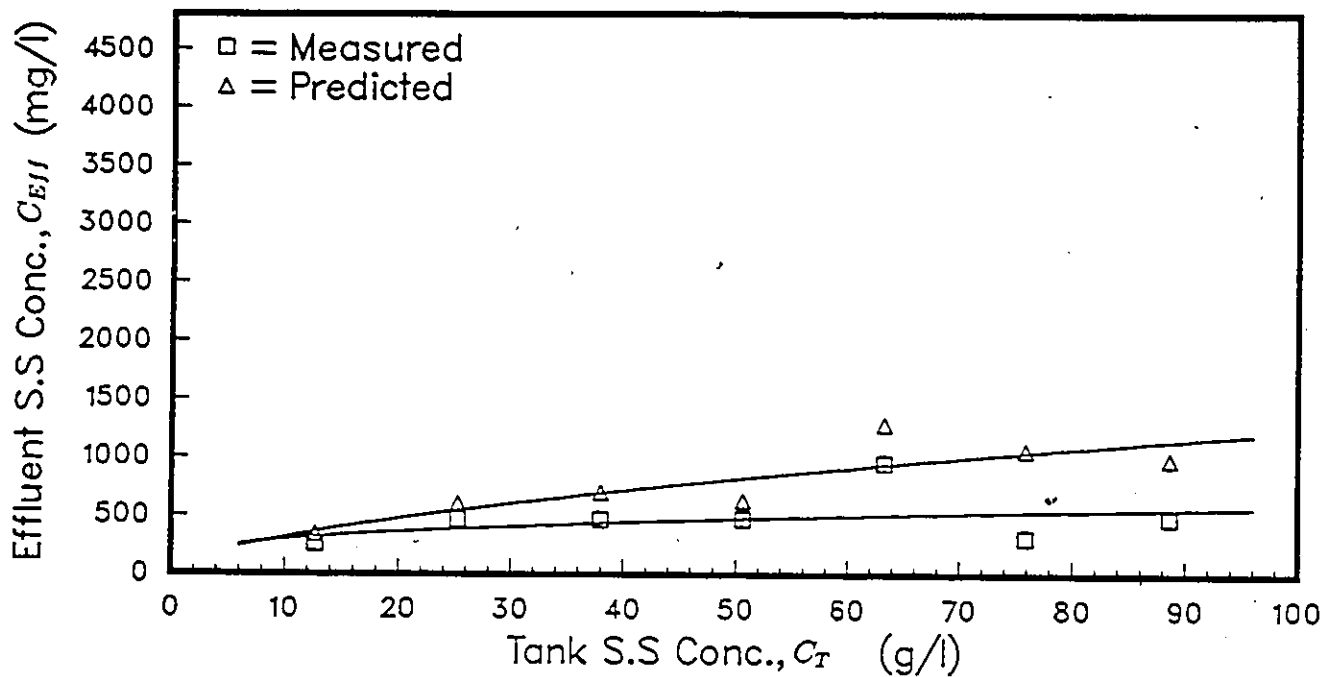


Figure B.17: Plot of the Measured and the Predicted S.S. Conc.
($S=1:2$, $V=15$ l, $D=0.3$ m, $L=0.5$ m)

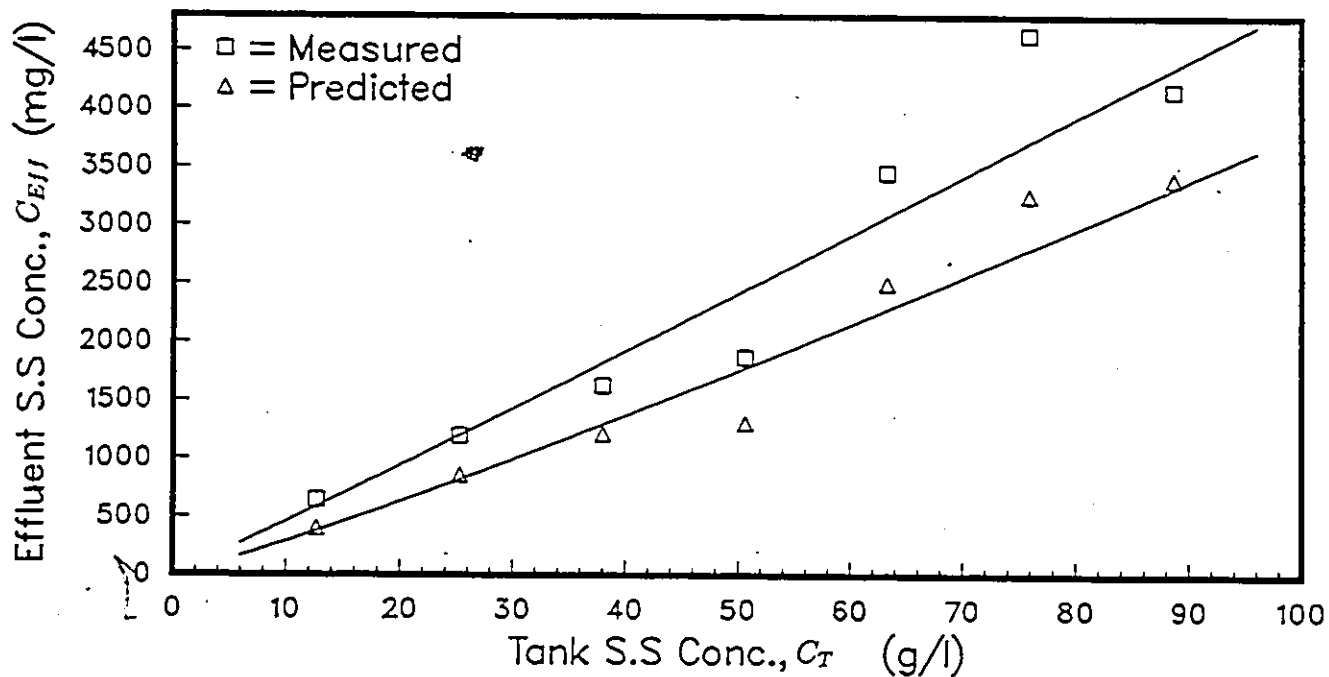


Figure B.18: Plot of the Measured and the Predicted S.S. Conc.
($S=1:2$, $V=45$ l, $D=0.3$ m, $L=0.5$ m)

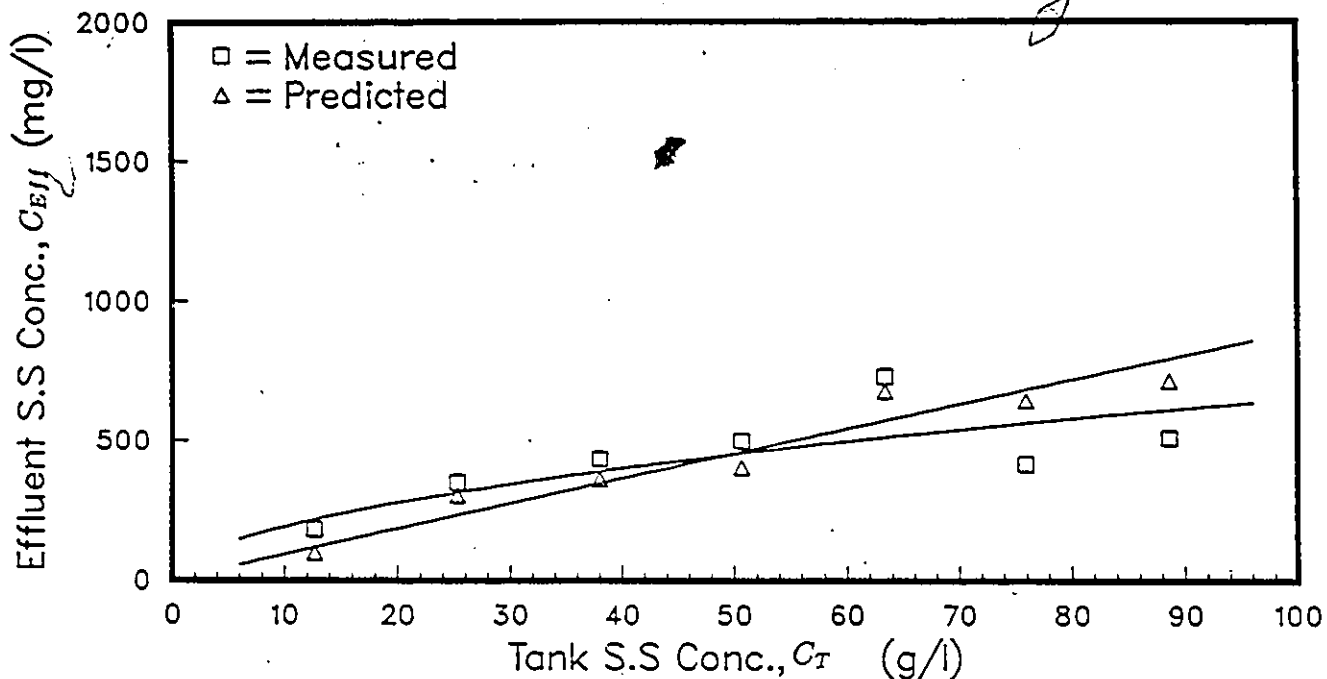


Figure B.19: Plot of the Measured and the Predicted S.S Conc.
($S = 1:2$, $V = 15$ l, $D = 0.3$ m, $L = 0.1$ m)

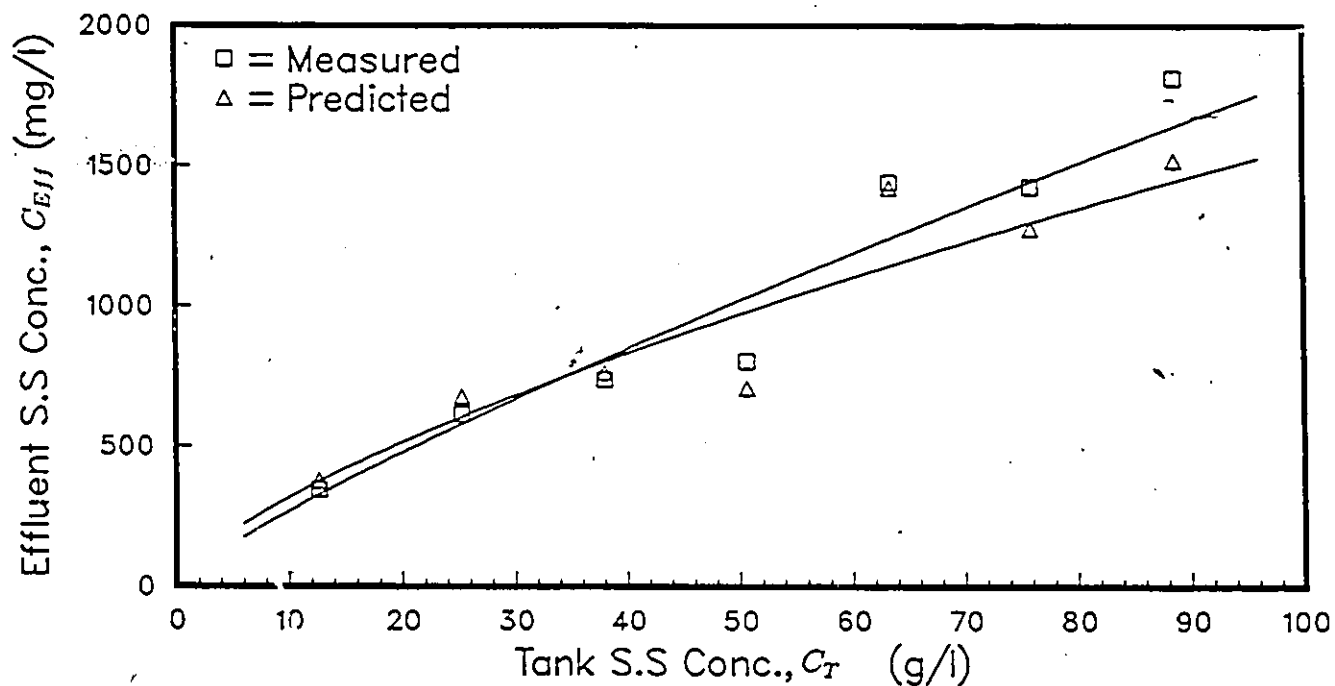


Figure B.20 : Plot of the Measured and the Predicted S.S Conc.
($S = 1:2$, $V = 45$ l, $D = 0.3$ m, $L = 0.1$ m)

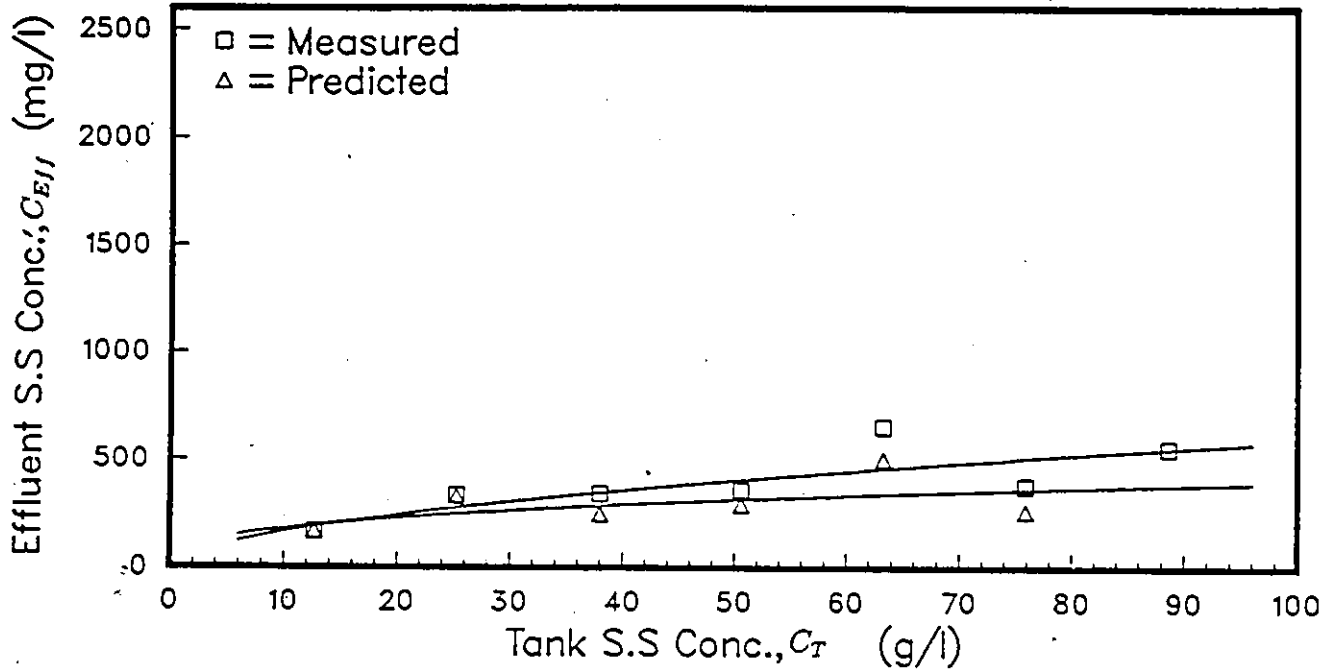


Figure B.21: Plot of the Measured and the Predicted S.S. Conc. ($S=1:2$, $V=15$ l, $D=0.5$ m, $L=0.3$ m)

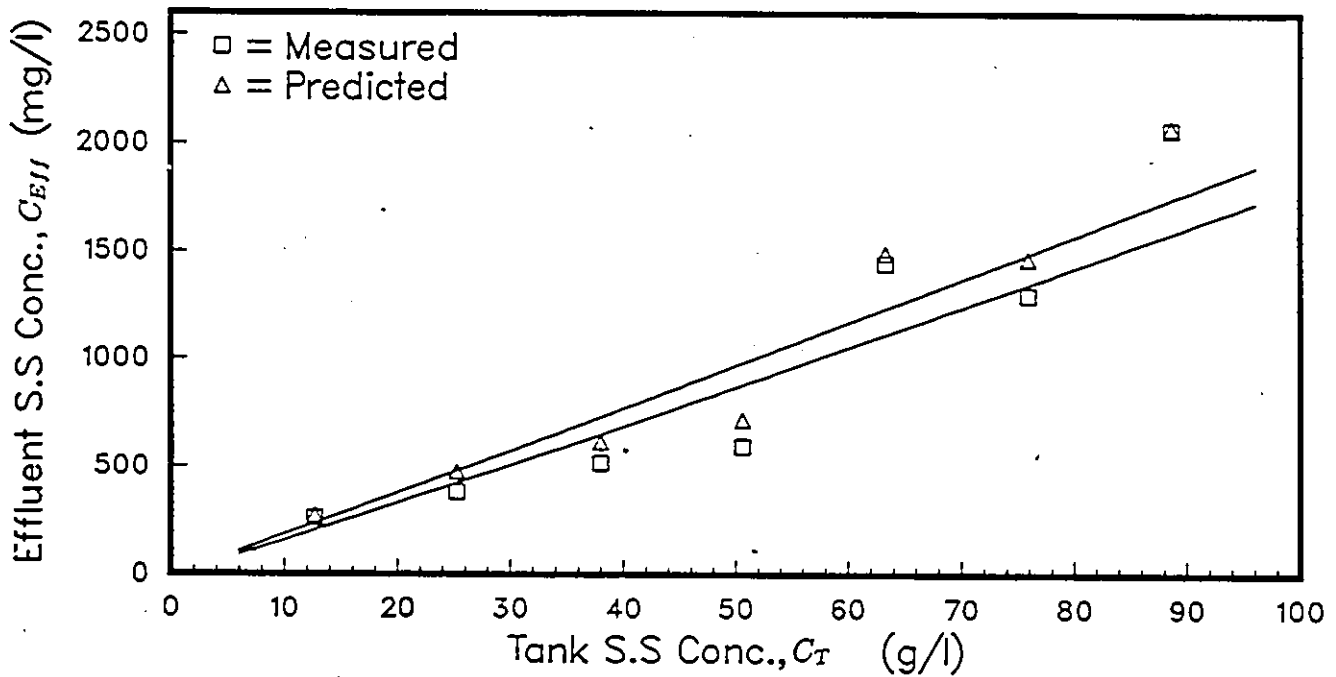


Figure B.22: Plot of the Measured and the Predicted S.S. Conc. ($S=1:2$, $V=45$ l, $D=0.5$ m, $L=0.3$ m)

