HYDRAULICS OF WASTE STABILIZATION PONDS AND
ITS INFLUENCE ON TREATMENT EFFICIENCY

by

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LIST OF SYMBOLS

\( a \) = thermal diffusivity

\( A_m \) = finite stage model constants

\( B_m \) = finite stage model constants

\( C(\theta) \) = C-diagram

\( c \) = concentration of tracer as a function of \( \theta \) (mg/V)

\( c_o \) = mass of tracer divided by volume of pond (mg/V)

\( \frac{c}{c_o^{pk}} \) = dimensionless peak concentration

\( D \) = axial dispersion coefficient (ft\(^2\)/hr)

\( D_{mass} \) = mass diffusion coefficient (ft\(^2\)/hr)

\( d \) = diffusivity coefficient (dimensionless)

\( d_o \) = depth of pond

\( E_h \) = eddy diffusivity of heat transfer

\( E_m \) = eddy diffusivity of momentum

\( F(\theta) \) = F-diagram

\( F_a \) = perfectly mixed flow area as a fraction of total system volume

\( F_b \) = fraction of total system volume as dead flow region

\( F_c \) = fraction of total system volume as plug flow (delay time)

\( F_e \) = Froude number

\( F_{\Delta} \) = densimetric Froude number

\( \frac{F_{\Delta}}{R_e} \) = densimetric Froude-Reynolds number

\( f_a \) = finite stage model constant
\text{g} = \text{gravity (ft/sec}^2\text{)}

\text{K} = \text{first order reaction coefficient (day}^{-1}\text{)}

\text{K}_h = \text{multiple of main flow interchange between live and dead flow regions}

\text{L} = \text{pond length}

\ell = \text{characteristic length}

\text{M} = \text{mass of tracer as a function of time (mg)}

\text{M}_t = \text{initial mass of tracer (mg)}

\text{N} = \text{waste material concentration as a function of time (mg/l)}

\text{N}_o = \text{initial concentration of waste material (mg/l)}

\overline{\text{N}}_\infty = \text{waste material concentration accounting for all pollutant elements (mg/l)}

\overline{\text{N}}_2 = \text{waste material concentration at two theoretical detention times (mg/l)}

\text{n} = \text{number of basic modules in series}

\text{P}_r = \text{Prandtl number}

\text{Q} = \text{flowrate}

\text{Q}_f = \text{fictitious flowrate}

\text{R}_e = \text{Reynolds number}

\text{R}_i = \text{Richardson number}

\text{r}_1 = \text{finite stage model constant}

\text{r}_2 = \text{finite stage model constant}

\text{S}_c = \text{Schmidt number}

\text{t} = \text{time}

\overline{\text{t}} = \text{theoretical detention time}

\overline{\text{t}}_c = \text{experimental detention time}
$U$ = average fluid velocity
$U_o$ = reference fluid velocity
$V$ = pond volume
$V_f$ = fictitious pond volume
$V_d$ = dead space volume
$\overline{V_d}$ = dead space parameter
$W$ = pond width
$X$ = independent variable—finite stage model
$x$ = horizontal coordinate
$Y$ = dependent variable—finite stage model
$z$ = vertical coordinate
$\theta$ = dimensionless time
$\theta_b$ = dimensionless time when tracer first appears at outlet
$\overline{\theta_c}$ = dimensionless experimental mean
$\theta_{pk}$ = dimensionless time to peak concentration of tracer
$\theta_{pf}$ = plug flow deviation parameter
$\rho$ = density
$\rho_o$ = reference density
$\nu$ = kinematic viscosity or diffusivity of momentum
ABSTRACT

Hydraulics of Waste Stabilization Ponds and its Influence on Treatment Efficiency

by

Kenneth A. Mangelson, Doctor of Philosophy

Utah State University, 1971

Major Professor: Dr. Gary Z. Watters
Department: Civil Engineering

The treatment efficiency of waste stabilization ponds depends on many factors, the most important of which are generally presumed to be the biological factors such as the type of waste and the organic loading. However, the biological activity in a pond is greatly influenced by the environmental conditions of temperature, wind, sunlight, and the hydraulic flow patterns. In the past little attention has been given to the hydraulic characteristics of waste stabilization ponds. Specifically, little consideration has been given to the gross flow patterns within stabilization ponds as affected by the shape of the pond or lagoon, the presence of dead spaces, the positioning of inlets and outlets and the degree of density stratification. These hydraulic flow characteristics will obviously have an effect on the dispersion of the waste as well as the average detention time for the waste and, ultimately, on the organic (BOD) and pathogenic organism removal efficiency of the treatment process.
This research considered the effects of these hydraulic flow characteristics on the treatment efficiency. The approach was to use certain information that can be obtained from the age distribution function of the fluid particles within a continuous flow process vessel. The age distribution function represents a history of the time of retention of the various fluid particles in the vessel and is generated by injecting a tracer into the process vessel and monitoring the outlet from the vessel. The concentration vs. time curves at the outlets, which lead to the age distribution functions for a waste or a tracer, were made dimensionless to aid in the evaluation of each experiment.

The prototype experimental data taken to establish existing flow patterns were obtained on the waste stabilization ponds of the city of Logan, Utah. Fluorometric techniques using rhodamine WT dye were used to trace the pollutant. A hydraulic model of the ponds 20 feet by 40 feet by 3 feet deep was constructed at the Utah Water Research Laboratory. This model, after verification, was used to generate data on the effects of inlet and outlet types and location, density stratification, length to width ratio, and baffling on the hydraulic flow characteristics.

The information gained from the tracer concentration vs. time curves was used in conjunction with the first order reaction equation to predict treatment efficiencies for various pond designs for determining optimum conditions.
Finally, a mathematical model of the mixing process is presented and outlet concentration vs. time curves generated by the model are compared with experimental results. This mathematical model can be used in conjunction with the first order reaction equation to predict treatment efficiencies.

The results of the experiments performed on the existing ponds in the Logan pond system are presented in Chapter 3. The results of the model experiments are presented in Chapter 5. The age distribution function diagrams and the significant parameters determined from these diagrams are presented along with the expected treatment efficiency for each model experiment. Also, derived mathematical models for the age distribution functions are illustrated along with experimentally determined age distribution functions for various pond and model experiments.
A waste stabilization pond can be defined as a shallow man-made basin which utilizes natural processes under partially controlled conditions for the reduction of organic matter and the destruction of pathogenic organisms in waste waters. Generally the design of the pond has been based on past experience utilizing empirical data. In the last ten years there has been an increase in interest in stabilization ponds as a method of waste treatment, basically because of the relatively low cost of construction and operation in comparison with conventional waste treatment processes. Recent design approaches have been related more to basic scientific principles rather than being based entirely on empirical data.

In stabilization ponds, the environment is beyond the control of the designer which adds to the already extreme complexity of pond behavior. To provide ponds which will be as efficient and economical as possible it is necessary to study the physical, biochemical, and environmental factors which influence pond behavior. This research was designed to basically study the physical factors with some reference to the environmental conditions that influence pond behavior.
The treatment efficiency of waste stabilization ponds depends on many factors, the most important of which are generally presumed to be the biological factors such as the type of waste and the organic loading. However, the biological activity in a pond is greatly influenced by the environmental conditions of temperature, wind, sunlight, humidity, and such physical factors as pond geometry, inlets and outlets, and the hydraulic flow patterns. In the past little attention has been given to the hydraulic characteristics of waste stabilization ponds. Specifically, little consideration has been given to the gross flow patterns within stabilization ponds which are affected by the shape of the pond or lagoon, the presence of dead spaces, the existence of density differences, and the positioning of inlets and outlets. These hydraulic flow characteristics will obviously have an effect on the dispersion of the waste as well as the average detention time for the waste and, ultimately, on the organic (BOD) and pathogenic organism removal efficiency of the treatment process.

This study considers the effects of these hydraulic flow characteristics on the treatment efficiency. The approach was to use information obtained from the age distribution functions of the fluid particles within continuous-flow process vessels or tanks. The age distribution functions represent a history of the retention of the various fluid particles in the vessel. Chemical engineers first introduced the general method of describing flow patterns by the use of age distribution
functions generated by injecting a tracer into the process vessel. A stabilization pond can be considered a biological or chemical process vessel. In addition, the biological reactions that occur in waste stabilization ponds have been found to closely follow a first order chemical reaction. Consequently, the age distribution functions can be coupled with the first order reaction equation to give a quantitative measure of treatment efficiency.

The application of a physically significant mathematical model for describing the flow patterns by use of age distribution functions is presented. This model, called the finite stage model, consists of building blocks which are composed of live and dead flow regions. The live flow section consists of plug flow and completely mixed flow regions, while the dead flow region is completely mixed and interchanges fluid slowly with the live flow region. This model was used because of its obvious physical similarity to real flow systems. The mathematical model can be derived from model studies or existing pond studies and then coupled with the first order reaction equation to give the expected treatment efficiency. After mathematical models for the model pond designs have been determined, they can be used to predict pond performance of proposed new designs.

The prototype experimental data on hydraulic circulation were taken in the waste stabilization ponds operated by the city of Logan, Utah. These ponds were also modeled in the Utah Water Research
Laboratory using a 20 foot by 40 foot tank 3 feet deep. The model was verified and then used to generate data on the effect of inlet and outlet placement and baffling on the hydraulic flow characteristics. In addition, the sensitivity of the model to Reynolds's number variation and model scale distortion was measured to provide some information on how precisely modeling laws must be satisfied to achieve good agreement between model and prototype. Additional model experiments were performed to determine the influence on pond performance of density currents in a density stratified flow.

**Literature Review**

**Review of waste stabilization ponds**

General concept and fundamental principles

The terms "stabilization pond," "sewage lagoon," "oxidation pond," and others are generally applied to artificially created bodies of water intended to retain waste flows containing degradable organic compounds until biological processes render them stable and hence either unobjectionable from an oxygen-demand viewpoint for discharge into natural waters or removal by percolation and evaporation. The theoretical minimum detention time of these ponds is that sufficient to permit biodegradation of organic matter and die-away of pathogenic bacteria and parasites. The theoretical maximum time is the minimum
time plus that necessary to tie up the stable products of biodegradation in algal cells.

Stabilization ponds are generally classified according to types of inflow or outflow conditions. Some of these types are raw sewage ponds, primary and secondary sewage ponds, overflow and non-overflow ponds and others. They are also classified according to the types of biological processes that occur.

The three types of biological conditions that occur in stabilization ponds are (1) aerobic conditions, (2) anaerobic conditions, and (3) combined aerobic and anaerobic conditions termed facultative.

The biological process depends on the effective use of bacteria for the degradation of organic material and the availability of green algae for oxygen-production. The bacteria break down and use up many complex organic waste materials; the algae, helped by fungi, consume the simpler degradation products.

The relative rate of production and consumption of oxygen by algae and bacteria respectively determines the nature of the pond. Whenever the rate of production of oxygen by algae is greater than the rate of consumption of oxygen by bacteria, the process is termed "aerobic." If the reverse is true, "anaerobic" conditions prevail.