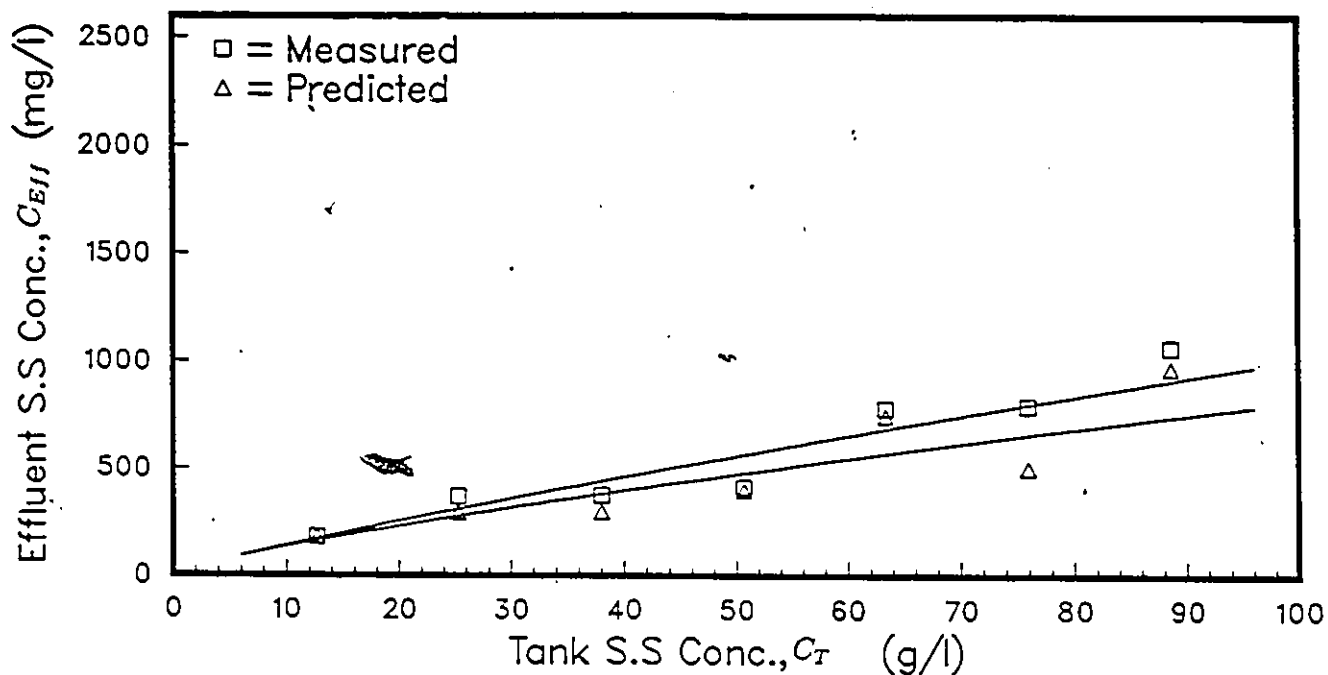


Figure B.23: Plot of the Measured and the Predicted S.S. Conc. (S= 1:2 , V= 30 l, D= 0.5 m, L= 0.1 m)

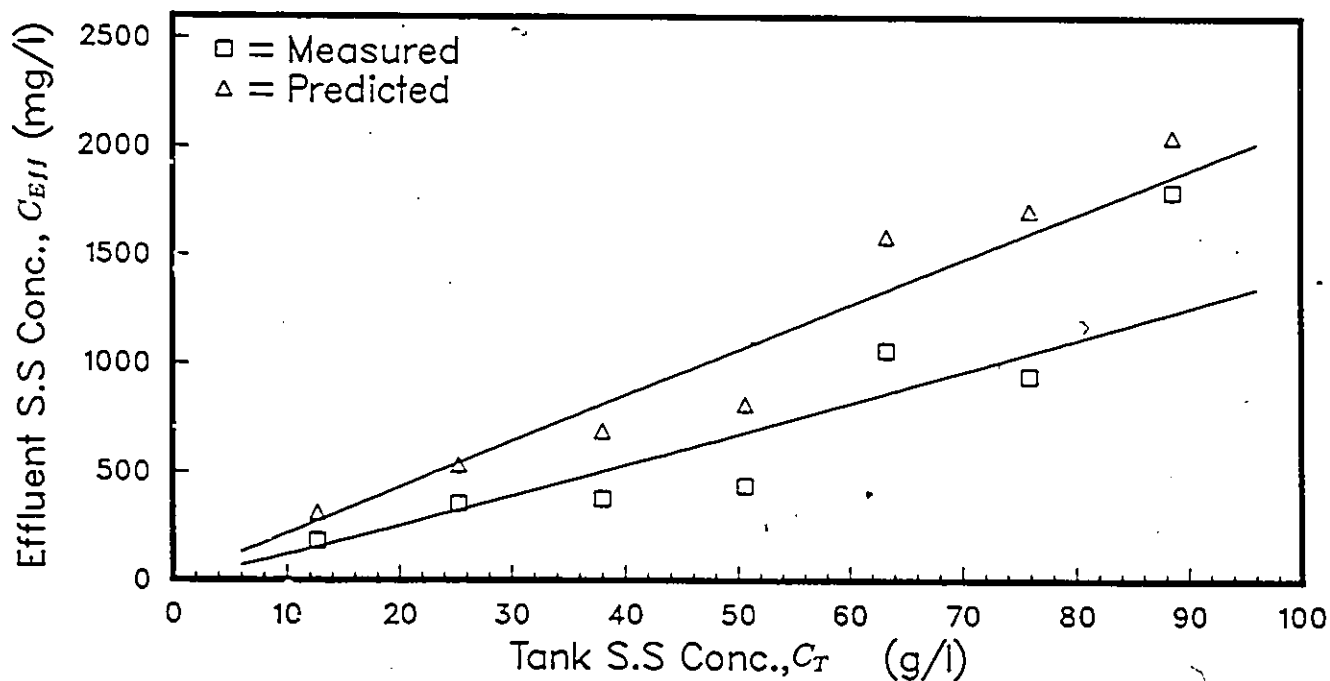


Figure B.24: Plot of the Measured and the Predicted S.S. Conc. (S= 1:2 , V= 30 l, D= 0.5 m, L= 0.5 m)

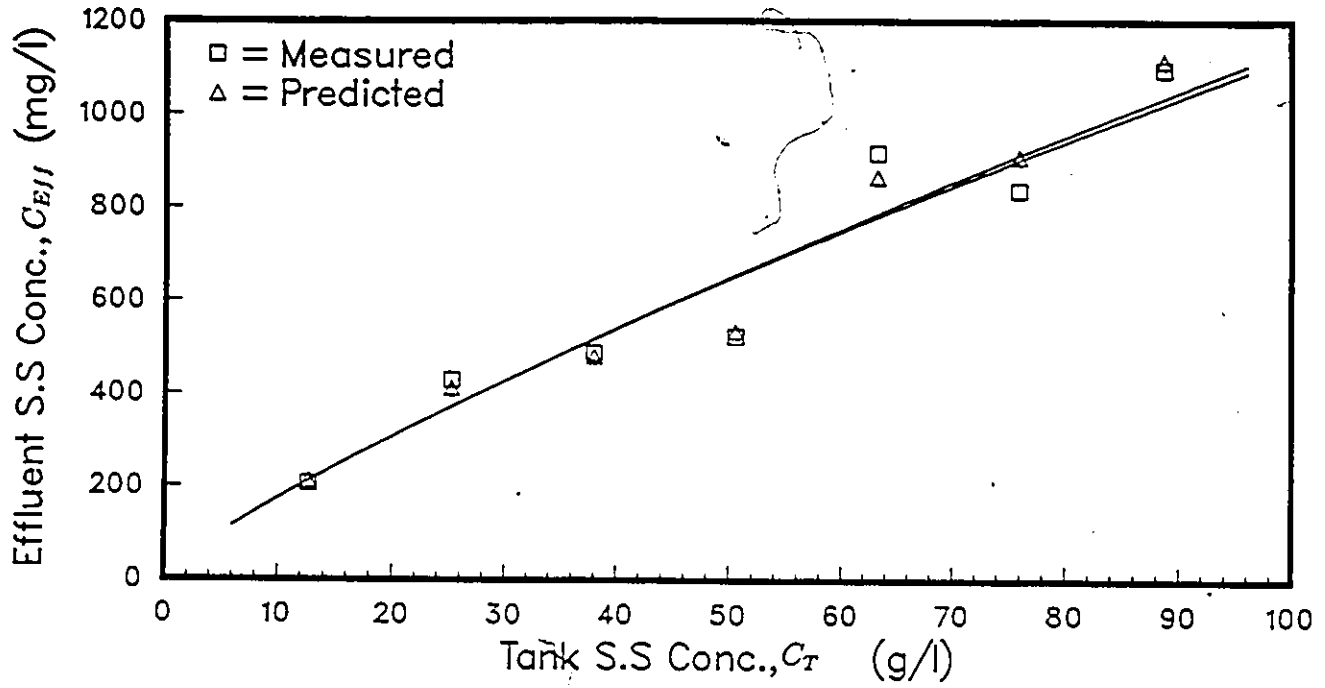


Figure B.25: Plot of the Measured and the Predicted S.S. Conc.
($S = 1:2$, $V = 30$ l, $D = 0.3$ m, $L = 0.3$ m)

APPENDIX C

DATA FOR THE SETTLING COLUMN TESTS

SAMPLE CALCULATION

The settling column used is shown in Fig. (3.2), and described in Section (4.2.1).

The initial SS concentration was calculated based on the average value of SS concentration in the 4 taps.

$$\%SS \text{ Setl.} = \frac{(\text{Initial SS conc.}) - (\text{SS conc. at time= t})}{(\text{Initial SS conc.})} * 100$$

$$(\text{e.g., } 1.1 = \frac{89 - 88}{89} * 100)$$

$$\text{Avg. \%SS Setl.} = (\%SS \text{ Setl. at Tap\#1} + \%SS \text{ Setl. at Tap\#2} + \%SS \text{ Setl. at Tap\#3} + \%SS \text{ Setl. at Tap\#4})/4$$

$$(\text{e.g., } 12.4 = (1.1 + 15.7 + 21.3 + 11.2)/4)$$

In the following tables the SS concentration is expressed in mg/L.

Table C.1 : Settling Column Analysis of the Tank Effluent

(S= flat, Sludge Level= 21%)

(V= 30 l, D= 0.3 m, L= 0.1 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	89	0.0	89	0.0	89	0.0	89	0.0	0.0
5	89	1.1	75	15.7	70	21.3	79	11.2	12.4
15	84	5.6	64	28.1	76	14.6	70	21.3	17.4
25	74	16.9	74	16.9	73	18.0	55	38.2	22.5
40	69	22.5	83	6.7	71	20.2	42	52.8	25.6
60	67	24.7	70	21.3	72	19.1	67	24.7	22.5

Table C.2 : Settling Column Analysis of the Tank Effluent

(S= flat, Sludge Level= 43%)

(V= 30 l, D= 0.3 m, L= 0.1 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	237	0.0	237	0.0	237	0.0	237	0.0	0.0
5	215	9.3	220	7.2	226	4.6	237	0.0	5.3
15	204	13.9	217	8.4	215	9.3	215	9.3	10.2
25	167	29.5	190	19.8	205	13.5	216	8.9	17.9
40	191	19.4	225	5.1	188	20.7	188	20.7	16.5
60	191	19.4	220	7.2	232	2.1	214	9.7	9.6

Table C.3 : Settling Column Analysis of the Tank Effluent

(S= flat, Sludge Level= 64%)

(V= 30 l, D= 0.3 m, L= 0.1 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	527	0.0	527	0.0	527	0.0	527	0.0	0.0
5	409	22.4	419	20.5	480	8.9	468	11.2	15.7
15	381	27.7	394	25.2	411	22.0	420	20.3	23.8
25	364	30.9	363	31.1	392	25.6	390	26.0	28.4
40	376	28.7	373	29.2	388	26.4	374	29.0	28.3
60	343	34.9	359	31.9	366	30.6	345	34.5	33.0

Table C.4 : Settling Column Analysis of the Tank Effluent

134

(S= 1:2 , Sludge Level= 21%)

(V= 30 l, D= 0.3 m, L= 0.3 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	332	0.0	332	0.0	332	0.0	332	0.0	0.0
5	249	25.0	260	21.7	273	17.8	285	14.2	19.7
15	235	29.2	219	34.0	246	25.9	260	21.7	27.7
25	245	26.2	253	23.8	257	22.6	253	23.8	24.1
40	250	24.7	240	27.7	252	24.1	248	25.3	25.5
60	235	29.2	243	26.8	243	26.8	231	30.4	28.3

Table C.5 : Settling Column Analysis of the Tank Effluent

(S= 1:2 , Sludge Level= 43%)

(V= 30 l, D= 0.3 m, L= 0.3 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	455	0.0	455	0.0	455	0.0	455	0.0	0.0
5	353	22.4	414	9.0	455	0.0	438	3.7	8.8
15	369	18.9	409	10.1	400	12.1	402	11.6	13.2
25	319	29.9	339	25.5	390	14.3	378	16.9	21.6
40	247	45.7	279	38.7	315	30.8	299	34.3	37.4
60	232	49.0	232	49.0	275	39.6	271	40.4	44.5

Table C.6 : Settling Column Analysis of the Tank Effluent

(S= 1:2 , Sludge Level= 64%)

(V= 30 l, D= 0.3 m, L= 0.3 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	753	0.0	753	0.0	753	0.0	753	0.0	0.0
5	701	6.9	741	1.6	731	2.9	677	10.1	5.4
15	589	21.8	647	14.1	669	11.2	611	18.9	16.5
25	412	45.3	538	28.6	620	17.7	595	21.0	28.1
40	269	64.3	350	53.5	552	26.7	560	25.6	42.5
60	163	78.4	198	73.7	290	61.5	364	51.7	66.3

Table C.7 : Settling Column Analysis of the Tank Effluent

(S= 1:1 , Sludge Level= 21%)

(V= 30 l, D= 0.3 m, L= 0.1 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	341	0.0	341	0.0	341	0.0	341	0.0	0.0
5	263	22.9	293	14.1	300	12.0	338	0.9	12.5
15	261	23.5	261	23.5	271	20.5	282	17.3	21.2
25	249	27.0	261	23.5	264	22.6	267	21.7	23.7
40	250	26.7	260	23.8	257	24.6	250	26.7	25.4
60	242	29.0	255	25.2	254	25.5	248	27.3	26.8

Table C.8 : Settling Column Analysis of the Tank Effluent

(S= 1:1 , Sludge Level= 43%)

(V= 30 l, D= 0.3 m, L= 0.1 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	408	0.0	408	0.0	408	0.0	408	0.0	0.0
5	256	37.3	334	18.1	384	5.9	408	0.0	15.3
15	239	41.4	266	34.8	272	33.3	290	28.9	34.6
25	225	44.9	254	37.7	232	43.1	267	34.6	40.1
40	193	52.7	216	47.1	227	44.4	241	40.9	46.3
60	181	55.6	228	44.1	239	41.4	235	42.4	45.9

Table C.9 : Settling Column Analysis of the Tank Effluent

(S= 1:1 , Sludge Level= 64%)

(V= 30 l, D= 0.3 m, L= 0.1 m)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	839	0.0	839	0.0	839	0.0	839	0.0	0.0
5	718	14.4	713	15.0	680	19.0	687	18.1	16.6
15	563	32.9	590	29.7	634	24.4	630	24.9	28.0
25	496	40.9	520	38.0	541	35.5	557	33.6	37.0
40	403	52.0	472	43.7	522	37.8	487	42.0	43.9
60	382	54.5	428	49.0	451	46.2	434	48.3	49.5

Table C.10: Settling Column Analysis of the Tank OM

(S= flat, Sludge Level= 43%)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	1491	0.0	1491	0.0	1491	0.0	1491	0.0	0.0
5	603	59.6	533	64.3	492	67.0	783	47.5	59.6
15	216	85.5	289	80.6	349	76.6	316	78.8	80.4
25	148	90.1	190	87.3	208	86.0	243	83.7	86.8
40	98	93.4	143	90.4	191	87.2	174	88.3	89.8
60	79	94.7	93	93.8	117	92.2	115	92.3	93.2

Table C.11: Settling Column Analysis of the Tank OM

(S= flat, Sludge Level= 64%)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	633	0.0	633	0.0	633	0.0	633	0.0	0.0
5	236	62.7	301	52.4	338	46.6	388	38.7	50.1
15	116	81.7	133	79.0	188	70.3	189	70.1	75.3
25	93	85.3	109	82.8	113	82.1	148	76.6	81.7
40	71	88.8	82	87.0	96	84.8	107	83.1	85.9
60	64	89.9	71	88.8	86	86.4	91	85.6	87.7

Table C.12: Settling Column Analysis of the Tank OM

(S= 1:1 , Sludge Level= 21%)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	625	0.0	625	0.0	625	0.0	625	0.0	0.0
5	75	88.0	167	73.3	196	68.6	269	57.0	71.7
15	56	91.0	76	87.8	69	89.0	81	87.0	88.7
25	9	98.6	27	95.7	40	93.6	35	94.4	95.6
40	18	97.1	37	94.1	39	93.8	34	94.6	94.9
60	21	96.6	23	96.3	49	92.2	24	96.2	95.3

Table C.13: Settling Column Analysis of the Tank OM

(S= 1:1 , Sludge Level= 43%)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	1171	0.0	1171	0.0	1171	0.0	1171	0.0	0.0
5	402	65.7	631	46.1	792	32.4	648	44.7	47.2
15	164	86.0	200	82.9	317	72.9	315	73.1	78.7
25	117	90.0	126	89.2	177	84.9	178	84.8	87.2
40	72	93.9	80	93.2	117	90.0	135	88.5	91.4
60	63	94.6	64	94.5	112	90.4	113	90.4	92.5

Table C.14: Settling Column Analysis of the Tank OM

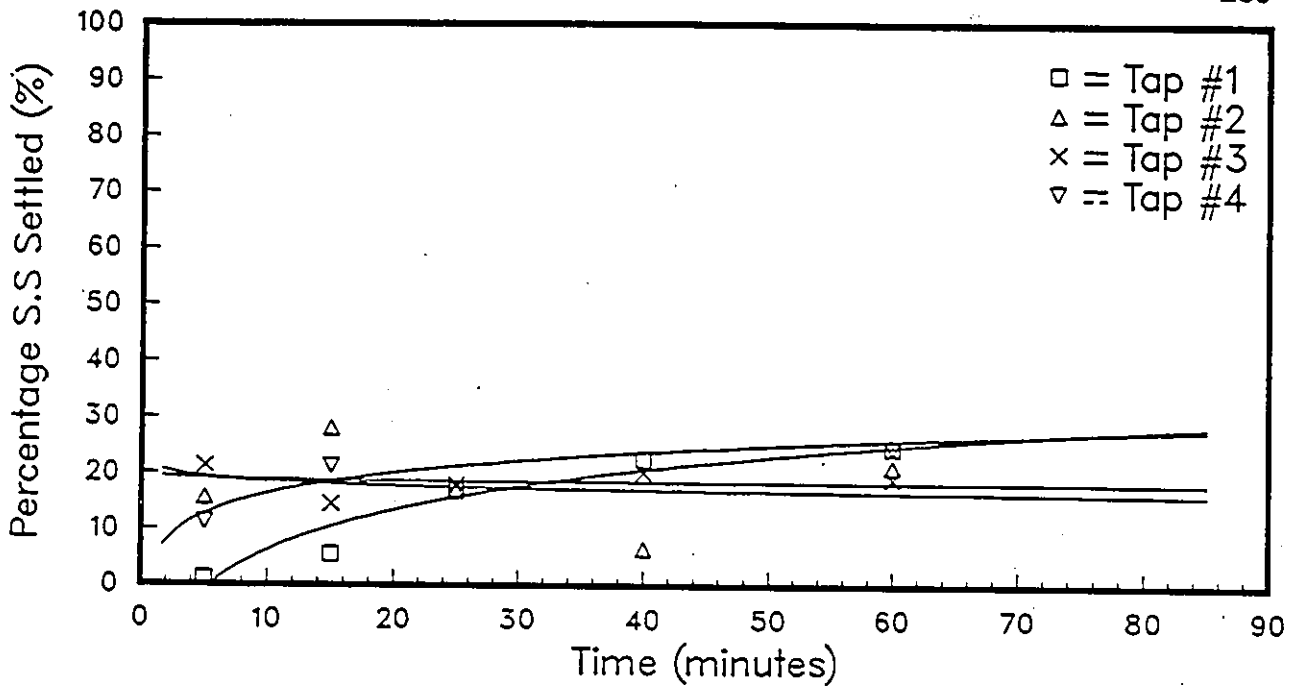
(S= 1:1 , Sludge Level= 64%)

Time (min)	Tap#1		Tap#2		Tap#3		Tap#4		Avg.
	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	SS Conc.	%SS Setl.	%SS Setl.
0	645	0.0	645	0.0	645	0.0	645	0.0	0.0
5	189	70.7	249	61.4	430	33.3	448	30.5	49.0
15	80	87.6	126	80.5	157	75.7	160	75.2	79.7
25	56	91.3	61	90.5	71	89.0	92	85.7	89.1
40	37	94.3	64	90.1	67	89.6	71	89.0	90.7
60	31	95.2	53	91.8	49	92.4	54	91.6	92.8

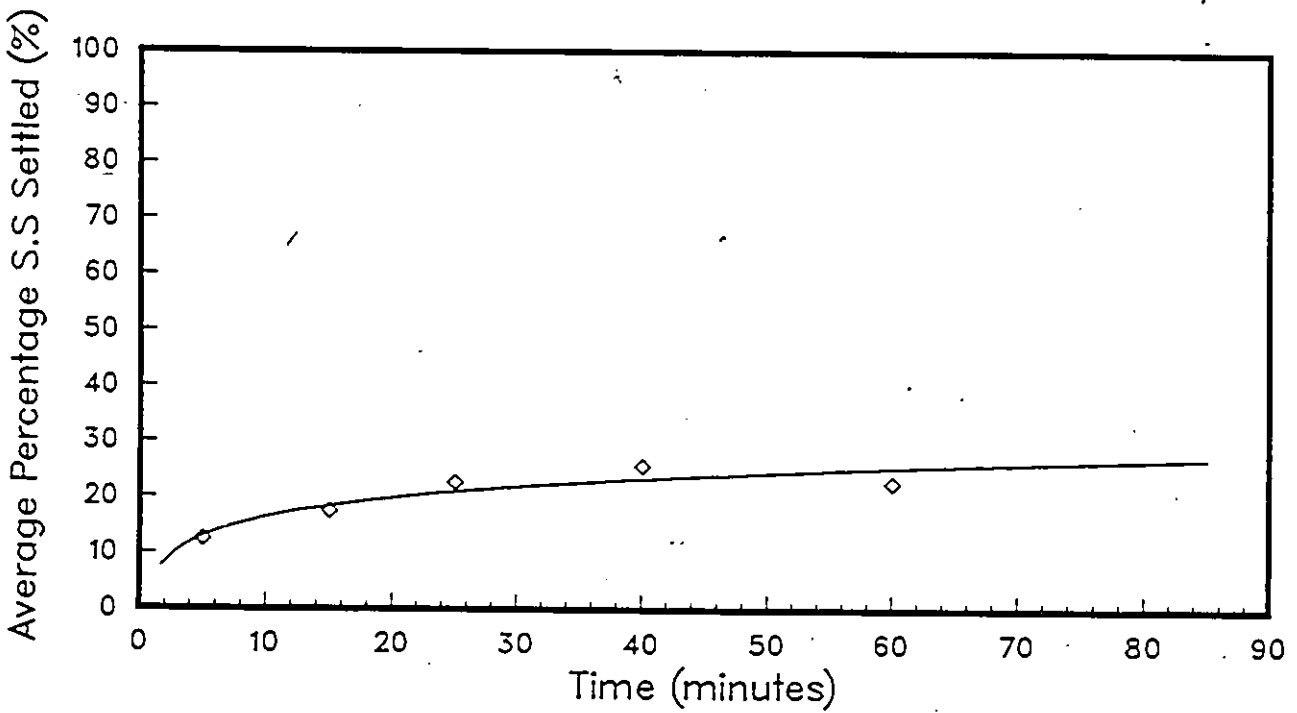
APPENDIX D

FIGURES OF THE SETTLING COLUMN TESTS



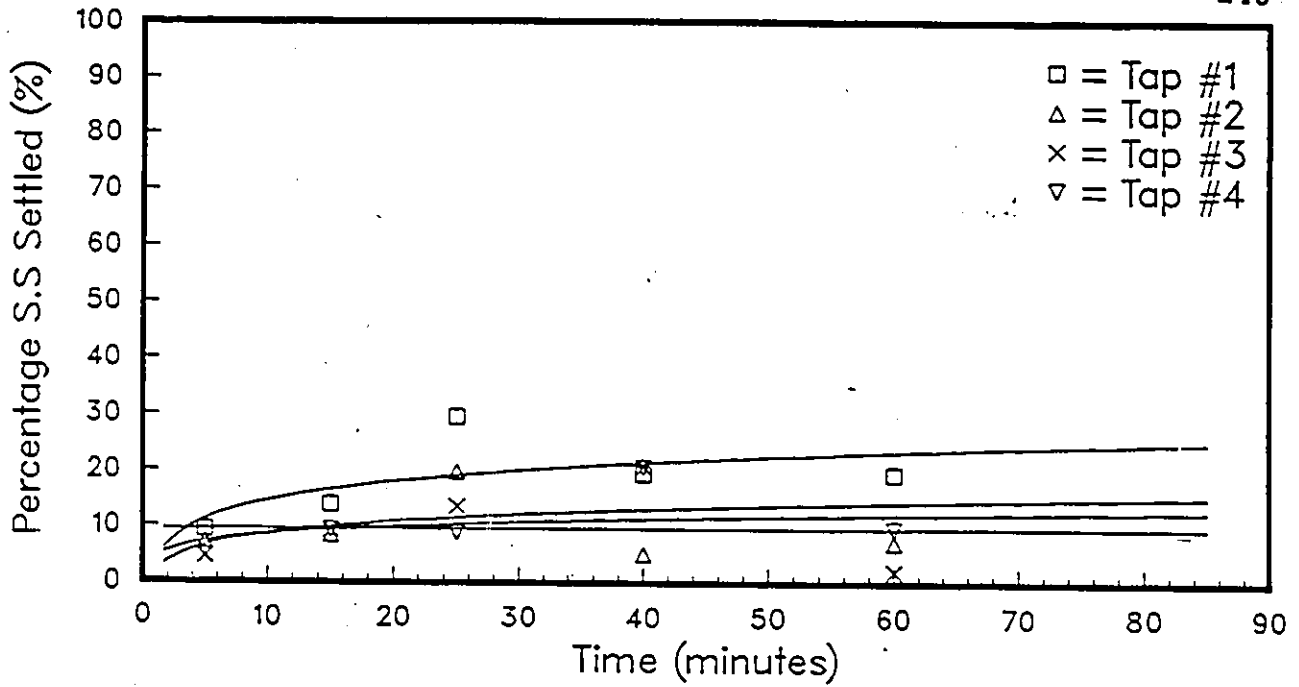


(a) Percentage S.S Settled vs Time.

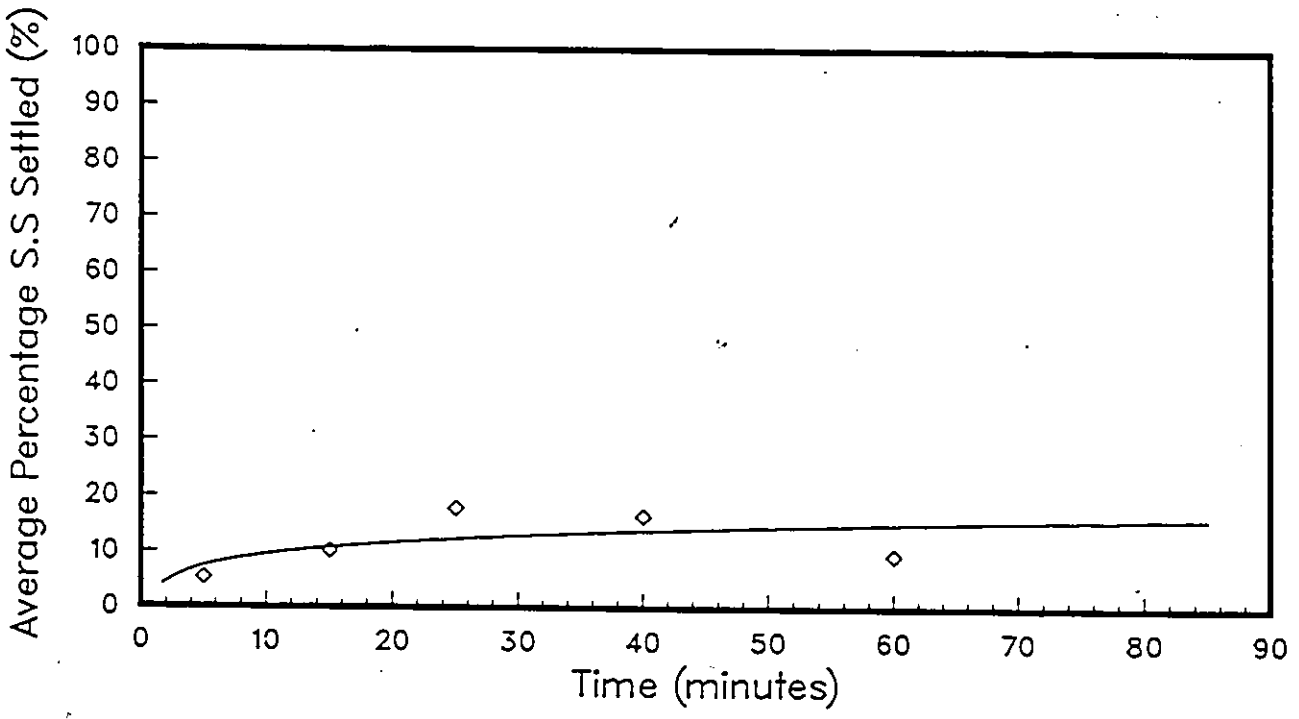


(b) Average Percentage S.S Settled vs Time.

Figure D.1 : Settling Column Test of the Tank Effluent.
 S= flat, Sludge Level= 21%
 (V= 30 l, D= 0.3 m, L= 0.1 m)

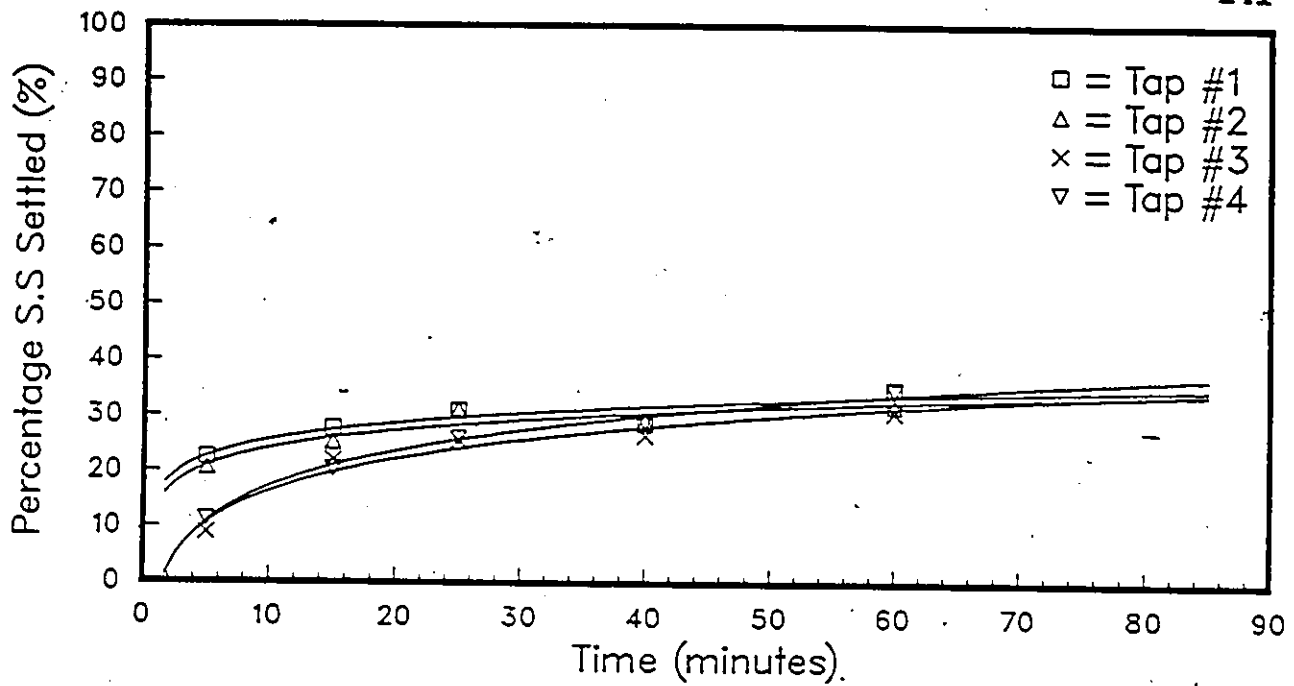


(a) Percentage S.S Settled vs Time.