Anaerobic conditions are characterized by the foul odors accompanying this biological process. This is due to the complex products of fermentation, mainly hydrogen sulfide. The gaseous product of aerobic
digestion is carbon dioxide which makes this process relatively odor free.

Most stabilization ponds are actually facultative; that is, they have combined aerobic and anaerobic zones. Aerobic conditions occur in the upper strata of the water body where greater sunlight penetration activates the photosynthesis process to greater completion than at lower depths. Anaerobic conditions occur in the lower depths because of reduced sunlight penetration which results in an oxygen deficiency. Even in shallow ponds, anaerobic conditions often exist because of the settleable organic matter at the bottom which depletes the available oxygen. Gloyna (no date given) says that the design and operation of all waste stabilization ponds must aim at an algal-bacterial balance wherein the amount of waste water discharged into the pond commensurates with the amount of available dissolved oxygen.

The fundamental principles underlying the biological operation of waste stabilization ponds is that their action depends upon the simultaneous and continuous functioning of both the right-hand and left-hand sectors of the aerobic cycle of organic growth and decay (McGauhey, 1968). The conventional system carries out only the degradation process and leaves the growth potential to be exerted in the receiving water. The significance of this principle is that the input to sewage ponds is biodegradable dead organic wastes and the output is living organic matter at a higher energy level. This brings up one very important consideration. The potential biological oxygen demand (BOD) of the effluent may be
greater than that of the influent. The living algal cells, however, are not quickly available for biodegradation because of their hardiness of life. But the underlying factor seems to be that in terms of water quality, the stabilization pond effluent may substitute an aesthetic factor for the quality factors associated with biodegradation unless the algal cells are harvested.

The following summarizes some of the most important factors in the operation of stabilization ponds.

1. Algae--The life cycle of algae influences the biology and chemistry of the aquatic environment greatly. The growth rate of algae is influenced by available light source, temperature, the presence of nutrients, pH and other environmental factors.

2. Light Energy--This factor has a direct influence on the growth of algae.

3. Temperature--This factor also, has a direct influence on algae growth as well as the hydrodynamics of stabilization ponds.

4. Bacterial Nutritional Requirements--For the most efficient reduction of organic wastes through biological oxidation adequate nutrient supplies should be provided for the bacteria. Deficiency in any of the important substances such as phosphorous, nitrogen, and sulfates, and minerals such as potassium and calcium, will result in a serious curtailment of bacterial growth and activity.

5. Algal Nutrients--These are derived from two main sources, (a) the photosynthetic process by which carbon dioxide together with ammonia and other nitrogen containing compounds produced by the
decomposition of organics and released by hydrolysis, and (b) endogenous metabolism by which algae use the products of degradation of other algae (Rich, 1963).

6. Physical Environment--The hydrodynamic and physical shape of the pond along with such parameters as depth, length, width, inlet and outlet devices and location, and the porosity of the soil have a distinct influence on the ecological performance of the pond. Hermann and Gloyna (1958) have shown that the BOD removal efficiency, for given influent loads and detention periods, is a function of pond depth. Thermal microstratification (Stahl and May, 1967) and weed growth (McKinney, 1967) have also been shown to affect BOD removal efficiency. The pond surface area has been shown to be directly related to wind agitation, reaeration, evaporation, percolation, and precipitation (Oswald, 1963).

Design consideration and criteria

The previous approaches to the design of waste stabilization ponds have been primarily empirical with such parameters as depth, retention time, physical shape of pond, and BOD reduction derived from observed practical experience. Advances in biological oxidation, photosynthetic phenomena, and algalogy make the theoretical approach both possible and feasible from the biological standpoint.

The design parameters which must be established for stabilization ponds include detention period, hydraulic loading, depth, recirculation, mixing, pond size and shape, and inlet and outlet systems. Because
there are many different methods for determining these parameters and design methods for the three types of ponds, aerobic, anaerobic, and facultative, a detailed listing of these will not be made here. But those parameters, which are all determined in essentially the same way, will be briefly mentioned.

1. Detention period (theoretical)--the volume of the pond divided by the flowrate into the pond.

2. Hydraulic loading--the depth of the pond divided by the theoretical detention time.

3. Depth.

The determination of the pond depth is somewhat arbitrary depending on the pond type. Most of the other parameters have been determined essentially from past experience with sewage ponds.

There has been at least one attempt to determine these design parameters from an experimental model and incorporate these into a design equation based on chemical reactor design methods (Thirumurthi, 1969). The goal was to try and develop design formulas based on sound scientific and mathematical principles related to chemical engineering unit operations and reactor design concepts.

It is felt by some that the future of waste stabilization ponds as a method of treatment depends on the improvement of design, and it appears that, initially, the problems of hydraulic design must be solved. Present design concepts tend to neglect the shape of the treatment pond, the
existence of dead spaces, short circuiting, density differences, and the inlet and outlet flow patterns. These hydraulic flow characteristics will have an obvious effect on molecular and turbulent diffusion as well as the detention time and, hence, on BOD removal efficiency.

Chemical reactor design

In recent years, some researchers (Oswald, 1963, and Thirumurthi, 1969) have tried to develop a design method based on sound scientific principles. The approach was to develop a method for designing algal waste stabilization ponds based on existing chemical-engineering practice for reactor design. The discussion of these approaches will be made later. First, some of the important chemical engineering terms must be defined.

1. **Chemical reactor** - A chemical reactor is a vessel in which a chemical reaction takes place. There are three general types of reactors, namely, the batch, the steady flow, and the unsteady flow reactors. A steady flow reactor is one in which the influent and effluent flow rates remain constant with time and as a result it is mathematically easier to work with than the unsteady flow reactor. Also, the steady flow reactor is more realistic than a batch reactor from the viewpoint of a sanitary engineer because the biological processes of stabilization pond operation are continuous rather than batch. Those two reasons point out the desirability of using steady flow reactor principles in the design of waste stabilization ponds.
2. **Types of fluid flow in a vessel** - When a fluid passes through a reactor, tank, or pond, a number of possible patterns of flow could exist depending on the entrance and exit arrangements, short circuiting, flow rate, velocity, volume of tank, and fluid properties. The two general types of flow are: (a) ideal flow (b) non-ideal flow.

(a) **Ideal flow** - Consists of two types: plug flow (piston, slug, tabular, or non-mix flow), which is characterized by the fact that the flow of fluid through the tank is orderly with no element of fluid overtaking any other element as shown in Figure 1-a. Consequently, there is no velocity gradient and diffusion. The residence or detention time of all fluid elements is the same. **Completely mixed** (total back mix, or stirred tank) flow is characterized by a uniform composition in the tank (Figure 1-b). Any fluid element has an equal chance of being found at the outlet.

(b) **Non-ideal flow** - An actual process vessel, like an aeration tank or a waste stabilization pond, is obviously far from ideal. In an actual situation stagnant pockets or dead space, short circuiting and dispersion will occur to create non-ideal type flows characterized as channeling, recycling, eddying, etc. Since the flow in a tank or pond is in reality non-ideal, it is obvious that the problem should be approached using non-ideal flow terms.

3. **Stimulus-response methods of characterizing flow** - A tracer or stimulus is applied at the inlet to a tank or vessel and its response is measured as a function of time at the outlet.
Figure 1. Ideal hydraulic flow patterns.
4. **Open and closed vessels** - A closed vessel is one for which the fluid moves in and out by bulk flow alone. Plug flow exists in the entering and leaving streams. An open vessel is one where neither the entering nor the leaving fluid streams satisfy the plug flow requirements of the closed vessel. A waste stabilization pond is considered to be a closed vessel.

5. **Mean residence time of fluid (theoretical)** - The theoretical detention time, \( \bar{t} \), is defined as the volume of the vessel divided by the flowrate through the vessel.

6. **Reduced or dimensionless time** - The dimensionless time \( \theta \), is defined as time, \( t \), divided by the theoretical detention time, \( \bar{t} \).

Age distribution functions

Danckwerts (1953) first introduced the general idea of age distribution functions. These functions give information about the fluid that resides a certain time in a closed vessel. This type of treatment does not give information about point-to-point changes of the variables and thus does not yield complete information about the behavior of the fluid in the vessel. But it does give some good information about the general behavior.

Some of the most useful age distribution functions follow:

**F-diagrams.** If the incoming fluid flowing into a vessel is suddenly changed from a white to a red color, then \( F(\theta) \) is the fraction of red material that occurs at the outlet as a function of dimensionless time. A
plot of $F(\theta)$ vs. $\theta$ is called the F-diagram. Figure 2 shows F-diagrams for some representative types of flow functions.

Since there is always some longitudinal mixing with Newtonian fluids, due to viscous effects and molecular or eddy diffusion, the perfect piston or plug flow (curve A, Figure 2) will never occur. Curve B in Figure 2 shows a representative F-diagram for flow with some longitudinal mixing. Curve C is a diagram for complete mixing given by the equation

$$F(\theta) = 1 - e^{-\theta}$$

Curve D is the F-diagram where there is considerable "dead water." The term "dead water" or "stagnant pockets" means that some fraction of the fluid is trapped in corner eddies, and spends much more than the average length of time in the vessel.

The shape of an F-diagram depends on the relative times taken by various portions of the fluid to flow through the vessel or in other words, on the distribution of particle residence times.

C-diagrams. The same information as that obtained with an F-diagram can be obtained with a C-diagram. If a known amount, $M_T$, of tracer is instantaneously injected into the inlet, then $C(\theta)$ is the dimensionless concentration of that tracer at the outlet as a function of dimensionless time. C-diagrams are dimensionless plots similar to F-diagrams where

---

1 The F-diagram, in reality, is a dimensionless mass diagram.
Figure 2. The F-diagram.

Figure 3. The C-diagram.
C(θ) = \frac{c}{c_0}, \ c \ \text{is the tracer concentration as a function of time, } \ c_0 = \frac{M_T}{V}

and θ is defined the same as for F-diagrams. Figure 3 shows some typical C-diagrams for the systems whose F-diagrams are given in Figure 2.

The relationship between F-diagram and C-diagrams can easily be shown as:

\[
C = \frac{c}{c_0} = \frac{dF}{d\theta} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (2)
\]

Equation (3) represents the total fraction of tracer material that has left the vessel at the exit having ages between 0 and θ.

\[
\int_0^\theta \frac{c}{c_0} \, d\theta = F(\theta) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (3)
\]

Also the total area under every C-diagram is equal to unity.

Previous design approaches—biological reactors

If a mass balance is determined for a one-dimensional flow through a chemical reactor with a first order chemical reaction occurring, the following differential equation results:

\[
D \frac{\partial^2 N}{\partial x^2} - U_0 \frac{\partial N}{\partial x} - KN = 0 \quad \ldots \ldots \ldots \ldots \ldots \ldots (4)
\]

in which

\[
D = \text{axial dispersion coefficient}
\]
\[ K = \text{first order reaction coefficient} \]

\[ N = \text{reactant concentration} \]

\[ U_0 = \text{fluid velocity} \]

\[ x = \text{coordinate in direction of flow} \]

This equation has been solved analytically by Wehner and Wilhellem (1958).