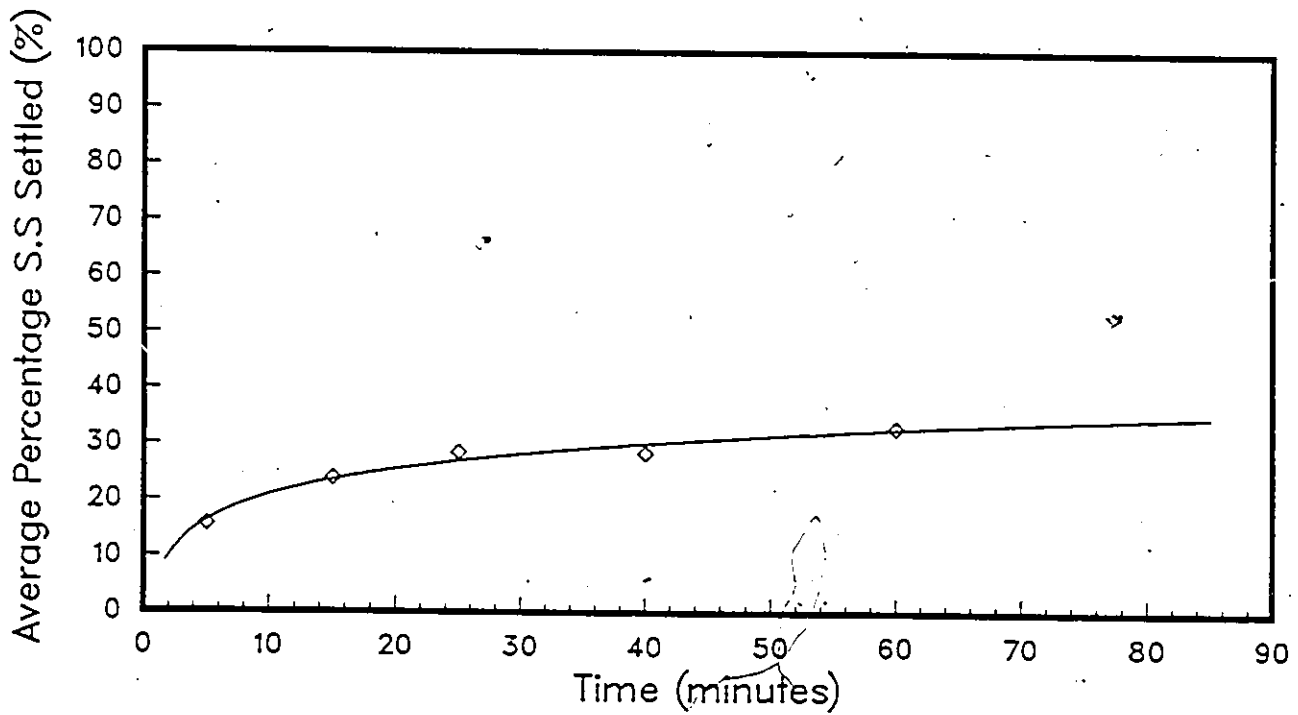


(b) Average Percentage S.S Settled vs Time.

Figure D.2 : Settling Column Test of the Tank Effluent.
 S= flat, Sludge Level= 43%
 (V= 30 l, D= 0.3 m, L= 0.1 m)



(a) Percentage S.S Settled vs Time.

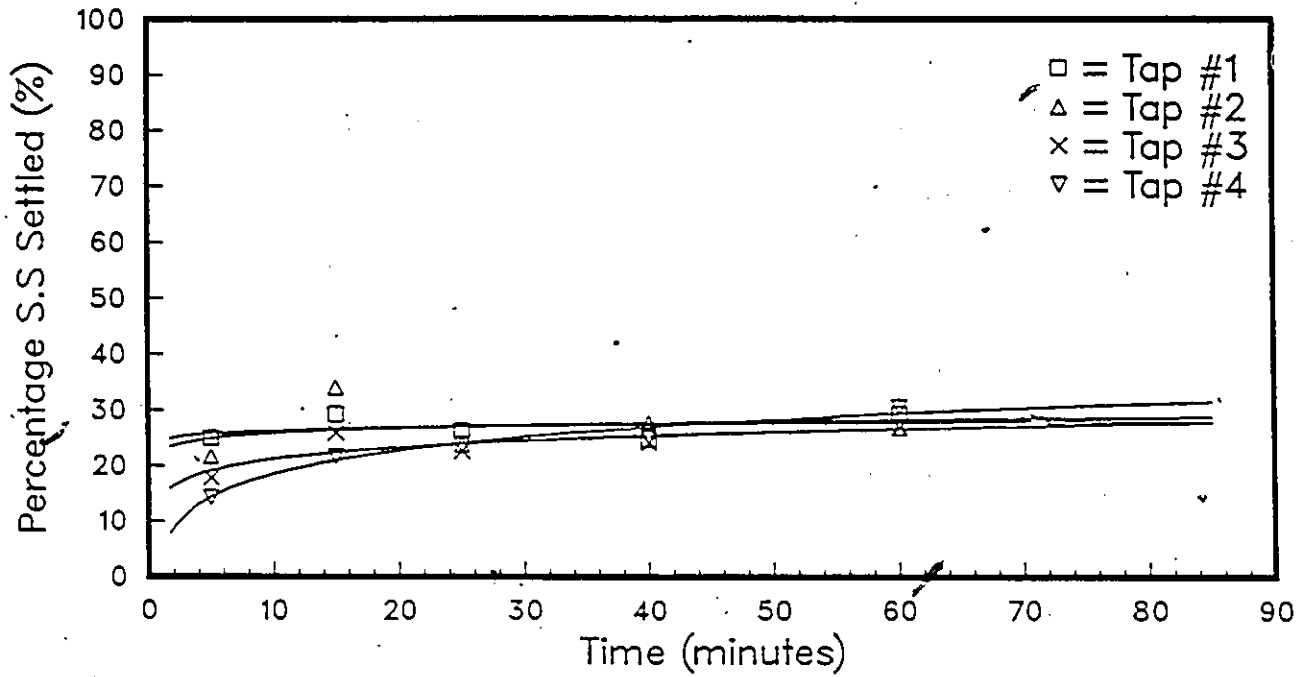


(b) Average Percentage S.S Settled vs Time.

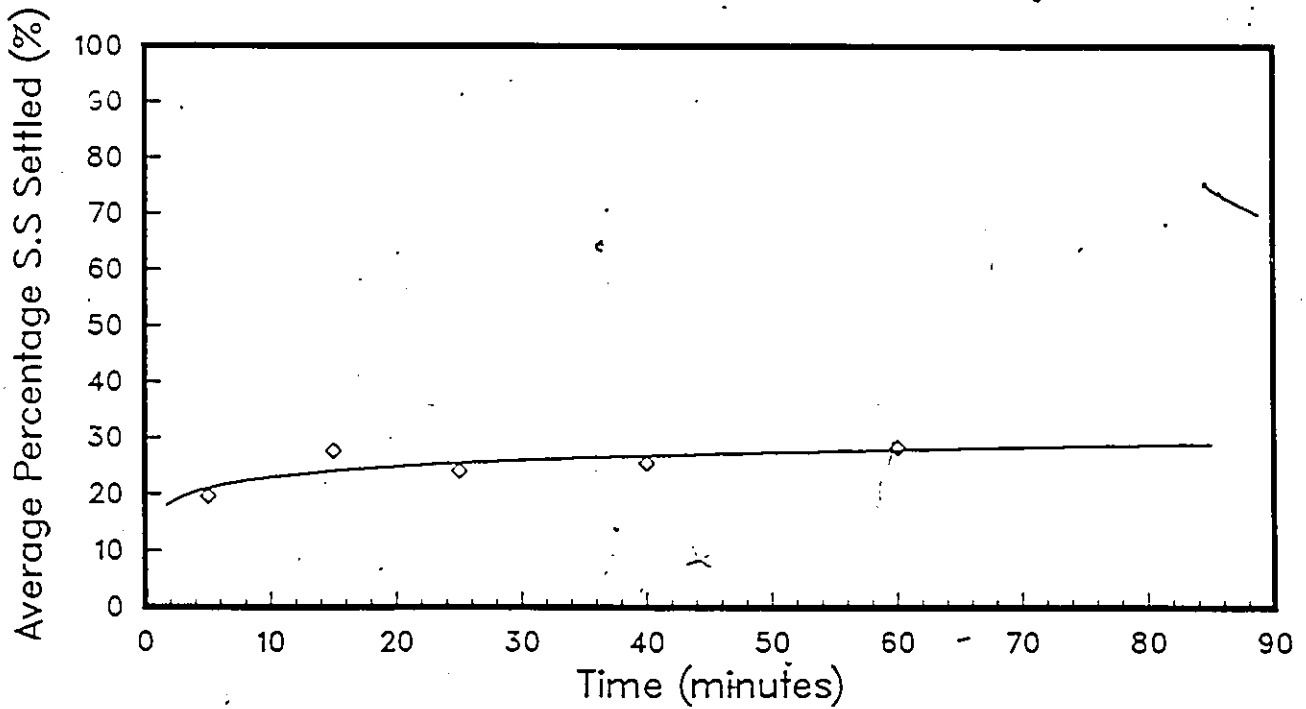
Figure D.3 : Settling Column Test of the Tank Effluent.

S= flat, Sludge Level= 64%

(V= 30 l, D= 0.3 m, L= 0.1 m)

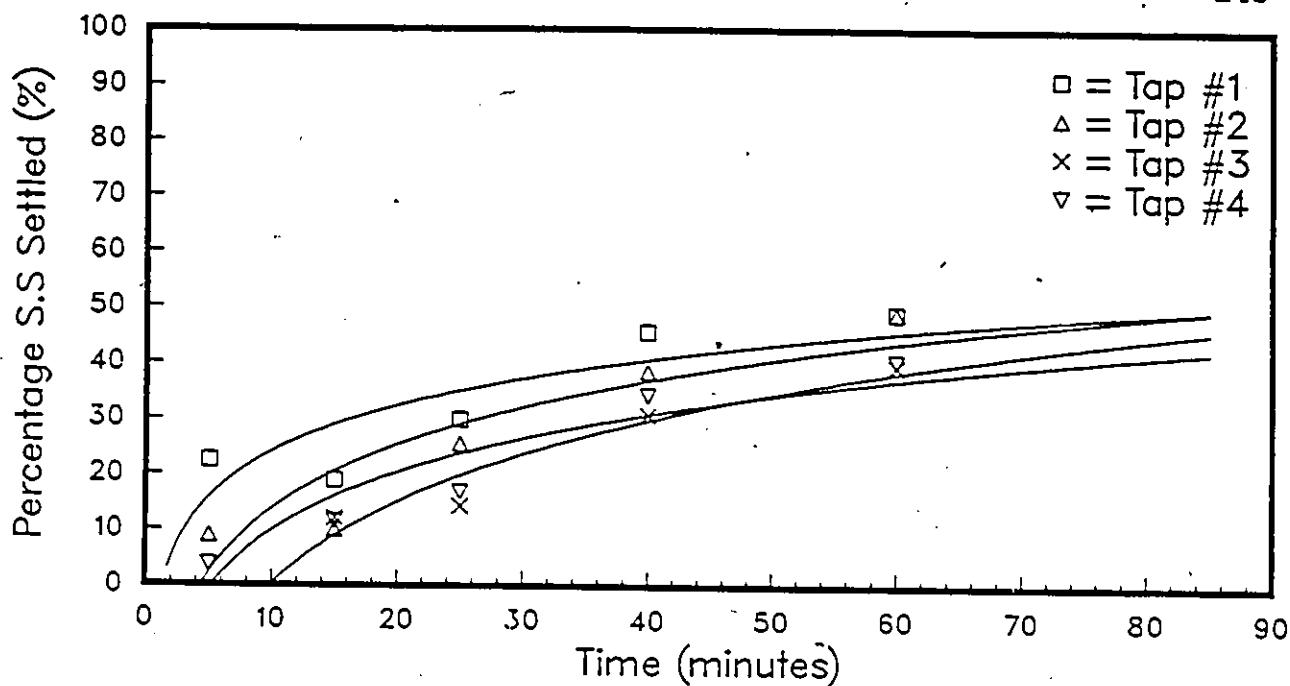


(a) Percentage S.S Settled vs Time.

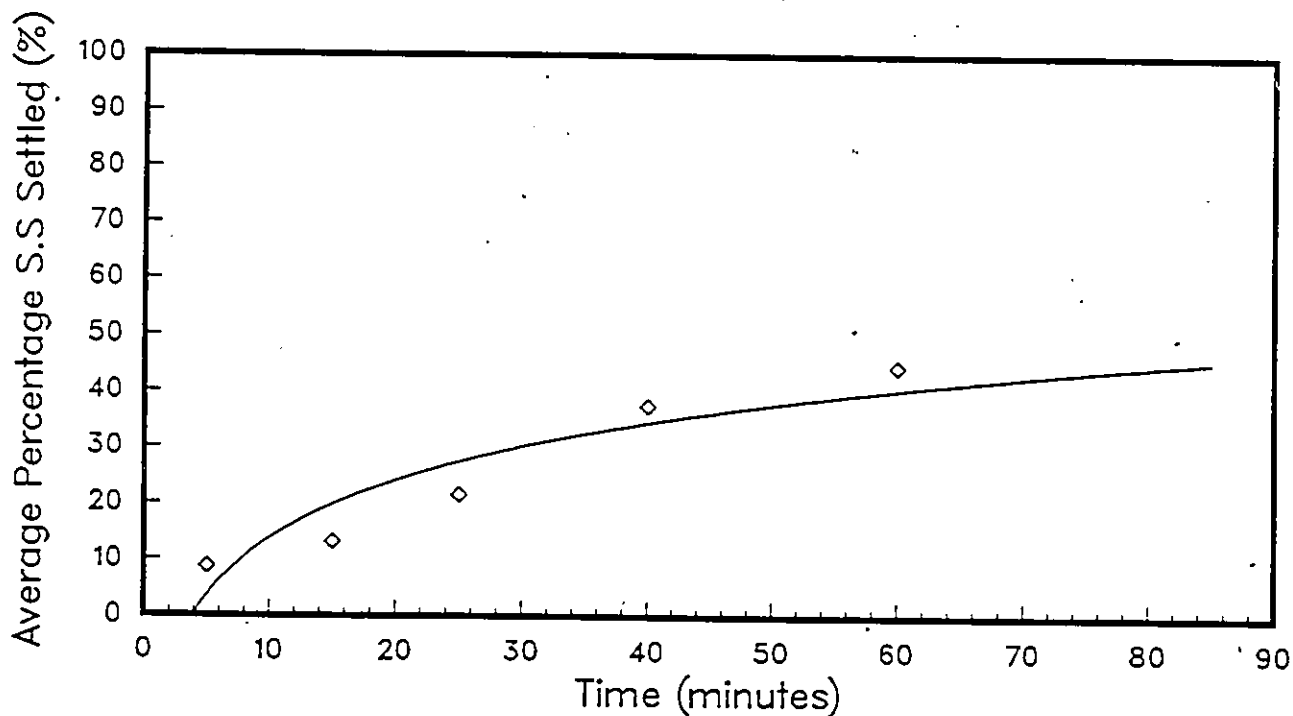


(b) Average Percentage S.S Settled vs Time.

Figure D.4 : Settling Column Test of the Tank Effluent.
 S= 1:2 , Sludge Level= 21%
 (V= 30 l, D= 0.3 m, L= 0.3 m)

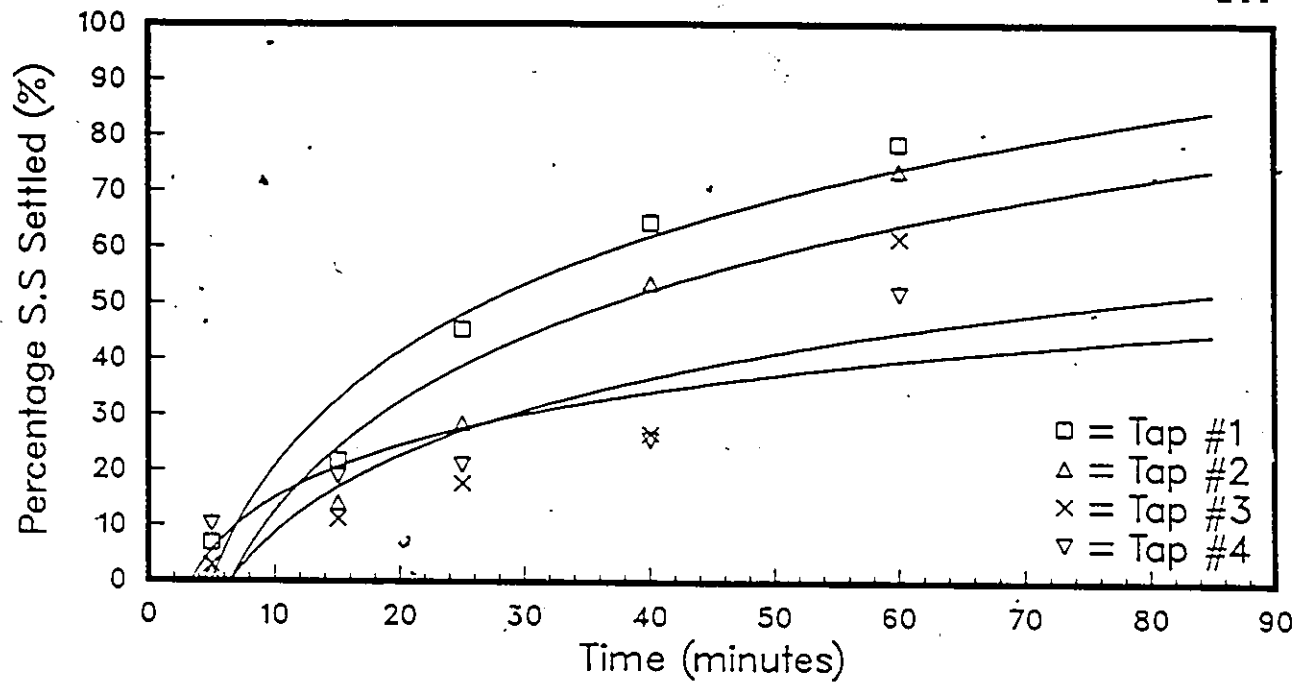


(a) Percentage S.S Settled vs Time.

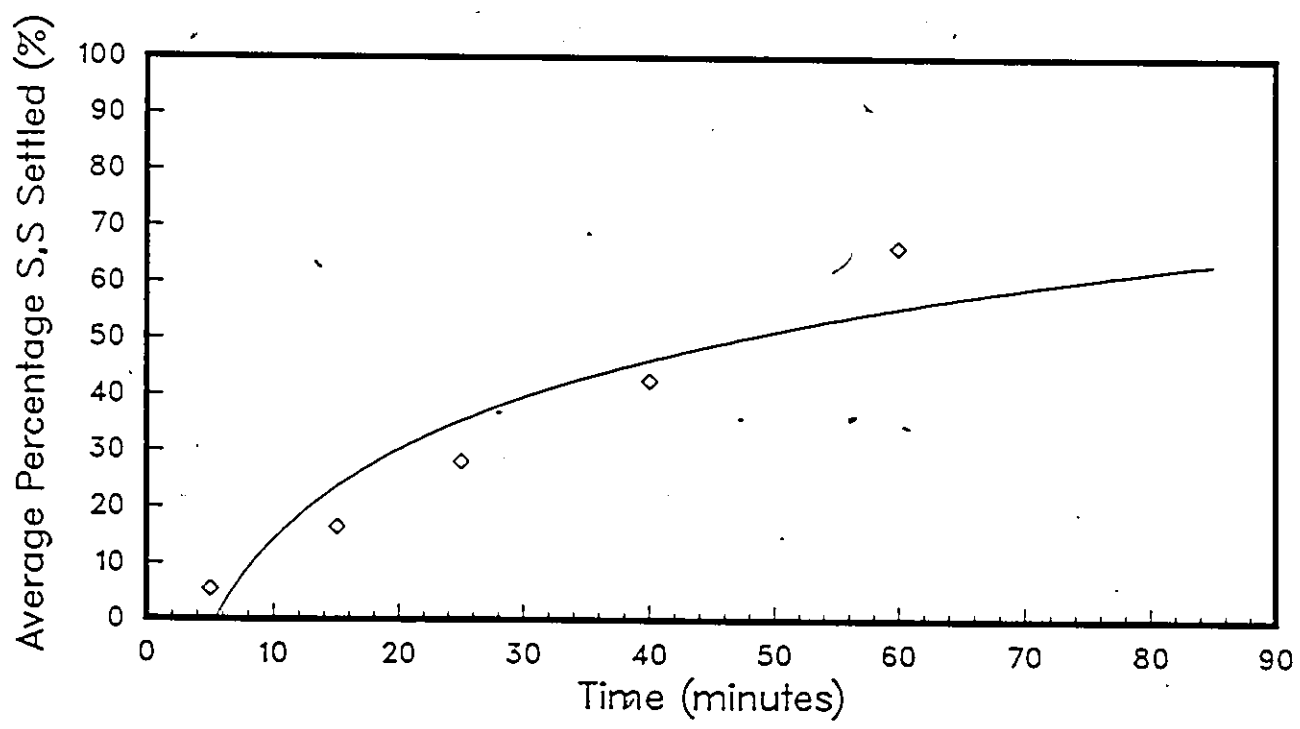


(b) Average Percentage S.S Settled vs Time.

Figure D.5 : Settling Column Test of the Tank Effluent.
 $S = 1:2$, Sludge Level = 43%
 $(V = 30 \text{ l}, D = 0.3 \text{ m}, L = 0.3 \text{ m})$

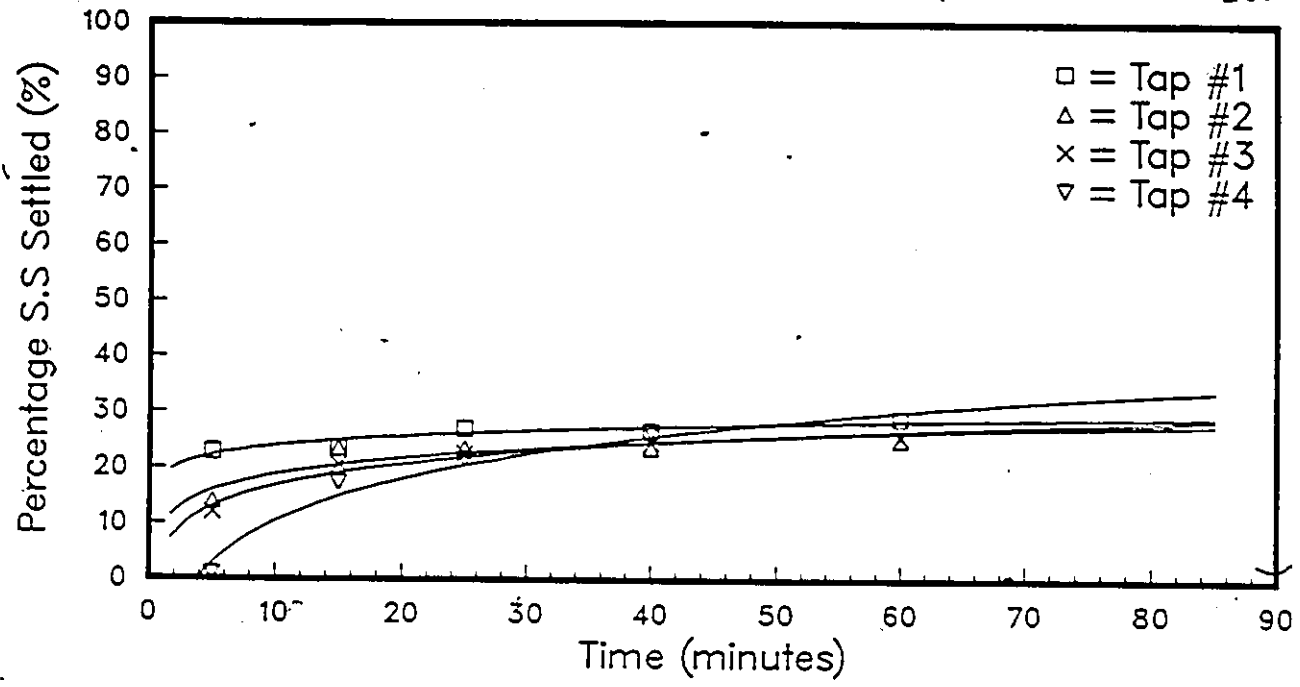


(a) Percentage S.S Settled vs Time.

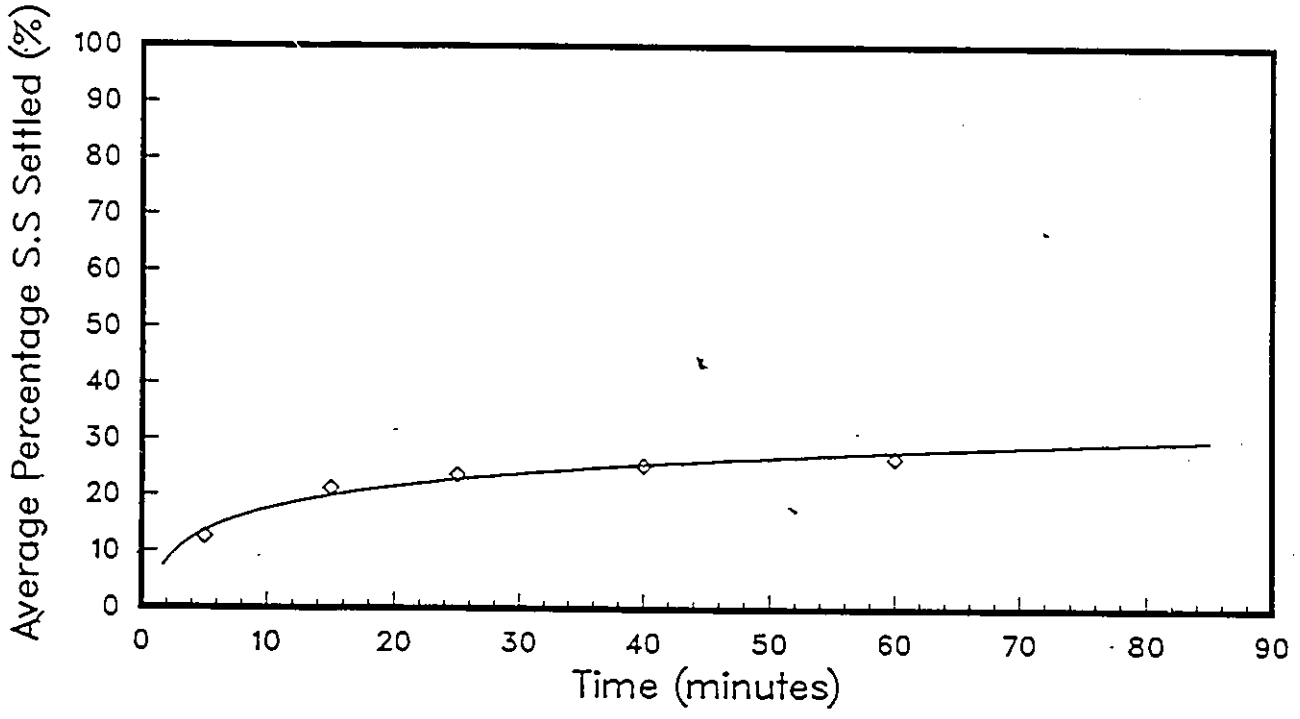


(b) Average Percentage S.S Settled vs Time.

Figure D.6 : Settling Column Test of the Tank Effluent.
S= 1:2 , Sludge Level= 64%
(V= 30 l, D= 0.3 m, L= 0.3 m)

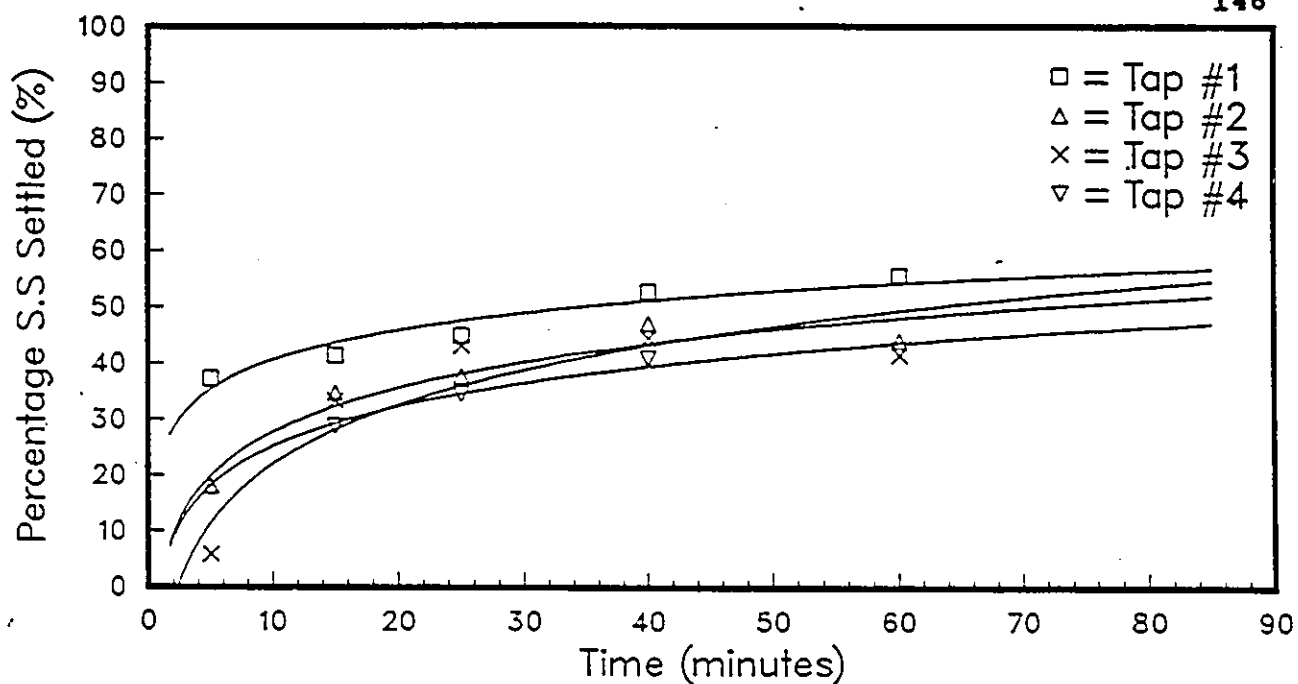


(a) Percentage S.S Settled vs Time.

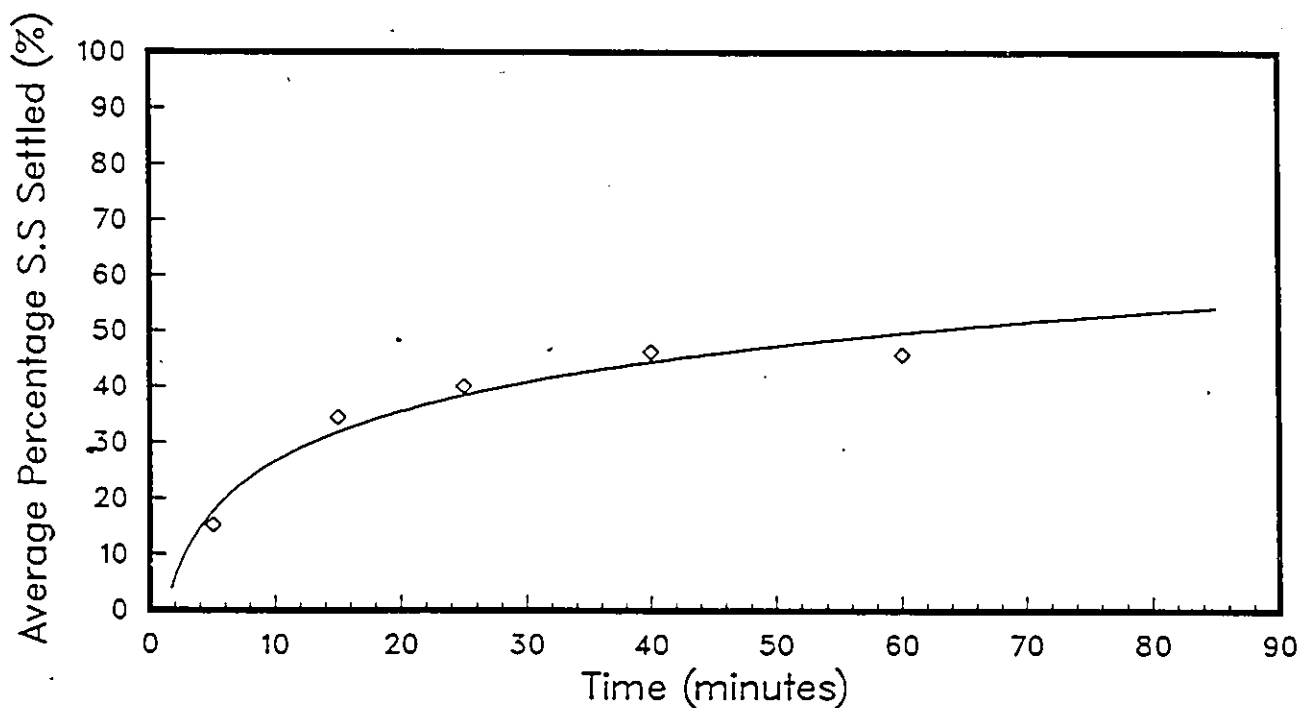


(b) Average Percentage S.S Settled vs Time.

Figure D.7 : Settling Column Test of the Tank Effluent.
S= 1:1 , Sludge Level= 21%
(V= 30 l, D= 0.3 m, L= 0.1 m)

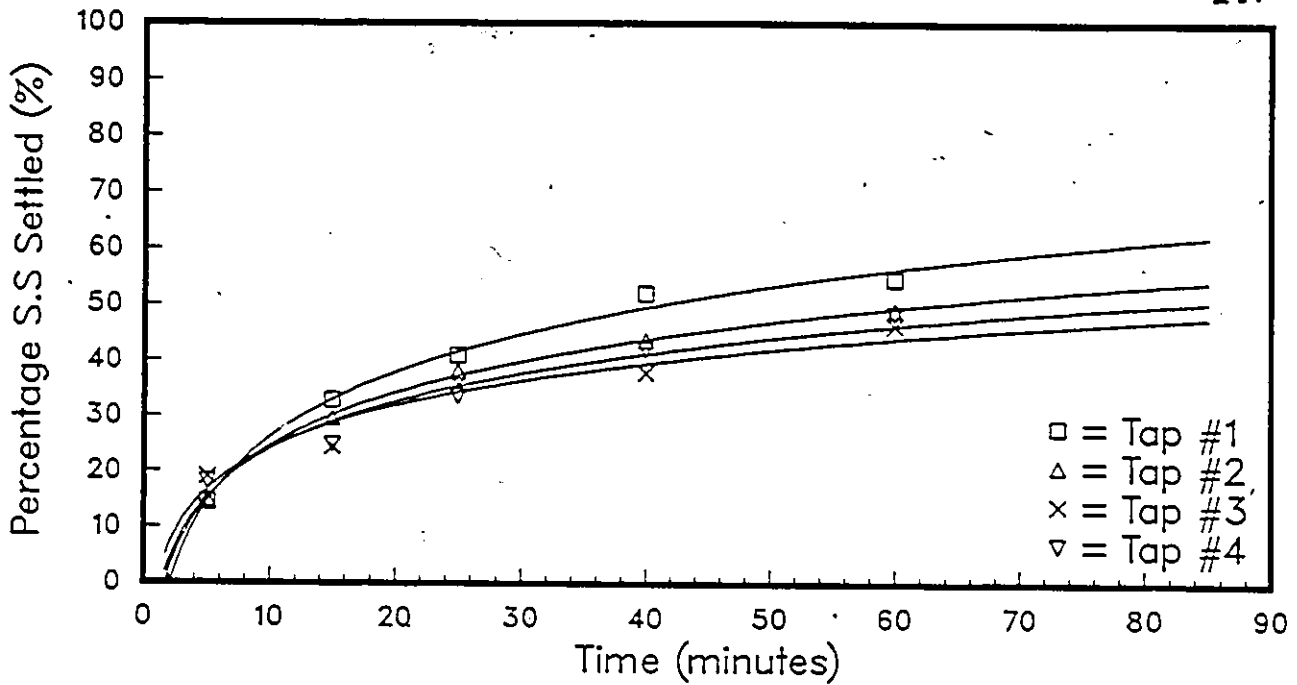


(a) Percentage S.S Settled vs Time.