For any kind of entrance and exit conditions the solution is:

\[
\frac{N_\infty}{N_0} = \frac{4a e^{1/2d}}{(1+a)^2 e^{a/2d} - (a-a)^2 e^{-a/2d}}
\]

\[ \bar{N}_\infty = \text{effluent concentration of reactant} \]

\[ N_0 = \text{influent concentration of reactant} \]

\[ a = 1 + 4Kt_c d \]

\[ d = \text{diffusivity coefficient} \]

\[ \bar{t}_c = \text{actual detention time} \]

The term \( d \) in Equation (5) characterizes the non-ideal flow in a pond.

\[
d = \frac{D}{U_0 l} \]

in which

\[ D = \text{axial dispersion coefficient} \]

\[ U_0 = \text{fluid velocity} \]

\[ l = \text{a characteristic length or size} \]

The coefficient, \( d \), varies from zero for plug flow to infinity for completely mixed flow. For non-ideal flow situations \( d \) will assume a value between zero and infinity. This equation is a conversion equation and
when solved represents the performance of a reactor. The equation is based on the concept of a dispersion model which represents the resident time distribution of fluid particles \( c(t) \) with \( d \) the important variable that characterizes the non-ideal nature of the flow.

Thirumurthi and Nashashibi (1967) have used Equation (5) in their research. They have stated that short-circuiting in tanks, exit and entrance hydraulic devices and other hydraulic mixing characteristics can be represented by the value of \( d \). Also, temperature, influent waste qualities, nutrient deficiencies, organic load and other biological factors can be accounted for by the value of \( K \). The hydraulic load is represented by the value of the actual (mean) detention time, \( \bar{t_c} \).

Thirumurthi and Nashashibi simulated a waste stabilization pond with a glass rectangular tank \((8'' \times 10'' \times 18'')\), with fluorescent lights and a synthetic chemical sewage. They determined values of \( K \), \( d \), and \( \bar{t_c} \) from their experiments. There was no direct tie with existing ponds to verify their work. However, they felt, from the results of their research, that the application of chemical engineering reactor design principles fits the complex biological reactions reasonably well.

Murphy and Timpany (1967) and Murphy and Boyko (1970) have also used a similar approach in their research to develop a design procedure and a method of analyzing aeration and spiral-flow aeration tanks.
Modeling of stratified flow

In designing a model to simulate existing conditions in waste stabilization ponds, the following types of situations can occur:

1. Inflow fluid more dense than ambient fluid (summer time flows).
2. Inflow fluid of same density as ambient fluid.
3. Inflow less dense than ambient fluid.

In reviewing past work performed on stratified flows and density currents caused from either temperature gradients or variations in composition (i.e. varying salt concentrations) use has been made of the densimetric Froude number as a modeling criteria in most all cases.

Stefan and Schiebe (1970) in their model studies of heated water flow into impoundments used an inlet Reynolds number and a densimetric Froude number with additional dimensionless heat transfer terms. The densimetric Froude number is defined as

\[ F_\Delta = \frac{U_0}{\sqrt{\Delta \rho / \rho g \ell}} \]  

in which

- \( F_\Delta \) = densimetric Froude number
- \( U_0 \) = reference fluid velocity
- \( \Delta \rho / \rho \) = degree of stratification-density difference divided by a reference density
- \( g \) = gravity
- \( \ell \) = characteristic length
Barr (1963b) used a densimetric Froude number as a criterion for similarity. He noted that sometimes comparisons of model and prototype behavior are made on the basis of the Richardson number $R_i$, although the Richardson number was originally intended as a criterion for the degree of stability of turbulence in flows of varying stratification. $R_i$ given by Prandtl is:

$$R_i = -\frac{g}{\rho} \frac{d\rho}{dz} \sqrt{\frac{du}{dz}}^2$$  \hspace{1cm} (8)$$

in which

- $R_i$ = Richardson number
- $u$ = fluid velocity
- $z$ = vertical coordinate

Barr defines $R_i$ for simulation criterion as:

$$R_i = -\frac{g}{\rho} \frac{\Delta \rho}{l} \sqrt{\frac{u}{l}}^2 = -F_{\Delta}^{-2}$$  \hspace{1cm} (9)$$

Barr derives the densimetric Froude number in two ways: (1) by expressing the ratios of the inertial and gravitational forces for model and prototype. (2) By energy concepts (kinetic energy available for mixing equals work done against the density gradient).

Barr stated that if the model scale must be distorted, i.e., the height ratio between model and prototype different than the length ratio, the discharge should be increased to maintain turbulent flow and allow the viscous forces to be neglected. If the discharge is increased, the
relative densities should be adjusted to maintain Richardson or densimetric Froude similarity.

This adjustment is:

\[
\left( \frac{\Delta \rho}{\rho} \right)_m = M^2 \left( \frac{\Delta \rho}{\rho} \right)_p \tag{10}
\]

in which the subscript \( m \) refers to the model and \( p \) refers to prototype, where

\[
M = \frac{Q_m}{Q_m} \text{ (computed)}
\]

and

\[
Q_m = \text{the actual flowrate in the model}
\]

\[
Q_m \text{ (computed)} = \text{the flowrate from model prototype relationships for the model for Reynolds numbers}
\]

Barr also suggested that exaggeration of the horizontal length scale is necessary for correct simulation of the rate of spread of the stratified layer.

In a later paper Barr (1963a) discussed modeling parameters for simulation of salinity intrusion. He combines \( F_\Delta \) and \( R_e \) to give the densimetric Froude-Reynolds number criterion for model simulation.

\[
\sqrt{\frac{F_\Delta}{R_e}} = \left( \frac{\Delta \rho}{\rho} \right) \cdot \frac{d^2}{\nu} \tag{11}
\]

In discussing the application of \( \sqrt{\frac{F_\Delta}{R_e}} \) criterion, Barr stated that a model study of internal movements resulting from small density differences would be expected to correspond closely with the prototype.
in all respects, if the prototype $\frac{FR}{\Delta e}$ number could be obtained in the model, providing that the model boundary was geometrically similar to that of the prototype and two main limitations were met.

These two limitations are:

1. $(\Delta \rho / \rho)_m$ can be considerably greater than $(\Delta \rho / \rho)_p$ only if the ratio of the densities of the two liquids do not differ significantly between model and prototype. This is because of the fixed relation between the gravity and inertia forces.

2. The introduction of a second liquid means that $\nu$ is a characteristic kinematic viscosity and

$$\left(\frac{\nu_a}{\nu_b}\right)_m \text{ must equal } \left(\frac{\nu_a}{\nu_b}\right)_p$$

Either $\nu_a$ or $\nu_b$ could be characteristic. Barr also stated that even equality of $\frac{FR}{\Delta e}$ numbers may not be necessary if the flow in both model and prototype were in the fully developed turbulence region.

Other authors have used similar criterion for modeling density current phenomena. A few of these are:

1. Fietz and Wood (1967) used both local and orifice or inlet Richardson and Reynolds numbers derived from dimensional analysis for their experimental three-dimensional density current study.

2. Wood (1967) used a densimetric Froude or Richardson number in his two-dimensional density current study.

3. In discussing the similarity of flow turbulence, Chen (1965) in a paper on the simulation of flow in cooling reservoirs, stated that the
model should be designed under Richardson's criterion:

\[ R_{i_r} = \left( \frac{V}{\sqrt{\Delta \rho / \rho_d}} \right)_r = 1 \quad \ldots \quad (12) \]

He also stated that similarity in the general flow pattern includes (1) the similarity of flow in the inflow channel and the flow into the reservoir, (2) the similarity in interchange between kinetic energy and potential energy of flow, (3) the similarity in buoyancy, and (4) the similarity in the tendency of forming a density current. The criteria thus required were:

\[ F_{r_r} = 1 \quad \ldots \quad (13a) \]

\[ R_{i_r} = 1 \quad \ldots \quad (13b) \]

\[ R_{e_m} > R_{e_{cr}} \quad \ldots \quad (13c) \]

\( R_{e_{cr}} \) denotes some critical value. In the case of \( (\Delta \rho / \rho) \) unequal in model and prototype, Equations (13a) and (13c) cannot be satisfied simultaneously; so Chen stated that it is better to give up \( F_{r_r} \) number as its scale effect is generally limited to the local inlet area and the similarity of the general flow pattern of the whole reservoir is not greatly affected. The third criterion for similarity is that the Reynolds number in the model should be greater than some critical value based on the flow pattern. Chen suggested dropping the length term and using the ratio \( Q/\nu \) to replace the Reynolds number. Therefore, the similarity rule becomes: \( (Q/\nu)_m > (Q/\nu)_{cr} \). Discharges above \( (Q/\nu)_{cr} \) will result
in the same general flow pattern. Chen suggested that \((Q/v)_{cr}\) be estimated from past experiments and checked in the model study. He also pointed out that \((Q/v)_r\) is governed by the model size, water depth, reservoir, topography, relative positions of the inlet and outlets, etc.

This chapter has consisted of a review of the general concepts and fundamental principles of waste stabilization ponds. Also, chemical reactor designing principles and their application to biological reactors such as sewage lagoons or aeration tanks have been discussed. Finally, the basic criteria for modeling stratified flow was presented.

The next chapter presents the important theoretical concepts that were an integral part of this study.
To be able to account exactly for nonideal flow requires knowledge of the complete flow pattern of the fluid within the tank or pond. In obtaining this knowledge, practical difficulties arise since it would require the complete velocity distribution picture of the fluid within the vessel. Therefore, an alternate approach was used requiring knowledge only of how long different elements of the fluid remain in the vessel. This partial information is rather easy to obtain experimentally and interpret. Although it will not completely define the nonideal flow pattern within the pond, this approach yields information which is sufficient in many cases to allow a satisfactory accounting of the actual existing flow pattern and to determine the conversion of waste in the pond.

The experimental technique used for finding this desired distribution of residence times of fluid in the pond is a stimulus-response technique using tracer material in the flowing fluid. The tracer is injected in the inlet at a known concentration and the response or concentration at the outlet is measured as a function of time.

The treatment will be limited to steady-state flow with one entering and leaving stream. From the concentration vs. time curve at the outlet,
C and F diagrams will be determined.

Several parameters can be defined from the C-diagram and F-diagram which give some measure as to the extent and effectiveness of mixing in the vessel. These parameters will be used to check quantitatively the effectiveness of various factors such as inlet and outlet positions, geometrical, and fluid properties of the pond. The C-diagrams will also be used to determine the expected conversion of the reactant for selected experiments.