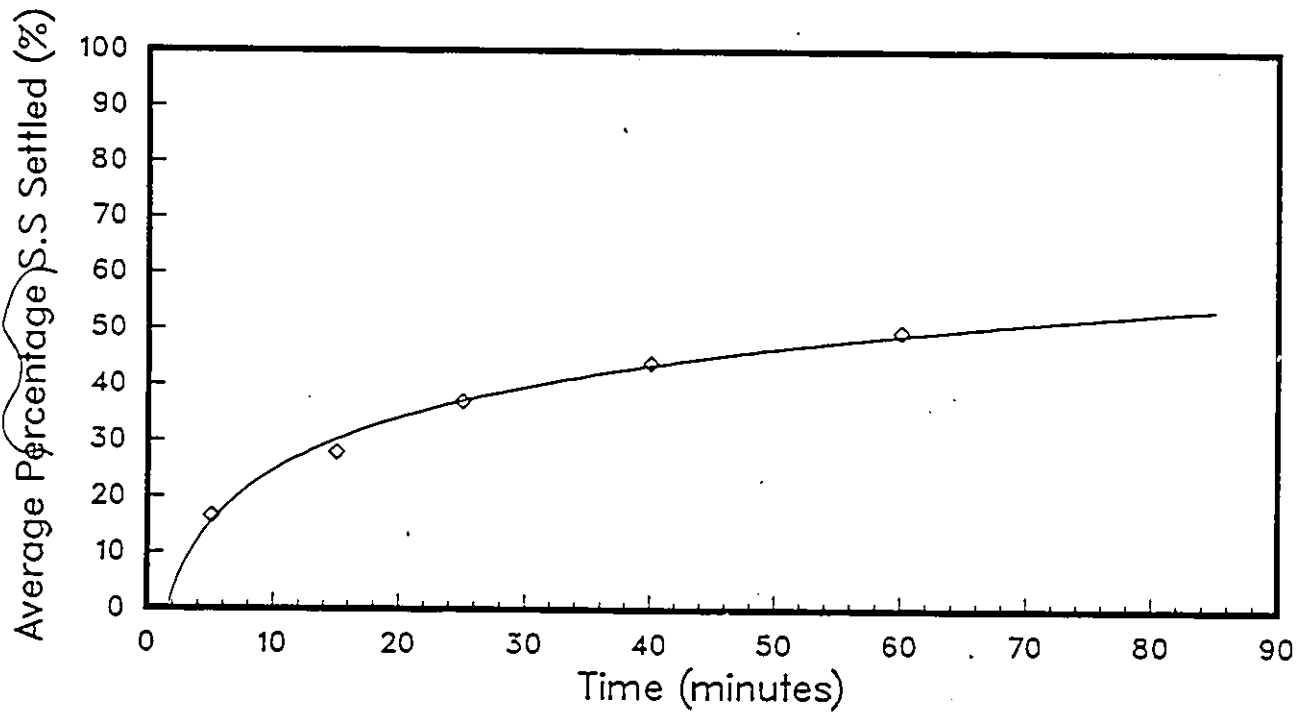


(b) Average Percentage S.S Settled vs Time.

Figure D.8 : Settling Column Test of the Tank Effluent.
 $S = 1:1$, Sludge Level = 43%
 $(V = 30 \text{ l}, D = 0.3 \text{ m}, L = 0.1 \text{ m})$

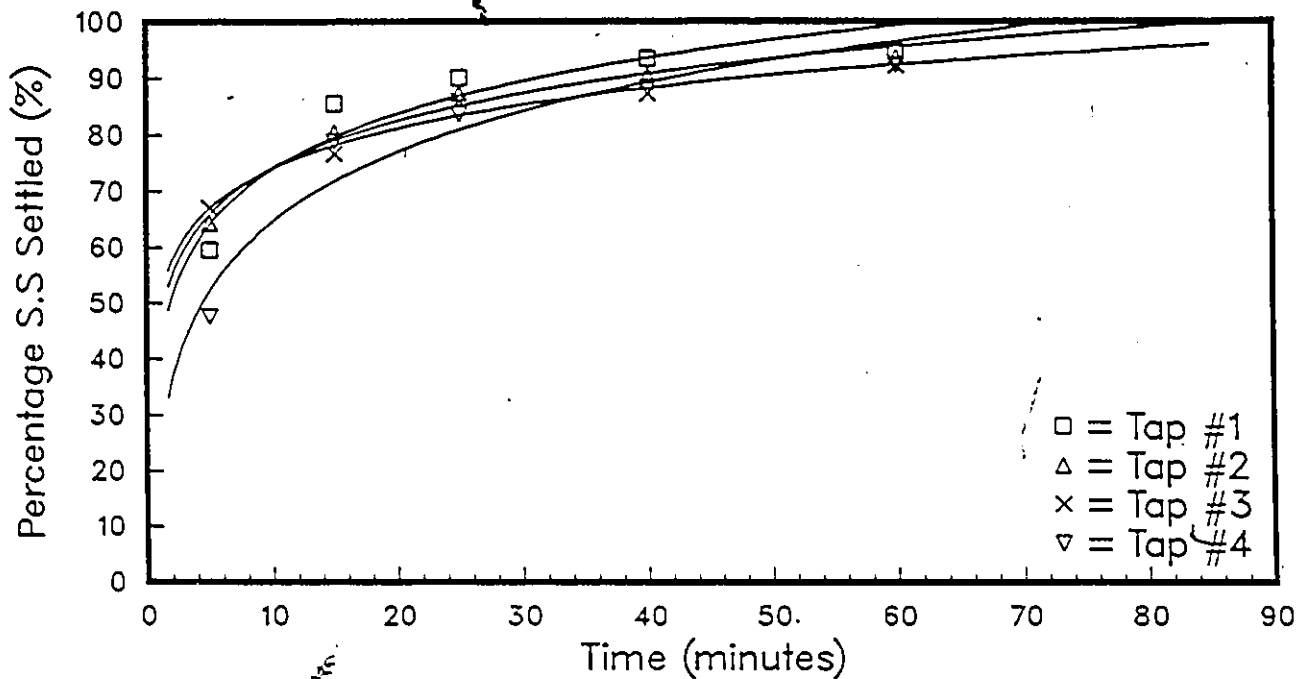


(a) Percentage S.S Settled vs Time.

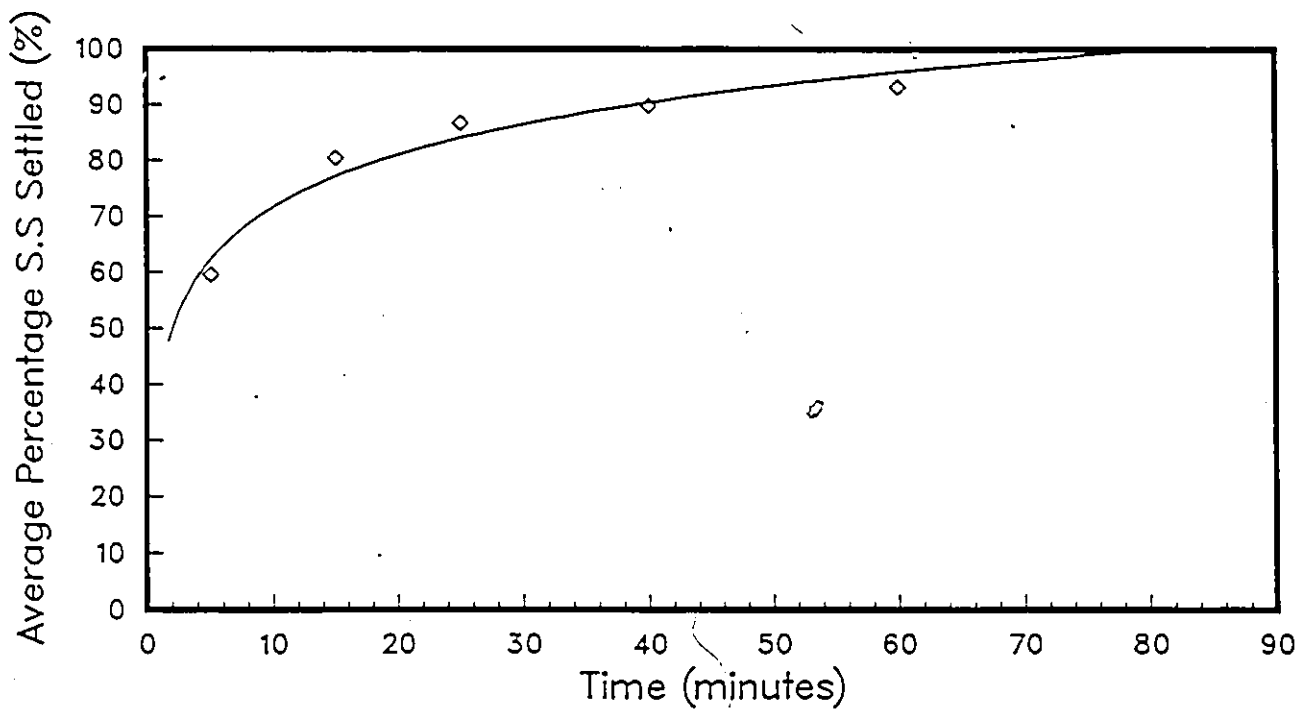


(b) Average Percentage S.S Settled vs Time.

Figure D.9 : Settling Column Test of the Tank Effluent.
 S= 1:1 , Sludge Level= 64%
 (V= 30 l, D= 0.3 m, L= 0.1 m)

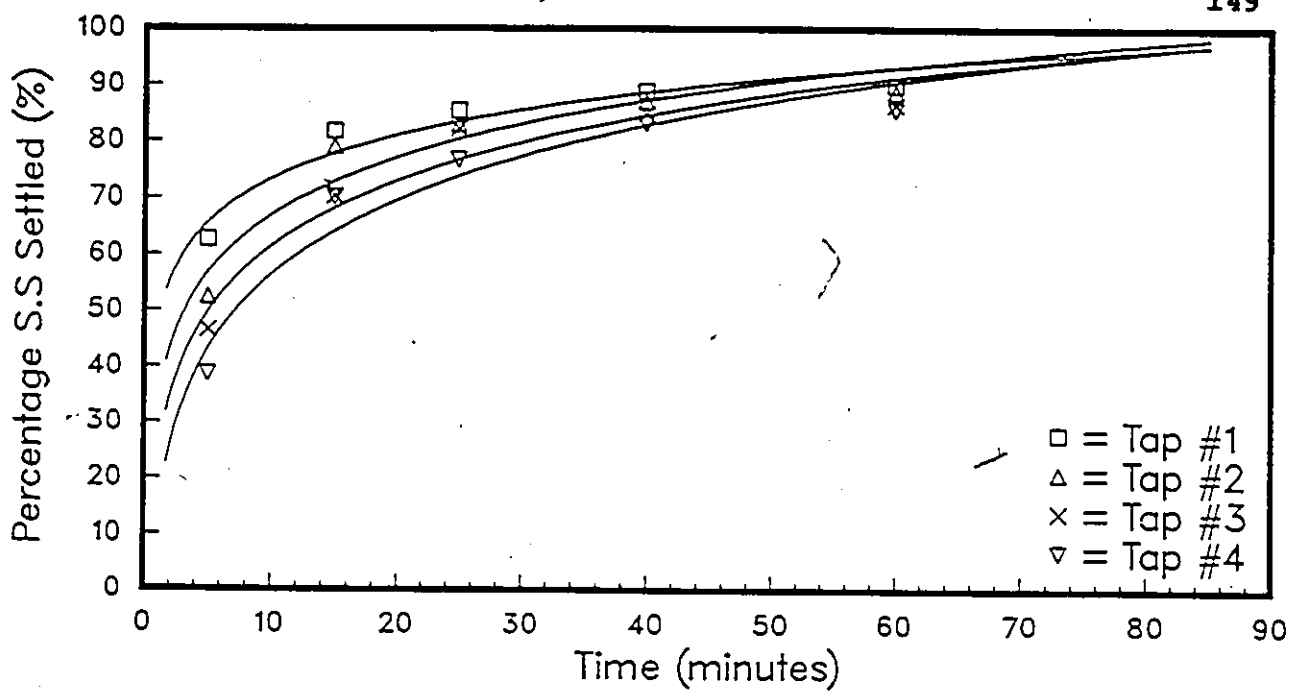


(a) Percentage S.S Settled vs Time.

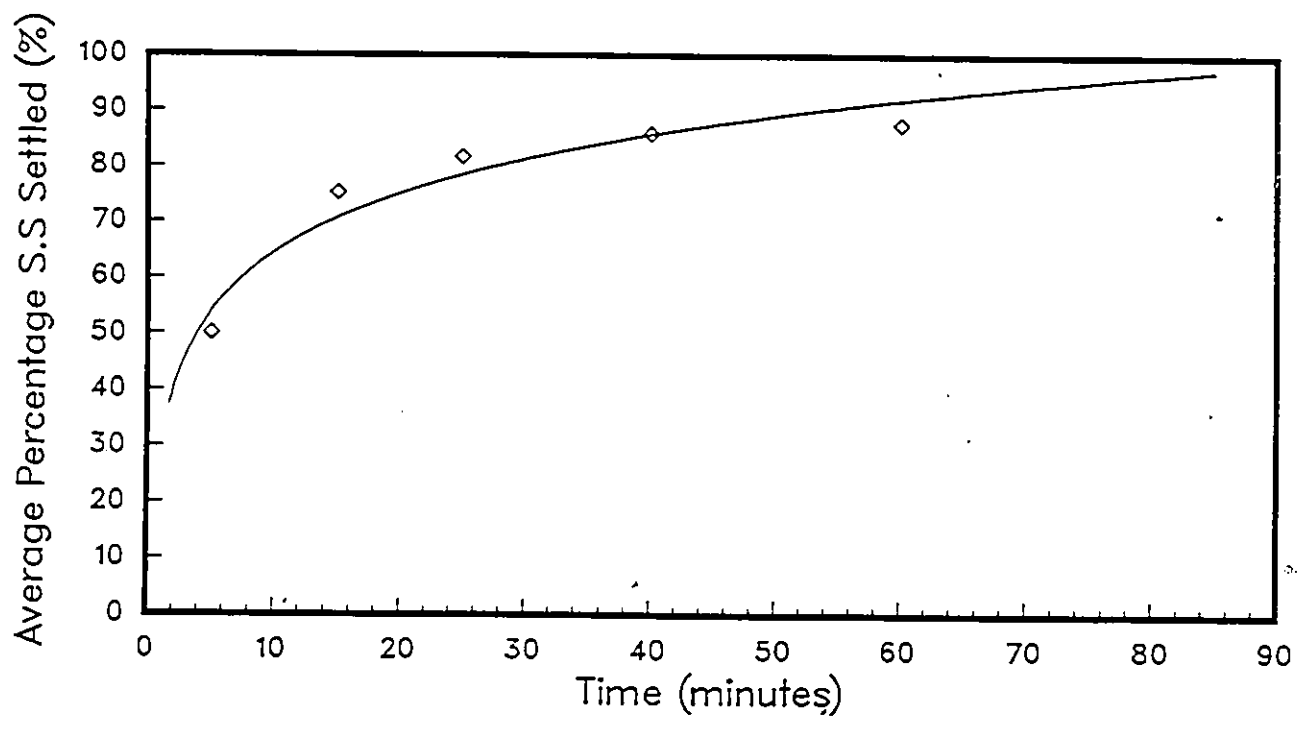


(b) Average Percentage S.S Settled vs Time.

Figure D.10: Settling Column Test of the Tank O.M.
 S= flat, Sludge Level= 43%

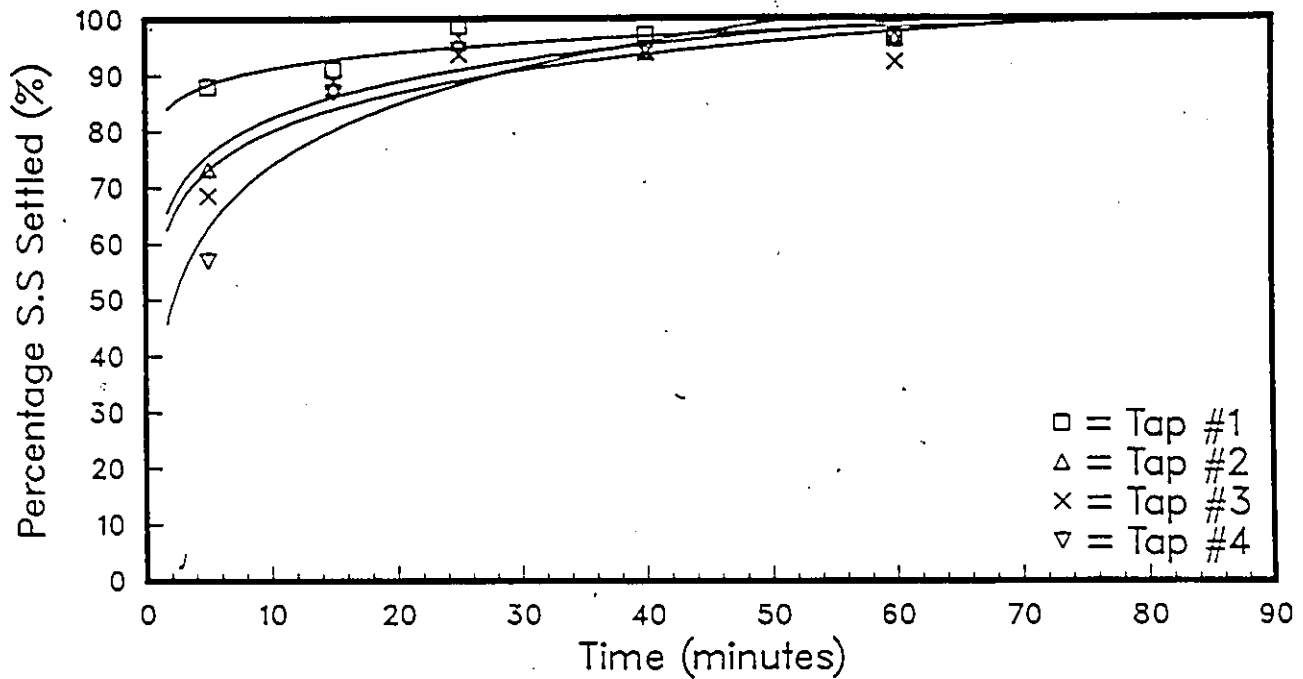


(a) Percentage S.S Settled vs Time.

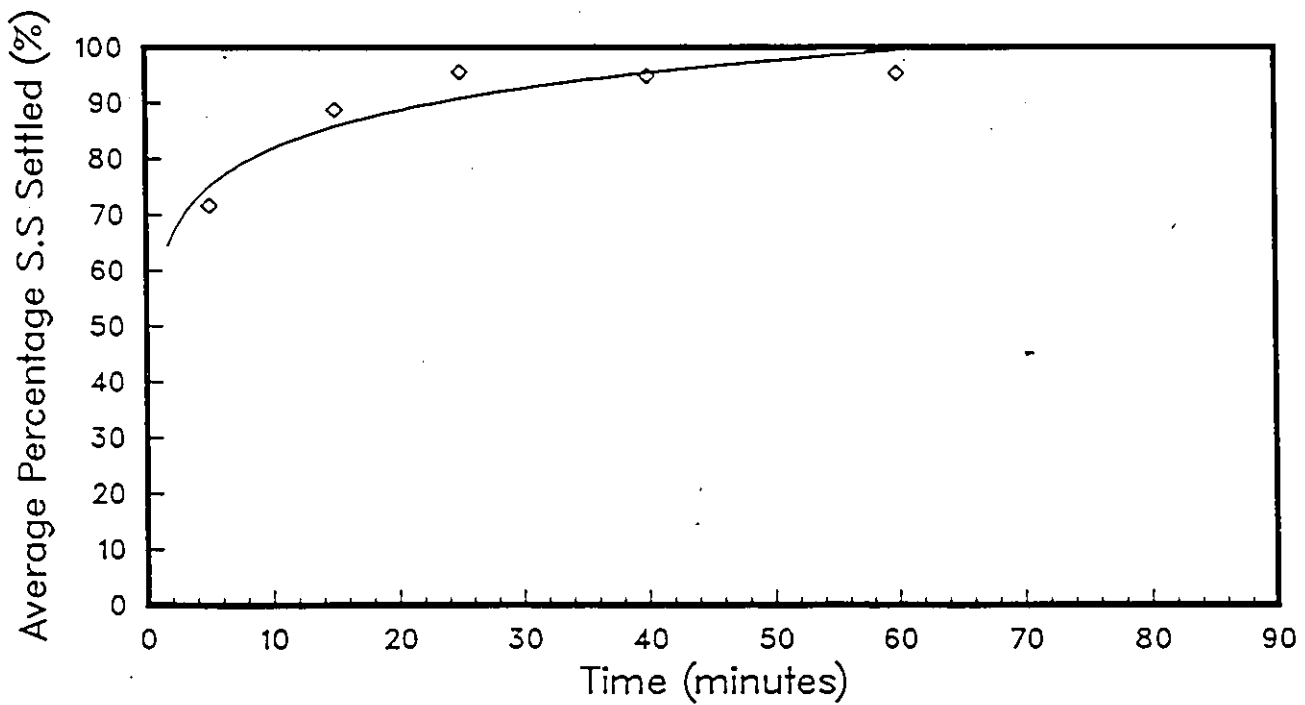


(b) Average Percentage S.S Settled vs Time.

Figure D.11: Settling Column Test of the Tank O.M.
S= flat, Sludge Level= 64%

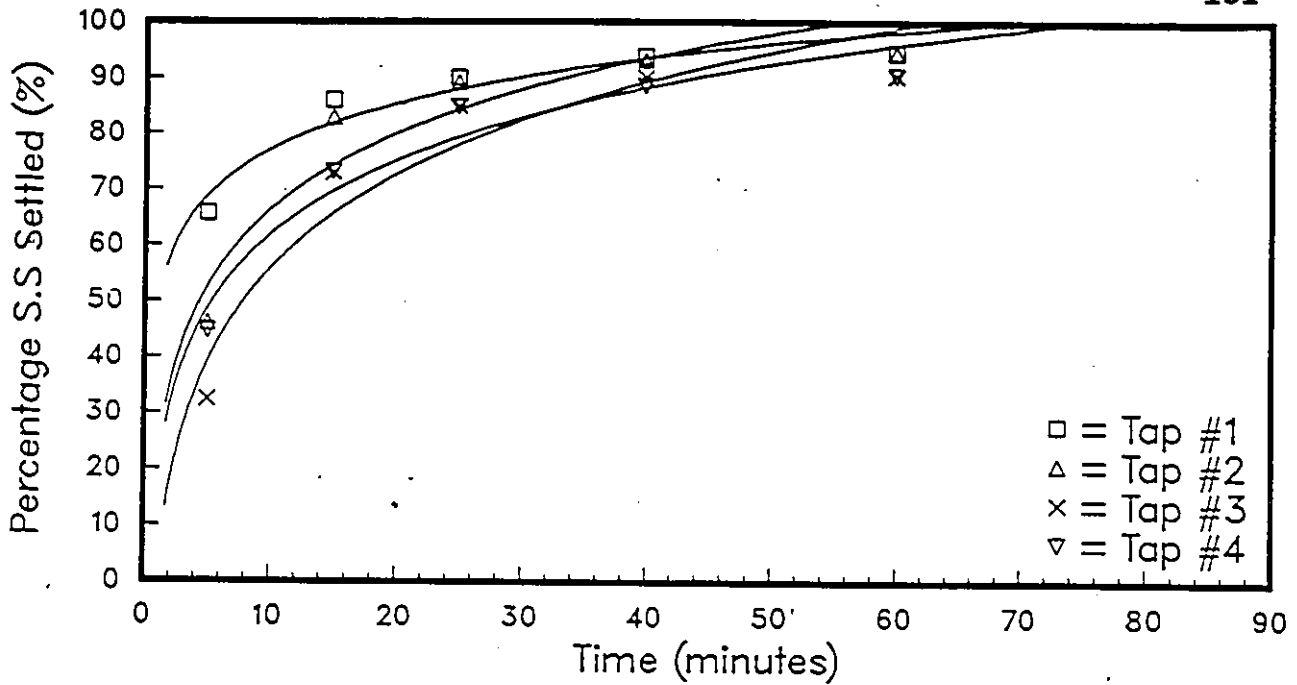


(a) Percentage S.S Settled vs Time.

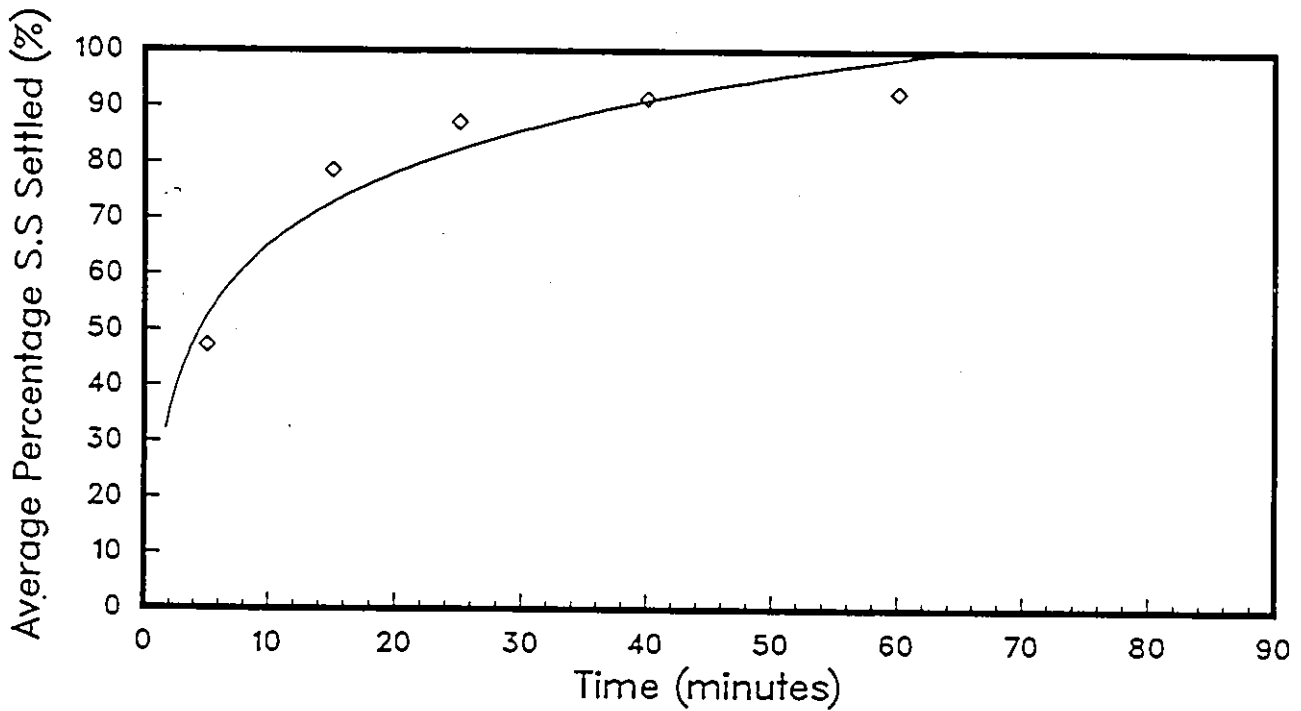


(b) Average Percentage S.S Settled vs Time.

Figure D.12: Settling Column Test of the Tank O.M.
 $S = 1:1$, Sludge Level = 21%

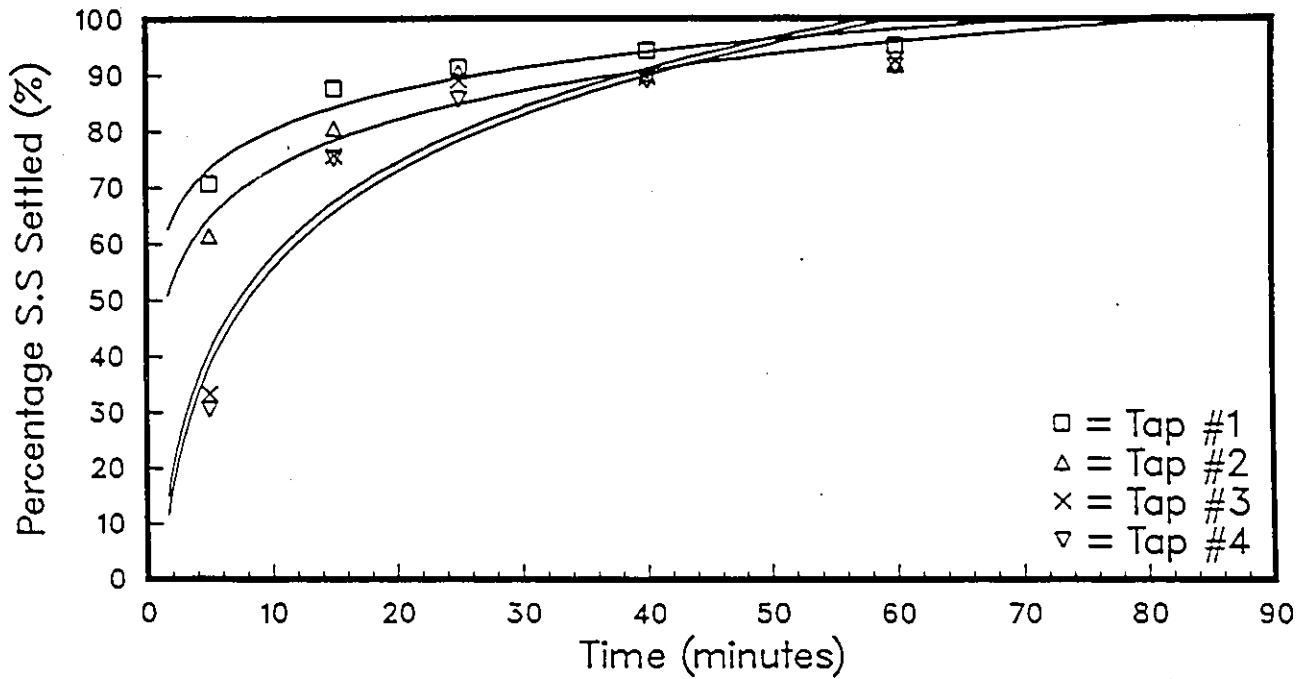


(a) Percentage S.S Settled vs Time.

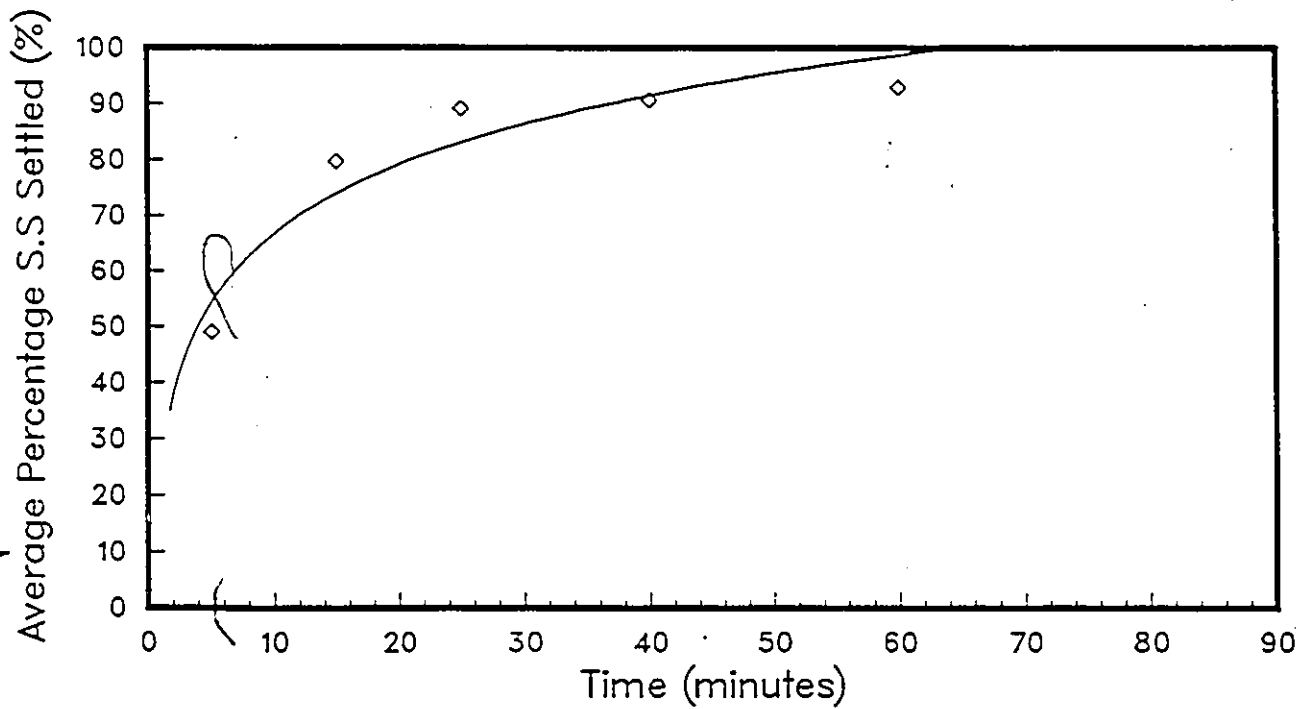


(b) Average Percentage S.S Settled vs Time.

Figure D.13: Settling Column Test of the Tank O.M.
 S= 1:1 , Sludge Level= 43%



(a) Percentage S.S Settled vs Time.



(b) Average Percentage S.S Settled vs Time.

Figure D.14: Settling Column Test of the Tank O.M.
 S= 1:1 , Sludge Level= 64%

APPENDIX E

SAMPLE PROCEDURE FOR THE ESTIMATION OF THE 15 COEFFICIENTS
AND THE ANALYSIS OF VARIANCE

The least squares method was used to estimate the 15 coefficients for the assumed second-degree polynomial model. The assumed model has the following form:

$$\begin{aligned}
 Y = & B_0 + B_1X_1 + B_2X_2 + B_3X_3 + B_4X_4 + \\
 & B_{11}X_1^2 + B_{22}X_2^2 + B_{33}X_3^2 + B_{44}X_4^2 + \\
 & B_{12}X_1X_2 + B_{13}X_1X_3 + B_{14}X_1X_4 + B_{23}X_2X_3 + \\
 & B_{24}X_2X_4 + B_{34}X_3X_4
 \end{aligned}
 \tag{E.1}$$

where Y = C_{222} , effluent SS concentration
 X_1 = coded variable for the slope S
 X_2 = coded variable for the volume V
 X_3 = coded variable for the distance D
 X_4 = coded variable for the length L

The detailed procedure is well documented in many statistical references, (Box, 1960; Bacon, 1970). The formulae for the estimation of the 15 coefficients and the analysis of variance are presented in Box (1960). In this section, only a numerical application for the estimation of the 15 coefficients for the case when the sludge level in the tank was 11% will be presented. The same notation as described in Box (1960) was used.