Mean residence time

The mean residence time $\bar{\theta}_c$ is a measure of the average time the tracer slug spends in the vessel.

$$\bar{\theta}_c(\theta_o) = \frac{\int_0^{\theta_o} \frac{c}{c_o} \theta \, d\theta}{\int_0^{\theta_o} \frac{c}{c_o} \, d\theta} \tag{14}$$

The value of $\bar{\theta}_c(\theta_o)$ is the distance from the origin to the centroid of that portion of the C-diagram between the origin and $\theta_o$. The value of $\theta_o$ is arbitrarily taken as 2 because after two detention times, $\frac{c}{c_o}$ on the C-diagram is generally small and data taken beyond this point are near the limit of readability of the tracer sensing instruments. Furthermore, if all parameters are constructed using data for $0 < \theta < 2$ then the comparison of the results will be meaningful. The equation for mean residence time then becomes
Dead space

In any flow vessel there are generally regions where mixing is less active than desirable. Generally, this occurs in corners of the vessel. These regions of poor mixing will be called dead spaces if the fluid moving through these spaces takes 5 to 10 times as long to pass through the vessel as does the main flow. An indication of the amount of dead space in a flow vessel is indicated on the C-diagram by a long tail on the C-curve.

If the flow through the vessel has a minimum of dead space then the mean residence time $t_c$ will approach the detention time and $\bar{\theta}_c$ will approach 1.0. If there are substantial dead water regions in the flow then a large portion of the tracer will leave the vessel before $\theta = 1$. This will shift the centroid of the C-diagram toward the origin and make $\bar{\theta}_c < 1.0$.

To define the dead space a fictitious vessel will be described which will have no dead space and consequently will be smaller than the actual vessel. Its volume will be designated as $V_f$. This fictitious vessel will pass only the tracer which passed through the actual vessel up to $\theta = 2$ and this amount will be considered the total amount of tracer for the fictitious vessel. The flowrate through the fictitious vessel is $Q_f$ where

$$\bar{\theta}_c = \frac{\int_0^2 c c_o \theta \, d\theta}{\int_0^2 c c_o \, d\theta}$$

(15)
$Q_f = Q[F]_\theta = 2$

$Q_f$ may be considered that portion of the original flow rate which was moving fluid through the vessel at an acceptable rate.

The mean residence time of the fictitious vessel will be equal to that of the actual vessel and the detention time $\bar{t}_f$ in the fictitious vessel will equal the mean residence time $\bar{t}_c$. This means that if

$$\bar{\theta}_c = \frac{\bar{t}_c}{\bar{t}} = \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (16)$$

and

$$t = \frac{V}{Q} = \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (17)$$

$$\bar{t}_c = \frac{\bar{t}_f}{Q_f} = \frac{V_f}{Q_f} = \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (18)$$

then

$$\bar{\theta}_c = \frac{V_f Q}{V Q_f} = \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (19)$$

Now calling the dead space

$$V_d = V - V_f = \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (20)$$

one obtains

$$\frac{V_d}{V} = 1 - \bar{\theta}_c \{F\}_\theta = 2 \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (21)$$

If $V_d/V$ is defined as the dead space parameter $\bar{V}_d$ then

$$V_d = 1 - \bar{\theta}_c \{F\}_\theta = 2 \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (22)$$
**Deviation from plug flow**

It is recognized that the ideal situation for flow through a waste stabilization pond is typified by plug flow. If the incoming waste mixes vertically and horizontally just enough to obtain good treatment and then moves through the pond as a slug, the optimum condition, called plug flow, has been realized. That is, all the fluid will have remained in the pond long enough for the desired degree of treatment and none will have remained longer than necessary. This condition leads to a pond of minimum volume for a given waste load and hence, minimum cost.

The deviation of a given flow from plug flow can, therefore, be considered a measure of pond efficiency. It is the waste that leaves the pond before one detention time that contributes to poor pond efficiency. Furthermore, it is not only the amount of waste leaving too soon but the amount of time it lacks of remaining in the pond one detention time that is important. Consequently, the deviation from plug flow parameter \( \bar{\theta}_{pf} \) will be defined, as shown in Figure 4, as the distance from line \( \theta = 1 \) to the centroid of the area under the C-curve from \( 0 \leq \theta \leq 1 \). The equation used to calculate \( \bar{\theta}_{pf} \) is

\[
\bar{\theta}_{pf} = \frac{\int_0^1 (1 - \theta) \frac{c}{c_0} \, d\theta}{\int_0^1 \frac{c}{c_0} \, d\theta} \tag{23}
\]
Figure 4. The plug flow deviation parameter.
Computer program N-1 in the Appendix was developed to calculate the C-diagram, F-diagram, and the other important parameters discussed in this section. Each experiment was evaluated with these parameters to determine the most efficient design from a hydraulic standpoint. For best hydraulic performance that will result in maximum biological performance, the dimensionless mean residence time $\overline{\theta}_c$ should approach one. As $\overline{\theta}_c$ increases for given hydraulic conditions, the biological conversion would increase because the average detention time of the fluid particles would be greater.

Dead space has an obvious effect on conversion. Quantitatively, as dead space parameter $\overline{V_d}$ increases, the effective flow area decreases which results in a decreased detention time of the waste water and poorer biological performance.

The plug flow deviation parameter, $\overline{\theta}_{pf}$ will decrease in value as the hydraulic efficiency increases. Since maximum hydraulic efficiency and maximum conversion occurs when $\overline{\theta}_c = 1.0$ the same will hold true when $\overline{\theta}_{pf} = 0.0$.

The best flow situation that would result in an optimal detention time, and a minimum value for dead space and for the plug flow deviation parameter, would be plug or one-dimensional flow. However, in an actual pond it is important that the incoming flow mixes well at the outset to effectuate intimate contact of the waste material with the microorganisms in the pond. It is felt that for maximum conversion,
the incoming waste water should be diffused in such a way as to effectuate complete vertical and lateral mixing and then travel as a slug toward the outlet. This type of flow would utilize the complete cross-sectional area of the pond which would result in a minimum value for \( \overline{v}_d \) and \( \overline{\theta}_{pf} \) and an optimum value for \( \overline{\theta}_c \). This flow situation would then give maximum conversion for the given geometrical dimensions and hydraulic characteristics of the pond.

**Mathematical Approach**

**Conversion equations**

In a waste stabilization pond, the major concern is the extent of reaction of the waste material during its stay within the pond. For a reaction with a rate that is linear with concentration \( r_c = \frac{dN}{dt} = -KN \) the extent of reaction can be predicted solely from knowledge of the length of time each reactant element has spent in the reactor. The exact nature of the surrounding elements is of little importance. The distribution of residence times (C-diagram) gives information on how long various elements of fluid spend in a reactor, but not on the detailed exchange of matter within and between the elements. Because of this, the distribution of residence times yields sufficient information for the prediction of the average concentration in the reactor effluent. A waste stabilization pond is a biological reactor that closely corresponds to a first order (linear) chemical reactor, thereby supporting the use of chemical reactor design principles.
To determine the conversion of a chemical or biological reactor using the distribution of residence times, the following equation (Levenspiel and Bischoff, 1963) can be used:

\[
\text{Mean concentration of reactant leaving the reactor unreacted} = \sum_{\text{all elements of exit stream}} \left( \frac{\text{Concentration of reactant remaining in an element of age between } t \text{ and } t + dt}{\text{Fraction of exit stream which consists of elements of age between } t \text{ and } t + dt} \right)
\]

This equation says that for a steady flow if a sample was taken of the entire exit stream at some time and the extent of conversion of each waste or reactant element was determined and summed for all the elements in the exit stream or sample, the resulting quantity would be the mean concentration of the waste or reactant leaving the reactor unreacted. This same information can be obtained in a more convenient way by the following equation:

\[
\bar{N}_\infty = \int_{0}^{\infty} N \left( \frac{c}{c_0} \right) d\theta
\]

in which

- \( N \) = concentration of reactant as a function of time
- \( \bar{N}_\infty \) = mean concentration of reactant leaving the reactor in an unreacted state
- \( \frac{c}{c_0} \) = residence time distribution of fluid particles
Equation 24b assumes a slug injection of reactant or pollutant and thereby requires a summation over time to account for the conversion of all the pollutant elements injected into the reactor. Referring to the first order reaction equation

$$\frac{dN}{dt} = -KN$$  \hspace{1cm} (25)$$

in which $K =$ first order reaction coefficient. When $t = 0$, $N = N_o$ with $N_o =$ initial concentration of reactant. Integration of Equation 25 yields

$$N = N_o e^{-Kt}$$  \hspace{1cm} (26)$$

which is the well known equation that gives the remaining BOD of a waste as a function of $N_o$, $K$, and $t$. Incorporating Equation 26 into Equation 24 gives

$$\frac{N^\infty}{N_o} = \int_0^\infty e^{-Kt} \left( \frac{c}{c_o} \right) d\theta$$  \hspace{1cm} (27)$$

Equation 27 is the general equation for the conversion of a reactant in a chemical or biological reactor. The reliability of this equation for accurately determining the fraction of unreacted material leaving a reactor is dependent on (1) how closely the reactant or waste material follows a first order reaction, (2) the value for the first order reaction coefficient, and (3) the appropriate expression for the residence time distribution of fluid particles in the reactor.
In using experimentally determined \( c/c_0 \) functions, the data are generally unreliable beyond \( \theta = 2.0 \). In determining the extent of reaction of a waste, Equation 27 can be modified to

\[
\frac{N_2}{N_o} = \int_{0}^{2} e^{-Kt \theta} \left( \frac{c}{c_0} \right) \, d\theta \quad \ldots \quad (28)
\]

where experimental \( c/c_0 \) functions are used.

Equation 28 will give the fraction of material that has not undergone reaction at \( \theta = 2.0 \). To determine the extent of reaction or treatment efficiency, as a percent, \( \frac{N_2}{N_o} \) should be subtracted from one and the resulting quantity multiplied by 100.

Values for \( \frac{N_2}{N_o} \) and expected treatment efficiencies have been determined for a number of actual pond designs. These results are found in Chapter 5.

**Mathematical models for age distribution functions**

**Ideal flow models**

In using either Equation 27 or 28 it would be desirable to express the age distribution function \( c/c_0 \) as a mathematical function. For ideal fluid flow situations this has been done.

For completely mixed flow in a single reactor, the age distribution function is:

\[
\frac{c}{c_0} = e^{-\theta} \quad \ldots \quad (29)
\]
when incorporating this expression into Equation 29 the solution is:

\[
\frac{N_\infty}{N_0} = \frac{1}{1 + Kt} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (30)
\]

For a series of equal-sized completely mixed flow reactors, the age distribution function is:

\[
\frac{c}{c_0} = \frac{1}{(j-1)!} (\theta)^{j-1} e^{-\theta} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (31)
\]

where \( j \) = number of reactors. And the conversion equation is:

\[
\frac{N_\infty}{N_0} = \frac{1}{(1 + Kt)^j} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (32)
\]

For plug flow, the conversion equation is

\[
\frac{N_\infty}{N_0} = e^{-Kt} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (33)
\]

Equations 30 and 33 have been used quite extensively in the design of various wastewater treatment facilities.

Non-ideal flow models

In actual flow situations, the ideal flow conditions of plug, or completely mixed flow, are never obtained. It would therefore be desirable to develop an appropriate equation to describe the residence time distribution for use in Equation 27 or 28 which would make it possible to predict organic waste or other reactant conversion.
There have been a number of models developed and these are classified as (1) dispersed plug flow models, (2) tanks in series or mixing cell models, and (3) combined models.

Equation (5) in Chapter 1 is a mathematical expression of the dispersed plug flow situation. This equation has been used to some extent in proposed design methods for wastewater treatment facilities. Figure 5 from Levenspiel (1962) is a graphical representation of Equation (5). The ratio of reactor volume needed with dispersion to the plug-flow volume \( V/V_p \) is plotted against the fraction of reactant remaining at the outlet, with \( d \) (dispersion effect) as the varying parameter.

From this plot, it can be seen for a given degree of treatment or percentage reactant remaining, that as \( d \) increases, the actual volume of the reactor increases in relation to the volume of a plug flow reactor to obtain the same degree of treatment. In other words this means that the best flow situation would be one which most nearly approximates a plug flow model. A design of a waste stabilization pond where its residence time distribution approaches \( c/c_o \) for a plug flow model, (i.e. \( t_c \rightarrow t \theta pf \rightarrow 0.0 \) and dead space is a minimum) maximum conversion of organic matter would result.

It has been found that the only mathematical model that can adequately characterize or represent the age distribution functions in existing sewage lagoons is a combined model. A combined model is different than dispersed plug flow or mixing cell models in that it
Figure 5. Graphical representation of Equation 5—dispersed plug flow model (Levenspiel, 1962).
consists of interconnected flow regions with various modes of flow between and around these regions. A combined model consists of plug flow regions, completely mixed flow regions, dispersed plug-flow regions, and dead water regions.

The next section describes the combined model that was used in this research to represent the age distribution functions of waste stabilization ponds.

The finite stage or combined model

A combined model called a finite stage model has been developed by Hovorka (1961) and Adler et al. (1963). This model combines networks of perfectly mixed and plug flow stages. It is flexible and permits characterization of such major flow characteristics as partial mixing in the lateral and longitudinal directions, relative dead flow regions, and short circuiting. The basic module which is composed of a plug flow unit, a dead water unit, and a backmix (completely mixed) unit is shown in Figure 6. The dead water region is viewed to be in backmix flow and to be interchanging fluid slowly with the active completely mixed flow region.

To characterize a non-ideal flow situation, the basic module may be repeated any integral number of times, n. Figure 7a shows a typical flow situation and Figure 7b gives the finite stage model for this case.
Figure 6. The basic module of the finite stage model.

Figure 7. Finite stage model for a given flow situation.
Figure 7a shows that the flow system may be divided into two types of regions:

1. A live flow region shown as \( a_1, a_2, c_1, \) and \( c_2 \), where the bulk of the fluid flow occurs.

2. A series of dead flow regions shown as \( b_1 \) and \( b_2 \), which have a small amount of fluid interchange with the live flow region.

Each live-flow region has limited mixing in the longitudinal direction where the dead-flow region is well-mixed, but has no direct means of mixing with other dead regions.

Any physical system with any degree of longitudinal mixing may be approximated by a model containing a combination of live and dead-flow regions. This is the basis of the finite stage model developed by Hovorka.

The finite stage model requires four parameters:

\[
F_a = \text{fraction of the total system volume represented by perfectly mixed units in the live-flow region.}
\]

\[
F_b = \text{fraction of the total system volume contained in dead-flow regions.}
\]

\[
F_c = \text{fraction of the total system volume contained in plug-flow elements. It also represents the delay time or time when tracer first appears at the vessel exit.}
\]

\[
K_H = \text{the fraction of the main flow which is interchanging between live and dead-flow regions.}
\]
\[ n = \text{the number of basic modules in series.} \]

Physically, the parameters of the model must be subject to the following restrictions:

\[
0 \leq F_a \leq 1 \\
0 \leq F_b < 1 \\
0 \leq F_c \leq 1 \\
0 < F_a + F_b \leq 1 \\
0 \leq K_H \\
n = 1, 2, 3 \ldots \text{(Integral values only)}
\]

\[ F_a + F_b + F_c = 1 \]

The live-flow region can approximate any level of longitudinal mixing from one extreme of no mixing to the other extreme of perfect mixing depending upon the relative size of \( F_a \) and \( F_c \). Varying the value of \( K_H \) can also cause various levels of mixing. The value of \( n \) effects the longitudinal mixing length, small \( n \) corresponding to long mixing lengths. The value of \( K_H \) also effects the amount of inter-change between dead-flow and live-flow regions.

**Parameter evaluation and model solution**

Finite stage models and their parameters have definite physical significance, and it would be desirable to predict these parameters from a knowledge of the flow rate, fluid properties, and geometry within the system. At the present there have not been ways developed to predict these parameters. The best alternative method is to use
experimental residence time distribution curves from existing systems or from smaller physical models.

The following is the procedure developed by Hovorka for determining the model parameters. It is based on fitting an analytic equation to the residence time distribution curve determined from tracer data. Because the tail of the curves is generally unreliable, the procedure is designed to rely basically on the front part of the concentration versus time curves.

The exact fit is assured at two points, the minimum residence time \( F_c \), where the tracer first appears, and the point at which the tracer concentration is a maximum.

The transformation equations to convert the curve in Figure 8a to the curve in Figure 8b are:

\[
Y = \frac{c}{c_0} (1 - F_c) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (34a)
\]

\[
X = (\theta - F_c)/(1 - F_c) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (34b)
\]

The initial steps of the procedure are illustrated in Figure 8. The curve in Figure 8a is the normalized C-diagram for the concentration versus time data of the injected slug of tracer. The dead or delay time, equal to \( F_c \), is removed using the transformation equations shown with Figure 8, thus reducing the data to \( Y \) vs. \( X \) coordinates shown as the curve in Figure 8b. A residence time equation without dead
Figure 8. Transformation of C-diagram to X and Y coordinates.

\[ Y = \frac{c}{c_0}(1 - F_c) \]
\[ X = \frac{\theta - F_c}{1 - F_c} \]
time is then fitted to the data. Equation (35) was determined by
Hovorka by writing material balance total differential equations about
the modules and solving these equations by Laplace transform tech-
niques. The derivation of this equation is shown in Appendix C of
Hovorka's dissertation.

\[ Y = n \sum_{m=1}^{n} \left[ A_{nm} (nX)^{m-1} e^{nr_1X} + B_{nm} (nX)^{m-1} e^{nr_2X} \right] \]

Equation (35)

The A's, B's, r₁, and r₂ are constants which are functions of
the model parameters K, n, Fₐ, and Fᵦ. These algebraic functions
and the expansion of Equation 35 for n = 1, 2, 3, 4, and 5 are listed in
Appendix C of Hovorka's dissertation.

Hovorka prepared special charts and tables to permit selection
of Kₜ and f₂, where f₂ = F/ (1-F), for each permissible value of
n in order to force Equation 35 to have the correct maximum points.

The following procedure illustrates the method.

1. The peak time and value of the peak concentration are trans-
formed to \( X_{pk} \) and \( Y_{pk} \) by the transformation Equations 34a and 34b.

2. A graph is consulted which gives a value of \( f₂ \) and \( Kₜ \) for
\( X_{pk} \) and \( Y_{pk} \) for each n.

3. The best n is chosen so as to minimize the average squared
deviation between the Y values of the data and of Equation 35.
4. Having a value of \( n, K_{H'}, F_a, \) and \( F_b, \) the coefficients of Equation 35 can be determined resulting in the transformed finite stage model.

5. The values of \( X \) and \( Y \) of the model equation are then transformed back to conform to the original C-diagram by the following equations:

\[
\frac{c}{c_0} = X(1-F_c) + F_c \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (36a)
\]

\[
\theta = \frac{Y}{(1-F_c)} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (36b)
\]

In summary, an approximate fit over the entire residence time distribution curve for a flow system such as a waste stabilization pond may be made with two pieces of information. This information includes:

1. The \((C, \theta)\) coordinates where the tracer first appears.

2. The \((C, \theta)\) coordinates at the point where the tracer concentration is a maximum.

The value of this model is readily recognizable. If a residence time distribution curve could be specified by use of a finite stage model for a proposed waste stabilization pond, then the extent of reaction or treatment efficiency could be determined by Equations 27 or 28 provided a reliable value of the first order reaction coefficient is known.
Purpose of Model Study

In general, there are three different types of flow conditions that exist in waste stabilization ponds which must be considered when designing a model as mentioned in the previous chapter.

The flow conditions that generally occur in the Logan Pond System (with some exceptions) are these three conditions. The differences in densities are due, at least in the Logan Pond System, to temperature variations in the lagoons as well as to temperature variations in the incoming flow. To correctly model these types of flow conditions, the model probably should use temperature as a means of creating different density conditions in the pond and incoming fluid. To do this the model would have to be insulated and the room temperature and humidity controlled. For a model of the size used in this study, this was not practical. Instead, a commercial salt (NaCl) was used to obtain the required density differences needed to simulate the prototype pond flows.

The question arises as to whether this technique is valid to simulate a basically convective and conductive heat and mass transfer phenomena. An inspection of the actual flow conditions in the prototype stabilization pond will reveal that the flows are essentially turbulent in nature where the eddy diffusivity is the most important mechanism of either heat or mass transfer. From turbulent flow characteristics, it is known that 

\[ v << \frac{E_m}{h} \text{ and } a << \frac{E_h}{h}, \]
in which
\[
\nu = \text{kinematic viscosity or diffusivity of momentum}
\]
\[
E_m = \text{eddy diffusivity of momentum}
\]
\[
a = \text{thermal diffusivity}
\]
\[
E_h = \text{eddy diffusivity of heat transfer}.
\]

This means that heat transfer due to conduction is negligible in comparison to convective heat transfer. The Prandtl number for water is \( Pr = \frac{\nu}{a} = 1.5 \), which means that \( a < \nu \).

Since \( \nu << E_m \), \( a << E_h \), and from \( Pr = 1.5 \), \( a < \nu \). Now the question becomes, can the molecular diffusion in the model (using salinity to create density stratification) be neglected when compared with convective mass transfer?

For a binary mixture of a low concentration of salt, the Schmidt number, which is the ratio of the diffusivity of momentum to the molecular mass diffusion coefficient is:

\[
S_c = \frac{\nu}{D_{\text{mass}}} \approx 750 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (37)
\]

so, \( D_{\text{mass}} << \nu << (E_h or E_m) \). Therefore the importance of mass diffusion by conduction is even less than heat transfer by conduction.

Since heat transfer by conduction can be neglected in the prototype, mass diffusion by conduction can also be neglected in the model. It appears that the use of salinity will work well in simulating density stratification in the model.
Another problem arises in using sodium chloride or any other salt in the model to simulate density stratification. In the prototype waste stabilization pond, the major mode of heat transfer to the ambient fluid is not from heat conduction or convection from the incoming fluid, but from solar radiation heat transfer. The heat transfer from radiation depends upon many factors of which the most important is the environmental conditions. During the summer months, the net effect of solar radiation is to increase the temperature of the ambient fluid thereby reducing the density.