The procedure for the evaluation of the 15 coefficients is presented in Table (E.1). The first 4 columns represent the 27 combinations of coded variables which were performed ex-

Table E.1 : (Continued)

X_1X_2	X_1X_3	X_1X_4	X_2X_3	X_2X_4	X_3X_4	Y	Y*Y
0	-1	0	0	0	0	103	10609
0	1	0	0	0	0	206	42436
1	0	0	0	0	0	26	676
-1	0	0	0	0	0	83	6889
0	0	1	0	0	0	26	676
0	0	-1	0	0	0	33	1089
0	1	0	0	0	0	242	58564
0	-1	0	0	0	0	306	93636
-1	0	0	0	0	0	239	57121
1	0	0	0	0	0	316	99856
0	0	-1	0	0	0	244	59536
0	0	1	0	0	0	220	48400
0	0	0	0	0	1	216	46656
0	0	0	1	0	0	130	16900
0	0	0	-1	0	0	350	122500
0	0	0	0	0	-1	667	444889
0	0	0	0	-1	0	258	66564
0	0	0	0	1	0	645	416025
0	0	0	0	1	0	182	33124
0	0	0	0	-1	0	345	119025
0	0	0	0	0	0	207	42849
0	0	0	-1	0	0	167	27889
0	0	0	1	0	0	259	67081
0	0	0	0	0	-1	177	31329
0	0	0	0	0	1	185	34225
0	0	0	0	0	0	235	55225
0	0	0	0	0	0	196	38416

{ 0Y} {Y*Y}
6263 2042185

Table E.1 : (Continued)

X_1Y	X_2Y	X_3Y	X_4Y	$X_1=Y$	$X_2=Y$	$X_3=Y$	$X_4=Y$
-103	0	103	0	103	0	103	0
-206	0	-206	0	206	0	206	0
-26	-26	0	0	26	26	0	0
-83	83	0	0	83	83	0	0
-26	0	0	-26	26	0	0	26
-33	0	0	33	33	0	0	33
242	0	242	0	242	0	242	0
306	0	-306	0	306	0	306	0
239	-239	0	0	239	239	0	0
316	316	0	0	316	316	0	0
244	0	0	-244	244	0	0	244
220	0	0	220	220	0	0	220
0	0	-216	-216	0	0	216	216
0	-130	-130	0	0	130	130	0
0	350	-350	0	0	350	350	0
0	0	-667	667	0	0	667	667
0	-258	0	258	0	258	0	258
0	645	0	645	0	645	0	645
0	-182	0	-182	0	182	0	182
0	345	0	-345	0	345	0	345
0	0	0	0	0	0	0	0
0	-167	167	0	0	167	167	0
0	259	259	0	0	259	259	0
0	0	177	-177	0	0	177	177
0	0	185	185	0	0	185	185
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
{ 1Y}	{ 2Y}	{ 3Y}	{ 4Y}	{11Y}	{22Y}	{33Y}	{44Y}
1090	996	-742	818	2044	3000	3008	3198

Table E.1 : (Continued)

$X_1 X_2 Y$	$X_1 X_3 Y$	$X_1 X_4 Y$	$X_2 X_3 Y$	$X_2 X_4 Y$	$X_3 X_4 Y$
0	-103	0	0	0	0
0	206	0	0	0	0
26	0	0	0	0	0
-83	0	0	0	0	0
0	0	26	0	0	0
0	0	-33	0	0	0
0	242	0	0	0	0
0	-306	0	0	0	0
-239	0	0	0	0	0
316	0	0	0	0	0
0	0	-244	0	0	0
0	0	220	0	0	0
0	0	0	0	0	216
0	0	0	130	0	0
0	0	0	-350	0	0
0	0	0	0	0	-667
0	0	0	0	-258	0
0	0	0	0	645	0
0	0	0	0	182	0
0	0	0	0	-345	0
0	0	0	0	0	0
0	0	0	-167	0	0
0	0	0	259	0	0
0	0	0	0	0	-177
0	0	0	0	0	185
0	0	0	0	0	0
0	0	0	0	0	0
{12Y}	{13Y}	{14Y}	{23Y}	{24Y}	{34Y}
20	39	-31	-128	224	-443

perimentally. The column under the heading Y represents the effluent SS concentration for the corresponding 27 runs (i.e., C_{SS}), as shown in Table (5.1).

The notation { } in Table (E.1) represents the summation of all values of the same column. The first coefficient B_0 was calculated based on the average value of the effluent SS concentration made at the center point (i.e., $X_1=0$, $X_2=0$, $X_3=0$, and $X_4=0$).

For this particular design, the constants a, b, c, d, and s are fixed by the least squares method and must be used to estimate the 15 coefficients. The values of a, b, c, d, and s are 1/12, 1/8, -1/48, 1/4, and 2 respectively. The procedure for the estimation of the 15 coefficients is straight forward and presented in the following paragraphs.

The 15 coefficients are estimated as follows:

$$B_0 = (207+235+196)/3 = 212.7$$

$$B_{11} = a\{1Y\} = 1/12*(1090) = 90.8$$

$$B_{22} = a\{2Y\} = 1/12*(996) = 83.0$$

$$B_{33} = a\{3Y\} = 1/12*(-742) = -61.8$$

$$B_{44} = a\{4Y\} = 1/12*(818) = 68.2$$

$$B_{12} = b\{11Y\} + c(\{11Y\} + \{22Y\} + \{33Y\} + \{44Y\}) - (B_0/s) \\ = 1/8*(2044) - 1/48*(2044+3000+3008+3198) - (212.7/2) = -85.2$$

$$B_{23} = b\{22Y\} + c(\{11Y\} + \{22Y\} + \{33Y\} + \{44Y\}) - (B_0/s) \\ = 1/8*(3000) - 1/48*(2044+3000+3008+3198) - (212.7/2) = 34.3$$

$$B_{34} = b\{33Y\} + c(\{11Y\} + \{22Y\} + \{33Y\} + \{44Y\}) - (B_0/s) \\ = 1/8*(3008) - 1/48*(2044+3000+3008+3198) - (212.7/2) = 35.3$$

$$B_{45} = b\{44Y\} + c(\{11Y\} + \{22Y\} + \{33Y\} + \{44Y\}) - (B_0/s) \\ = 1/8*(3198) - 1/48*(2044+3000+3008+3198) - (212.7/2) = 59.0$$

$$B_{12} = d\{12Y\} = 1/4*(20) = 5.0$$

$$B_{13} = d\{13Y\} = 1/4*(39) = 9.8$$

$$B_{14} = d\{14Y\} = 1/4*(-31) = -7.8$$

$$B_{23} = d\{23Y\} = 1/4*(-128) = -32.0$$

$$B_{24} = d\{24Y\} = 1/4*(224) = 56.0$$

$$B_{34} = d\{34Y\} = 1/4*(-443) = -110.8$$

A summary of the least squares estimates of the 15 coefficients in the second-degree polynomial model is listed in Table (E.2) along with the analysis of variance of the fitted model in the coded variables.

$$\begin{aligned}
 Y = & 212.7 + 90.8 X_1 + 83.0 X_2 - 61.0 X_3 + 68.2 X_4 - \\
 & 85.2 X_1^2 + 34.3 X_2^2 + 35.3 X_3^2 + 59.0 X_4^2 + \\
 & 5.0 X_1 X_2 + 9.8 X_1 X_3 + 7.8 X_1 X_4 - 32.0 X_2 X_3 + \\
 & 56.0 X_2 X_4 - 110.8 X_3 X_4
 \end{aligned}
 \tag{E.2}$$

Table E.2 : Summary of the Estimated Coefficients and Analysis of Variance for the Fitted Model

Estimated Coefficient	Sum of Squares	Degrees of freedom	Mean Square
$B_0 = 212.7$	1452784	1	1452784
$B_1 = 90.8$	99008	1	99008
$B_2 = 83.0$	82668	1	82668
$B_3 = -61.8$	45880	1	45880
$B_4 = 68.2$	55760	1	55760
$B_{11} = -85.2$			
$B_{22} = 34.3$			
$B_{33} = 35.3$			
$B_{44} = 59.0$	102972	4	25743
$B_{12} = 5.0$	100	1	100
$B_{13} = 9.8$	380	1	380
$B_{14} = -7.8$	240	1	240
$B_{23} = -32.0$	4096	1	4096
$B_{24} = 56.0$	12544	1	12544
$B_{34} = -110.8$	49062	1	49062
Residual	136690	12	11391
Total	2042185	27	

The sum of the squares in Table (E.2) was calculated as follows:

Sum of squares associated with the coefficient $B_0 =$

$$1/27 \{0Y\}^2 \quad (E.3)$$

Sum of squares associated with $B_1 = a \{1Y\}^2 \quad (E.4)$

Sum of squares associated with $B_2 = a \{2Y\}^2 \quad (E.5)$

Sum of squares associated with $B_3 = a \{3Y\}^2 \quad (E.6)$

Sum of squares associated with $B_4 = a \{4Y\}^2 \quad (E.7)$

Sum of squares associated with $B_{11}, B_{22}, B_{33}, B_{44} =$

$$B_0 \{0Y\} + B_{11} \{11Y\} + B_{22} \{22Y\} + B_{33} \{33Y\} + B_{44} \{44Y\} \\ -1/27 \{0Y\}^2 \quad (E.8)$$

Sum of squares associated with $B_{12} = d \{12Y\}^2 \quad (E.9)$

Sum of squares associated with $B_{13} = d \{13Y\}^2 \quad (E.10)$

Sum of squares associated with $B_{14} = d \{14Y\}^2 \quad (E.11)$

Sum of squares associated with $B_{23} = d \{23Y\}^2 \quad (E.12)$

Sum of squares associated with $B_{24} = d \{24Y\}^2 \quad (E.13)$

Sum of squares associated with $B_{34} = d \{34Y\}^2 \quad (E.14)$

The residual sum of squares was obtained as follows :

$$\{Y*Y\} - (\text{summation of the sum of squares associated with} \\ \text{all coefficients } B_0 \dots B_{34}) \quad (E.15)$$

The significance of each term in the fitted model could be tested by comparing Eq. (E.16) with an appropriate value of the F-distribution.

$$\frac{\text{Mean square associated with the term}}{\text{Residual mean square}} \quad (E.16)$$

For the estimate $B_1 = 90.8$, the computed ratio from Eq. (E.16) was found to be $99008/11391 = 8.69$. With 1 and 12 de-

degrees of freedom the appropriate value of the F-distribution is 4.75. It is clearly seen that B_1 is significant in the model. The estimate B_2 (-61.8), however, was found to be insignificant in the model since the computed ratio of Eq. (E.16) is 4.03. The term $B_2 X_2$ can be dropped from Eq. (E.2). After testing all other terms, Eq. (E.2) can be reduced to the following form:

$$Y = 212.7 + 90.8 X_1 + 83.0 X_2 + 68.2 X_4 - 85.2 X_1^2 + 34.3 X_2^2 + 59.0 X_4^2 - 110.8 X_2 X_4 \quad (E.17)$$

Either Eq. (E.2) or Eq. (E.17) can be used to predict the effluent SS concentration for any combination of variables. Eq. (E.17) was developed to illustrate that the model represented by Eq. (E.2) can be simplified to a less cumbersome equation.

APPENDIX F

PROGRAM LISTING AND SAMPLE COMPUTER OUTPUT FOR THE RUNGE-
KUTTA METHOD

FILE: RUNGE FORTRAN • UNIV D'OF OTTAWA CMS

PAGE 00001

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C PROGRAM LISTING
C
C RUNGE-KUTTA METHOD
C
C NUMERICAL SOLUTION FOR THE DIFFERENTIAL EQUATION
C SAMPLE PROCEDURE FOR THE CASE (S= FLAT, V= 15 L, D= 0.1 M, L=0.1 M)
C
  INTEGER N,IND,NW,IER,K
  REAL Y(1),C(24),W(2,9),X,TOL,XEND,A,B,CE(1)
  EXTERNAL FCN1
  NW=1
  N=1
C BOUNDARY CONDITIONS
C X= TIME IN DAYS
C Y= TANK SS CONCENTRATION
C CE= EFFLUENT SS CONCENTRATION
  X=1
  Y(1)=200
  CE(1)=0.0
  TOL=0.0001
  IND=1
  DO 10 K=1,60
  XEND=FLOAT(K)*5
  CALL DVERK(N,FCN1,X,Y,XEND,TOL,IND,C,NW,W,IER)
C A AND B ARE THE REGRESSION COEFFICIENTS
  A=0.001
  B=3.168
  CE(1)=A*(Y(1)/1000)**B
  WRITE(2,*) X,Y(1)/1000,CE(1)
  IF (IND.LT.0.OR.IER.GT.0) GO TO 20
10 CONTINUE
  STOP
20 CONTINUE
  STOP
  END
  SUBROUTINE FCN1(N,X,Y,YPRIME)
  INTEGER N
  REAL YPRIME(N),Y(N),X
  REAL CIN,A,B
C CIN= TANK INFLUENT SS CONCENTRATION
  CIN=200
  A=0.001
  B=3.168
  YPRIME(1)= CIN-A*(Y(1)/1000)**B
  RETURN
  END

```

```

RUN00010
RUN00020
RUN00030
RUN00040
RUN00050
RUN00060
RUN00070
RUN00080
RUN00090
RUN00100
RUN00110
RUN00120
RUN00130
RUN00140
RUN00150
RUN00160
RUN00170
RUN00180
RUN00190
RUN00200
RUN00210
RUN00220
RUN00230
RUN00240
RUN00250
RUN00260
RUN00270
RUN00280
RUN00290
RUN00300
RUN00310
RUN00320
RUN00330
RUN00340
RUN00350
RUN00360
RUN00370
RUN00380
RUN00390
RUN00400
RUN00410
RUN00420
RUN00430
RUN00440
RUN00450
RUN00460

```

FILE: RUNGE RESULT * UNIV D'/OF OTTAWA CMS

PAGE 00001

C SAMPLE COMPUTER OUTPUT

C	TIME (DAYS)	TANK SS CONC. (G/L)	EFFLUENT SS CONC. (MG/L)
C	5.0000000	0.999998033	0.999993645E-03
	10.0000000	1.99997616	0.898765400E-02
	15.0000000	2.99988079	0.724686542E-01
	20.0000000	3.99960899	0.807589293E-01
	25.0000000	4.99900723	0.163705349
	30.0000000	5.99788189	0.291538775
	35.0000000	6.99598789	0.474767506
	40.0000000	7.99301529	0.724079728
	45.0000000	8.98860455	1.05025959
	50.0000000	9.98235130	1.46409512
	55.0000000	10.9737844	1.97628021
	60.0000000	11.9623938	2.59733105
	65.0000000	12.9476013	3.33745384
	70.0000000	13.9287930	4.20646763
	75.0000000	14.9052963	5.21366310
	80.0000000	15.8764019	6.36769009
	85.0000000	16.8413391	7.67645645
	90.0000000	17.7993469	9.14703751
	95.0000000	18.7495880	10.7854328
	100.000000	19.6911926	12.5965967
	105.000000	20.6233063	14.5843382
	110.000000	21.5450439	16.7511292
	115.000000	22.4554901	19.0980835
	120.000000	23.3537445	21.6248322
	125.000000	24.2389221	24.3297729
	130.000000	25.1101379	27.2095490
	135.000000	25.9665375	30.2595978
	140.000000	26.8072510	33.4736481
	145.000000	27.6315308	36.8443146
	150.000000	28.4385529	40.3626099
	155.000000	29.2276459	44.0183868
	160.000000	29.9981537	47.8007812
	165.000000	30.7494507	51.6973877
	170.000000	31.4810028	55.6950989
	175.000000	32.1923370	59.7805634
	180.000000	32.8830566	63.9392090
	185.000000	33.5528259	68.1567993
	190.000000	34.2014008	72.4187317
	195.000000	34.8285828	76.7099915
	200.000000	35.4342651	81.0164795
	205.000000	36.0184021	85.3237305
	210.000000	36.5810242	89.6177368
	215.000000	37.1222534	93.8862915
	220.000000	37.6422272	98.1157990
	225.000000	38.1411591	102.295166
	230.000000	38.6193542	106.413803
	235.000000	39.0771332	110.461365
	240.000000	39.5148621	114.429291
	245.000000	39.9329681	118.309296
	250.000000	40.3319092	122.094177
	255.000000	40.7121735	125.778427
	260.000000	41.0742950	129.357101
	265.000000	41.4187775	132.825165
	270.000000	41.7462158	136.180359
	275.000000	42.0571594	139.420105
	280.000000	42.3521881	142.542038
	285.000000	42.6319122	145.545731
	290.000000	42.8969116	148.431229
	295.000000	43.1477966	151.199051
	300.000000	43.3851166	153.849106

APPENDIX G

NUMERICAL EXAMPLE

To illustrate the design of an interceptor tank for small-bore sewers, a numerical example is presented.

Assume the following:

- The volume of the interceptor tank is 1.5 m^3 ,
- The most frequently flushed volume is 15 L,
- The inlet suspended solids concentration is 200 mg/L,
- The maximum loading capacity of small-bore sewers is 100 mg/L.

Select the geometry of the interceptor tank (in terms of inlet and outlet locations), and determine the desludging interval.

To extend the desludging interval, the best geometry of the interceptor tank is obtained from Table (5.5a). Since the effluent SS concentration criterion is 100 mg/L, the inlet can be located 0.5 m from the bottom of the interceptor tank and the outlet can be located 0.5 m from the side wall of the tank. For this particular design, it takes 1280 days for the effluent SS concentration criterion to be reached. The desludging interval is approximately 3.5 years.