In modeling the flow conditions during this period of time with salinity, the net effect is to increase the density of the model pond by salt accumulation thereby reducing the density difference between the incoming flow and the ambient fluid. During the summer months' operation of the prototype ponds, the density difference actually increases. Even though this effect is wrong for correct modeling, it is assumed that it does not have a great effect on the overall flow conditions. A mathematical check was made to determine what influence an increase in salinity of the ambient fluid would have on the density difference between incoming salt water and the ambient fluid. In a conservative sense it was assumed that the pond was completely mixed with the incoming salt solution which would result in greater salinity build up and a more rapid decrease in the density difference. The change in density difference for the hypothetical flow situation was about 30 percent more than it should
have been when compared with the prototype pond data for a 30 day period of time.

In summary, the overall purpose of this research on waste stabilization ponds was to determine the existing flow patterns in the prototype ponds and to determine sound scientific design principles and methods to better design lagoons, from a hydraulic standpoint, to increase the effectiveness of this method of waste treatment. The method of analyzing pond behavior was to conduct tracer studies. Since this was expensive and time consuming, a hydraulic model was designed and used to aid in this research.

The purposes of the hydraulic model study were to:

1. Verify existing pond behavior.
2. Determine effects of geometry on the flow behavior.
3. Determine effects of the flowrate and subsequent Reynolds number on pond behavior.
4. Determine effects of different densities on the flow patterns and subsequent mixing.
5. Determine effects of inlet and outlet placement and design on the flow patterns.

There are other factors which are important and have an influence on the hydraulic characteristics of stabilization ponds such as, temperature, humidity, evaporation, precipitation, and wind. These environmental conditions were ignored. Wind is probably the most important of
these environmental conditions and the wind effects are being studied in a model at the present time by another Ph.D. candidate.

The flow patterns in waste stabilization ponds are complex and difficult--probably impossible to determine mathematically since the flow is really three-dimensional in nature. The three-dimensional velocity field must be determined before the three-dimensional dispersion equation can be solved. Consequently, hydraulic model studies to determine the general flow behavior are the most practical approach in studying the flow patterns.
CHAPTER 3

LOGAN POND SYSTEM STUDIES

Introduction

The purpose of this phase of the research was to determine the characteristics of the circulation patterns in waste stabilization ponds and to collect data on several of the existing ponds in the Logan Pond System (Figures 9 and 10). These data are compared with pond model data to ascertain the extent and influence of short circuiting, dead or stagnant areas, density difference effects, and to determine the influence of geometry, inlet and outlet types and locations on the flow patterns and hydraulic efficiency of the ponds.

The circulation patterns were determined by introducing a dye into the inlet flow and determining how rapidly and efficiently the flow disperses to all areas of the pond. By sampling at the exit, time of travel through the pond was established to determine if the requirements on the time of retention are being met. In addition, the sampling procedures were evaluated so that any corrective measures necessary could be taken.

Experimental Preparations

Experiments were performed on ponds A-1, A-2, D, and E, shown in Figure 9 as a plan of the Logan Pond System. Figure 9 also shows
Figure 9. The Logan pond system.
Figure 10. Photograph of Logan pond system.

Figure 11. Photograph of pontoon boat and sampling equipment used on the Logan City ponds.
the points where samples were taken in addition to the outlet samples of each pond. These sample points were established by using concrete anchors and small buoys. The grid size was chosen to give a good representation of the pond geometry, yet small enough (15 grid points) to allow the samples to be taken in a reasonable length of time.

The tracing dye, rhodamine WT, was ordered from DuPont in the form of a 20 percent solution. This dye was chosen because it is relatively nonabsorbent and inexpensive. A Turner Model 110 fluorometer, was calibrated to measure the dye concentrations. To avoid temperature sensitivity problems, it was decided that all dye samples from the pond would be brought to calibration temperature before readings were taken.

During the summer of 1969 a pontoon boat, Figure 11, was constructed along with the equipment necessary to obtain samples of the pond at a specified depth and at any location on the pond.

**Experimental Procedure**

The dye slug was introduced into a selected pond through the submerged culvert which supplies each pond. This was accomplished by inserting into the culvert a long plastic tube connected to a 30-gallon tank containing the dye-water solution. Thorough flushing of the tank and tube with water insured that the complete sample entered the pond.

At varying time intervals, depending on the dye movement, samples were taken at each of the grid points, as well as at the outlet of the pond. In some cases samples were taken of the top, middle, and bottom at
each of the grid points in addition to the outlet sample. The temperatures of the samples were measured periodically to determine if stratification was present. The sampling procedure for the entire pond took about 60 minutes to complete if samples were taken at the top, middle, and bottom of the pond. Any variation in this experimental procedure with a particular experiment is noted in describing the results of that experiment. Also the dye cloud positions were observed and noted as a function of time.

Results of Experiments Performed

Experiment No. 1

Dye was injected into the inlet of Pond E at 4:30 p.m., November 23, 1968, and the first samples were taken at 4:15 p.m., November 24, 24 hours later. Instead of the dye being concentrated near the upstream end of the pond, it was found only in the downstream one-third of the pond. The sample taken at the exit showed a strong dye concentration. Since the retention time for the pond was nearly 6 days, it was apparent that something was wrong drastically.

At the end of 48 hours, the second and third set of sample stations from the inlet towards the south (Figure 9) exhibited fluorescence. However, the concentration was only about one-half that at the fourth and fifth set of sample stations from the inlet.

At the end of 72 hours the concentration throughout the pond was essentially uniform, indicating that uniform dispersion had taken
place. But, by this time, about one-half of the dye had already left the pond.

The results show that serious short-circuiting of the pond is occurring. However, since samples were not taken the first 24 hours of operation, it was not known exactly how the dye stream progressed through the pond, therefore the results are inconclusive.

Experiment No. 2

In February Pond E was again injected with dye. The ponds were mostly ice-covered at this time so all that could be done was inject dye at the pond inlet and collect samples at the pond exit. There was a small ice-free area at both of these locations. The dye was injected into the inlet at 8:00 a.m., February 15, 1969.

After dye injections, the dye stream was observed to form into a flow streaming toward the exit. Four hours after injection a visible dye stream was flowing out of the exit, and it was coming straight from the inlet. This was good visible proof of serious short-circuiting.

The highest concentration of dye passed through the exit some 6-10 hours after injection (Figure 12). This indicates an average velocity of the dye slug of about 0.05 feet per second through the pond.

In this test the dye load was twice that of test No. 1 to insure more accurate readings. This change, coupled with more frequent samples, has substantiated the tentative conclusions of test No. 1.
Figure 12. Concentration vs. time curve at outlet for Experiment 2 - Pond E.
Experiment No. 3

On October 23, 1969, 12 liters of rhodamine WT 20 percent solution were placed into the transfer structure between ponds "C" and "D." Concentrated slugs of dye appeared on the surface in an erratic, random pattern.

Top, mid-depth, and bottom samples were collected at each grid point at various time intervals, varying from 2 to 12 hours, and the dye concentration determined. Also, samples were collected at the outlet of the pond at various time intervals. The dye cloud moved rapidly through the pond as detailed below.

By 12:00 noon, October 23, the dye cloud had spread across the north end of the pond. The movement of the visible dye cloud is shown in Figure 13. By 6:00 p.m., 8 hours after release, the dye had traveled more than half way through the pond. The exact time when the dye first began to leave the pond could not be visually determined because of darkness. Fluorescence was not detected in the samples taken at the outlet at 10:00 p.m., October 23, but readings were obtained on outlet samples taken at 6:00 a.m., October 24.

Various curves were obtained from the data collected. Figure 14 shows the concentration (mg/L) of dye vs. the time at the outlet. The peak concentration at the outlet was attained 28 hours after the dye had been released into the pond. The dye concentration dropped off quickly, then steadily decreased to only trace amounts. A more informative
Figure 13. Dye clouds observed from boat at various time (hrs) after dye injection experiment.
Figure 14. Concentration vs. time curve at outlet for Experiment 3 - Pond D.
picture is presented by the mass diagram in Figure 15. One-half of the dye placed in the pond had reached the outlet 120 hours after the dye had been released. The retention time for this flow rate is about 250 hours, so it is apparent that the waste water is passing through too quickly. Samples were last taken 340 hours after release and 88 percent of the dye had left the pond. Figure 16 shows the average concentration of all grid points in the ponds. The concentration at the bottom was initially lower than that at the top, due to the lower velocities along the bottom. The pond became fairly well mixed 33 hours after the dye had been released. By this time about 11 percent of the dye had left the pond. After this time the dye was quite evenly dispersed throughout the pond. Most of the fluorometer readings indicated that the concentration of the dye was decreasing fairly evenly throughout the pond. Eventually a region of clear water developed at the north end of the pond and the dyed fluid slowly moved toward the outlet.

Calculations based on the average flow rate indicated a design velocity of .0026 fps. However, based on the time for the dye to reach the outlet, the velocity was .035 fps which is an order of magnitude greater. The maximum flow velocity was higher than the average velocity by a factor of approximately 1.5 to 2.0.

There seemed to exist a certain amount of vertical stratification as the dye cloud moved through the pond. Low readings were obtained on the bottom, but roughly equal readings were obtained on the surface
Figure 15. Mass curve for Experiment 3 - Pond D.

\[ M_t = 2.4 \times 10^6 \text{ mg} \]
Figure 16. Average concentration of all grid points vs. time curves for the top, middle, and bottom of Pond D - Experiment 3.
and at the 4 foot depth. Temperature measurements indicated the bottom was 1°C cooler than the top of the pond. After the edge of the dye cloud had passed, nearly equal readings were obtained throughout the depth. Thirty-three hours after the release the dye had become evenly dispersed. The pond decreased in fluorescence evenly until about 110 hours after release.

**Experiment No. 4**

On November 20, 1969, at 7:00 a.m., 12 liters of rhodamine dye were injected into the 30" pipe that runs into pond "D." Shortly after all the dye had been placed in the pipe, it appeared at the outlet coming to the surface as slugs of dye. As time went on, it spread out and moved laterally across the north end of the pond towards the west bank. The dye first appeared at the outlet between 30-33 hours after the dye was introduced into the pond.

At 10:00 a.m., 1:00 p.m., and 4:00 p.m. on November 20 aerial photographs were taken of the pond to record the dye movement. At 2:00 p.m. on November 21, 1969, aerial photographs were again taken of the dye movement in the pond. These photographs were reduced to sketches, and Figure 17 shows the position of the dye cloud at each time photographs were taken. For this experiment, samples were taken only at the outlet and not at the sample points throughout the pond. The concentration versus time curve for this data is shown in Figure 18.
Figure 17. Dye clouds as observed from the air at various times after dye injection for Experiment 4 - Pond D.
Figure 18. Concentration vs. time curve at outlet for Experiment 4 - Pond D.
Figure 19 shows the accumulated mass curve versus the time following the introduction of dye. This curve represents the quantity of dye that has left the pond as a function of time.

At 2:30 p.m. on November 20, 1969, a longitudinal traverse was made through the front of the advancing dye cloud. Samples were taken at 6 different points as shown in Figure 20 to determine concentration profile of the dye as a function of depth. Figure 20 shows the concentration profiles at the 6 sample points.

In the front portion of the dye cloud the concentration profile was similar to the actual velocity profile as measured with drogues. After moving into the dye cloud, the concentration became more uniform with depth because of the vertical convective and conductive mass diffusion of the dye from the region of greater mass concentration near the surface to the regions of lower concentration near the bottom of the pond.

Four drogues were placed on the I-3 side of I-2 (Figure 20) to determine the approximate velocity profile by time displacement. It was assumed that the bulk fluid movement would be in the direction of the outlet. It was surprising to find that the movement was not toward the outlet as was thought, but was circular in motion and moving in the general direction of H-2. The flow near the west bank of the pond was in the general direction of the outlet while on the east bank it was in the direction of the inlet. These visual observations indicate a counterclockwise circular flow pattern apparently caused by the jet type inflow
Figure 19. Mass vs. time curve for Experiment 4 - Pond D.

\[ M_t = 2.4 \times 10^7 \text{ mg} \]
Figure 20. Dye concentration profiles as a function of depth for Experiment 4 - Pond D.
from the 30" diameter inlet pipe.

This experiment strengthened the conclusions of the previous experiments, which indicated "short-circuiting" in the waste stabilization ponds. The aerial photographs, referred to above, show the gross movement of the fluid directly towards the outlet. There is some lateral mixing, but as shown in the pictures, some areas of the pond were void of high concentrations of dye. The concentration versus time curves, Figures 14 and 18 indicate a circular flow pattern for Experiments 3 and 4. The series of mild peaks shown in Figure 14 is a result of a quantity of tracer that is continually being circulated and diffused so that its concentration is decreased. But each time around, a quantity of this tracer leaves the pond at the outlet which results in the peaks shown in Figure 14. Figure 14 indicates a more serious problem than does Figure 18. The peak concentration was higher for Experiment No. 3 than for Experiment No. 4. This can be explained by the Reynolds number variation of the inlet flow pattern. The Reynolds number in Experiment No. 3 was about twice that of Experiment No. 4. This is due mainly to the increased flowrate, which causes a strong circulation pattern to be developed. This results in a more serious short circuiting problem as is evidenced by the higher concentration of dye at the outlet, as is shown in Figure 14.
Experiment No. 5

On May 19, 1970, at 2:00 p.m., 44 liters of rhodamine WT 20 percent solution were placed in the inlet channel just ahead of the Parshall flume that measures the total wastewater flow before it enters ponds A-1 and A-2. After passing through the Parshall flume, the wastewater flow enters a splitter box that equally divides the flow into the primary ponds A-1 and A-2. It was assumed for the purposes of this experiment that the flow was equally divided (22 liters) into each pond.

Samples at the outlets of both ponds were collected at varying time intervals. Also, samples were collected at the grid points of pond A-1 as shown in Figure 9 at various time intervals.

It is not known when the dye first appeared at the outlets of both ponds, but 7 hours after the dye placement, the dye had been leaving pond A-1 for some time as the concentration had already reached its peak. Six days after the dye was first introduced into the ponds, the sampling was discontinued due to an unexpected change in the flow conditions caused by an operational change made by Logan City personnel. At this time, the dye concentration of the fluid leaving pond A-2 had not reached its peak. In brief, there was a serious short circuiting problem in A-1 but it wasn't nearly so severe in A-2.

The probable reasons for this different flow behavior are: on the day that the dye was placed into the pond, special note was made of a strong wind blowing from the southwest to the northeast during the whole
day. By referring to Figure 9 of the Logan pond system, the wind movement in relation to the layout of the inlets and outlets at ponds A-1 and A-2 can be seen.

It is felt that this consistently strong wind from the southwest caused a vertical circulation pattern to be developed in the ponds with the surface fluid being dragged along by the wind in a northeasterly direction and the fluid in the lower depths of the pond moved in a southwesterly direction. Since the waste water entering the ponds was of a higher density, due to the lower temperature than the receiving fluid, it tended to stay near the bottom and was carried directly to the outlet of pond A-1 by the bulk fluid movement in the lower depths. This resulted in a large quantity of dye being carried through the pond via the short circuit route. Pond A-2 did not reflect this short circuit problem because of the placement of the outlet (Figure 9). The dye, due to this vertical circulation pattern, tended to collect in the southwest corner of the pond and since the outlet was located in the northwest corner, the problem of short circuiting was much less severe in pond A-2.

This shortened experiment did point out the need for considering outlet and inlet placement in relation to prevailing winds in order to prevent serious short circuiting and subsequent loss of hydraulic efficiency of waste stabilization ponds.
Experiment No. 6

On August 5, 1970, 19.57 liters of rhodamine WT 20 percent solution were placed into the inlet pipes of primary pond A-1. On the same date and time 6 liters of the same dye were placed into the inlet pipes of primary pond A-2.

Samples at the outlets of both ponds were collected at varying time intervals. Also, samples were taken at the grid points shown in Figure 9 at various time intervals. Temperature readings were taken at both inlets and outlets of the two ponds to determine the existing density difference between the ponds and the influent to the ponds.

Dye first appeared at the outlet of pond A-1 somewhere between 5 and 24 hours after dye placement in pond A-2. Dye didn't appear until 28 hours after dye placement in pond A-2. The peak concentration in both ponds A-1 and A-2 occurred 56 hours after dye placement. Figure 21 shows the normalized concentration vs. time curves for these two ponds. The quantities $c_o$ and $t$ are defined as:

$$c_o = \frac{\text{mass of dye placed in pond}}{\text{volume of pond}}$$

$$t = \frac{\text{volume of pond}}{\text{flow rate}}$$

Samples taken at the grid points of pond A-1 indicated the location or movement of the dye. The data revealed that the dye seemed to stay near the bottom especially near the diffuser. This is as expected since
Figure 21. Normalized concentration vs. time curves at outlets, uncorrected for Experiment 6 - Ponds A-1 and A-2.
the density of the influent was greater than in the pond. After 72 hours
the dye concentration was essentially uniform.

Figure 23 is the mass diagram for ponds A-1 and A-2. At the
time (72 hours) that the dye became thoroughly mixed in pond A-1, about
6 percent of the dye had left the pond. After one theoretical detention
time, 37 percent of the dye had left the pond. This percentage, when
added to the percentage of dye left in the pond, should have been equal
to the amount of dye placed in the pond. For pond A-1 only about 54
percent of the total quantity of dye was accounted for. For pond A-2,
after one theoretical detention time, 77 percent of the dye was accounted
for. The data for the experiments on ponds A-1 and A-2 were corrected
for this loss of dye by reducing the quantity of dye placed in the ponds by
the amount that was determined to be lost. The corrected curves for
ponds A-1 and A-2 are shown in Figures 22 and 23. This is an approxi-
mation, but it was felt that the results are more realistic when compared
with this reduced mass of injected tracer than with the amount originally
placed in the ponds.

In an attempt to locate the missing dye, samples were taken of the
sludge on the pond bottom after the dye concentration had reached zero
at the outlet. Samples of water were also taken just above the bottom at
several locations throughout the pond. The sludge samples were mixed
thoroughly and the solids centrifuged out. The water samples near the
bottom indicated a zero concentration of dye. The dye concentration of
the fluid remaining from the centrifuged sludge samples indicated an
Figure 22. Normalized concentration vs. time curves at outlets, corrected for Experiment 6 - Ponds A-1 and A-2.
Figure 23. Normalized mass diagrams for Experiment 6, corrected and uncorrected for Ponds A-1 and A-2.
average of 0.001 mg/L for three samples. This means that the dye was either in the sludge or in the wastewater located in the pond bottom depressions. Another test was conducted to determine if the dye had been adsorbed on the sludge. Samples of sludge were taken at several locations throughout the pond and the solids centrifuged out. The solids were then mixed with water containing a known concentration of dye. After two weeks the concentration was measured and compared with the original concentration. The samples indicated no change in the dye concentration. Apparently there was little or no adsorption of the dye on the organic matter in the pond. The only explanation for the loss of dye must be that a certain quantity of dye, mixed with the more dense liquid entering the pond became entrapped in small depressions or pockets in the boundary layer on the bottom of the pond and thus was a lost quantity.

There did exist in pond A-1 a definite vertical dye stratification as the dye moved through the pond. High readings were generally found near the bottom. This was because the fluid entering the pond was more dense than the pond fluid itself, and because of the vertical temperature profile that existed in the pond. Temperatures varied from 1°C to 10°C cooler at the bottom than at the top. After 72 hours the dye concentration in the pond became essentially uniform.
Experiment No. 7

On November 17, 1970, 9,475 liters of rhodamine WT 20 percent dye were placed into the transfer pipes between ponds D and E. The temperatures at the inlet and outlet of the pond were essentially constant throughout the test run.

This experiment was performed to get better performance data for the particular type of inlet and outlet facilities that exist in pond E. The inlet is two 30" pipes and the outlet is one 36" pipe located in plan as shown in Figure 9. The inlet and outlet pipes are on the bottom of the pond. Samples were taken hourly at the outlet for 24 hours, then at varying time intervals after that.

The dye first appeared 5 hours after placement into the pond. The peak concentration was reached 11 hours after the start of the experiment. Two curves were obtained from the data collected. Figure 24 is the normalized concentration vs. time curve at the outlet and Figure 25 is the normalized mass curve. The dye seemed to stay near the west bank after entering the pond, and it occupied essentially the west half until reaching the outlet. The tracer was carried in a counter-clockwise direction to the east bank. Figure 24 shows the typical peaks that would be expected due to the counter-clockwise circulation patterns developed in the pond.

The ponds in the Logan pond system that were studied all experienced short circuiting to some degree depending upon the environmental
Figure 24. Normalized concentration vs. time curve at outlet for Experiment 7 - Pond E.
Figure 25. Normalized mass curve for Experiment 7 - Pond E.
conditions, the hydraulic characteristics, and the geometry of the pond.

Reliable data were difficult to obtain in the field studies because of (1) inaccurate flowrate measurements, (2) the location of the sampling points, and (3) temperature variations.

Tied closely to flowrate measurements is the unknown quantity of seepage and evaporation and its effect on the flowrate out of each pond. Flowrate measurements were only made at the inlet of ponds A-1 and A-2 and at the outlet of pond E. It was assumed that the flowrate out of pond A-1 and A-2 was equal to the flowrate into these ponds. Similar assumptions were made on ponds D and E. This, of course, neglects seepage and evaporation losses. The effect of the location of the sampling point at the outlet on the reliability of the results is shown in Figure 18. Temperature was constantly changing which made it difficult to obtain representative measurements. Temperature was particularly important in Experiments 5 and 6.

Considering all of the possibilities for experimental error and its influence on the data and other factors that affect tracer experiments, it is felt that the results reported herein do give reliable estimates of pond behavior, and show the influence of certain design factors on the flow patterns through waste stabilization ponds.
CHAPTER 4

EXPERIMENTAL PROCEDURES-MODEL

Description of Experimental Equipment

Experimental apparatus

The hydraulic model was constructed with plywood coated on the inside with fiberglass and resin. The facility is shown schematically in Figure 26 and photographically in Figure 27. The tank is 40 feet long, 20 feet wide, and 3.5 feet deep. The facility was designed so that the tracer concentration at the outlet could be continuously monitored with a Turner Fluorometer. The effluent of the model was carried by a 2 1/2" diameter pipe. Samples of the outflow were pumped at a constant rate through the fluorometer from a sample tap in the pipe. The fluorometer dial reading was recorded with a strip chart recorder. The flowrate at the outlet was measured with a Venturi meter and controlled with a gate valve. Figure 28 shows the outlet apparatus and tracer concentration measuring equipment.

The inflow apparatus was designed so that the flowrate could be continuously recorded and the salt water flow could be adjusted to give the required density for the density experiments. The inflow apparatus is shown in Figure 29. The apparatus consisted of two tanks, one for fresh water, and the other for a salt water solution. The fresh water
Figure 26. Experimental apparatus and model pond.
Figure 27. Photograph of hydraulic model.

Figure 28. Photograph of outflow apparatus and tracer concentration measuring equipment.
Figure 29. Photograph of inflow apparatus.

Figure 30. Photograph of diffusers used in experimental studies.
was pumped to a constant level tank and then mixed with a metered flow of salt water to give the required density when making density experiments. A booster pump was then used to give the required flowrate, which was carried by a 2 inch diameter pipe.

The model pond was equipped with a motorized carriage that spanned the width to enable samples to be taken at any location in the pond.

An extra wall was constructed along with baffles and a number of inlet and outlet devices. The inlet devices were constructed so as to break up or diffuse the incoming flow to nullify the effects of jets on the flow patterns. This was accomplished by diffusers made out of plastic pipe and graded gravel. The outflow was mixed so as to give good representative sample before being pumped through the fluorometer.

Diffusers used in experiments

Three types of diffusers were used in the experiments. Diffuser A in Figure 30 was designed and constructed to represent the inflow behavior of Pond A-1 in the Logan pond system. It was the same length as the diffuser in Pond A-1 (reduced by the length ratio between model and prototype) that being 8 feet long. Diffuser B in Figure 30 was 2 feet long and was constructed like diffuser A. Diffuser C was a vertical diffuser 2 feet in height which was designed to break up and distribute the flow uniformly around the diffuser. Diffuser C was constructed by drilling a large number of holes around the two foot length of plastic pipe and then placing the pipe in a cylindrical screen which was 2 inches
greater in diameter than the pipe. This annular space was then filled with graded gravel.

Experimental Method and Design

Test procedure

1. Tank was filled to desired depth with river water and allowed to become quiescent to enable the eddies and currents to die out.

2. Inlet and outlet flowrate was adjusted to desired quantity.

3. The fluorometer was turned on.

4. Saltwater was injected at desired rate to give desired density of incoming flow. (For constant density experiments, this step was omitted.)

5. The model was operated for a certain time to allow for the flow to stabilize.

6. A known quantity of rhodamine WT 20 percent dye was injected into the inlet essentially as a slug. The recorder on the fluorometer was started at this same time.

7. The experiment was terminated when two theoretical detention times were reached.

8. The temperature of the fluid at the outlet was measured at different times during the experiment.
**Data analysis and design of experiments**

The raw data from the strip charts were extracted at equal intervals of time and the concentrations of the tracer were determined from temperature dependent calibration charts. The data were processed using computer program N-1 to calculate dimensionless C and F-diagrams and other pertinent parameters. The computer program used a simple numerical method for making the required calculations.

The following are the different groups of experiments that were performed.

1. Identically designed experiments to determine reproducibility.

These experiments all had the following dimensions and hydraulic characteristics.

\[
\begin{align*}
\text{Length} & = 40 \text{ feet} \\
\text{Width} & = 20 \text{ feet} \\
\text{Depth} & = 1.5 \text{ feet} \\
Q & = 80 \text{ gpm}
\end{align*}
\]

The 8 foot diffuser was centered in the model 8 feet from the end wall. The outlet consisted of one \(2 \frac{1}{2}\) inch diameter pipe located one foot from the floor of the model and one foot from the side wall.

2. Reynolds number variations - The model was designed the same as number 1 above except that \(Q\) was varied for the purpose of determining the effect of the Reynolds number on the C-diagram and pertinent parameters.
3. Depth variations - The model was designed the same as number 1 above except the depth was varied for the purpose of determining the effect of depth on the flow patterns.

4. Outlet and inlet variations - The model was designed as number 1 above except the inlet and outlet devices were varied as well as $Q$ in some instances for the purpose of evaluating inlets and outlets in terms of the hydraulic efficiency.

5. Length to width ratio variations - The 8 foot diffuser was placed against the end wall centered between the two side walls of the model pond. The length was varied for most of the experiments with the proper placement of an extra wall with the width held constant. When baffling was used, both the width and the length was changed to give large $L/W$ ratios. The depth was 1.5 feet and $Q = 80$ gpm generally. The purpose here was to determine the effect of the $L/W$ ratio on the hydraulic efficiency.

6. Verification experiments for comparing with existing ponds - These model experiments were designed according to the proper model laws and selected geometrical ratios. The purpose of these experiments was to verify existing pond behavior.

7. Density stratified flow experiments - The model was designed as the experiments in number 1 except the pond was either filled with a saltwater of predetermined density or the inflow was saltwater of appropriate density to give the desired densimetric Froude number.
desired density of the inflow to the model pond was obtained by mixing saltwater, containing a high concentration of salt, with the incoming fresh water in the proper quantities to give the inflow the desired density. The highly concentrated saltwater was pumped from the small supply tank, shown in Figure 29, through a constant level tank and into the suction side of the booster pump, located immediately in front of the flowmeter. An orifice meter and valve were used to control the saltwater flowrate.

The purpose of the density stratified flow experiments was to determine the influence of density differences, quantitatively measured by the densimetric Froude number, on the C-diagrams and other pertinent parameters.
CHAPTER 5

PRESENTATION AND DISCUSSION OF RESULTS

Verification of the Hydraulic Model

Experimental data from Experiments 4, 5, and 6 of Chapter 3 for ponds D, A-1, and E, respectively, were used to compare with the model experiments for verification purposes. The results of the model experiments performed to verify existing pond behavior were divided into two main groups: (1) unstratified flow (Reynolds number criterion) and (2) density stratified flow (densimetric Froude number criterion).

Plain flow (constant density)

Hydraulic model experiments were performed to verify the experimental tracer data of ponds D and E of the Logan pond system. The model parameters were determined using Reynolds number criterion since the prototype data for ponds D and E indicated constant density flows.

Figure 31 gives the dimensionless C-diagrams for the prototype data, P-2 and two model runs, M-12 and M-4. Table 1 gives the model and prototype hydraulic parameters, the values for the dimensionless mean residence time, plug flow deviation parameter, dead space, and the modeling parameters.
Figure 31. C-diagram for Experiments P-2, M-12, and M-4.
Table 1. Model and prototype pond data and experimental results for constant density experiments.

<table>
<thead>
<tr>
<th>Experiment No.</th>
<th>Depth (ft)</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Q (gpm)</th>
<th>Re No.</th>
<th>t (min)</th>
<th>$\bar{\theta}_c$</th>
<th>$\theta_b$</th>
<th>$\theta_{pk}$ (c/c)</th>
<th>$\bar{\theta}_p$</th>
<th>$\bar{V}_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-2 (pond D)</td>
<td>7.5</td>
<td>870</td>
<td>1980</td>
<td>4039</td>
<td>617</td>
<td>398.8 hr.</td>
<td>.652</td>
<td>.080</td>
<td>.183</td>
<td>1.94</td>
<td>.509</td>
</tr>
<tr>
<td>M-12</td>
<td>1.5</td>
<td>20</td>
<td>40</td>
<td>76</td>
<td>613</td>
<td>118.1</td>
<td>.625</td>
<td>.103</td>
<td>.159</td>
<td>2.72</td>
<td>.573</td>
</tr>
<tr>
<td>M-4</td>
<td>1.5</td>
<td>20</td>
<td>40</td>
<td>80</td>
<td>606</td>
<td>112.2</td>
<td>.632</td>
<td>.083</td>
<td>.176</td>
<td>1.75</td>
<td>.555</td>
</tr>
<tr>
<td>P-5 (pond E)</td>
<td>7.75</td>
<td>870</td>
<td>1421</td>
<td>6790</td>
<td>1040</td>
<td>175.9 hr.</td>
<td>.599</td>
<td>.026</td>
<td>.063</td>
<td>3.42</td>
<td>.646</td>
</tr>
<tr>
<td>M-6</td>
<td>1.5</td>
<td>20</td>
<td>32.5</td>
<td>80</td>
<td>594</td>
<td>91.2</td>
<td>.560</td>
<td>.101</td>
<td>.165</td>
<td>3.67</td>
<td>.630</td>
</tr>
</tbody>
</table>

Modeling Parameters (Experiments P-2, M-12 and M-4)  

- $d_o r = 1/5^a$  
- $R_{er} = \frac{v_r (d_o r)}{v_r} = \frac{Q_r}{L_r v_r} = 1.0$  
- $L_r = 1/46$  
- $Q_r = 1/50.3$  
- $v_r = .915$  
- $t_r = \frac{L_r d_r}{v_r} = 1/210$

Modeling Parameters (Experiments P-5 and M-6)  

- $Q_r = L_r v_r = 1/45.7^b$  
- $L_r = 1/43.5$  
- $t_r = L_r^2 (d_o r) = 1/125.4$

(a) $d_o r$ means the depth of the model divided by the depth of the prototype.

(b) $Q_r$ was not satisfied for M-6.
The main difficulty in modeling pond D was that of duplicating the inlet flow patterns. The difference between model Experiment M-12 and M-4 was that of inlet design. It was found by trial and error that the inlet jet in the model had to be greatly diffused in order to approximate the correct peak concentration of the tracer, the peak time, and the time when the tracer first appeared at the outlet. The C-diagrams for Experiments M-12 and M-4 show the sensitivity of the inlet design in the modeling of pond D. Experiment M-4 appears to be fairly close to P-2 but the inlet design was somewhat arbitrary in that the inlet flow was diffused in such a way as to nullify the jet. This influence of the inlet indicates a need to be able to model the spread of prototype jet flows. An improvement in inlet design would undoubtedly lead to better duplication of the prototype by hydraulic model studies.

For Experiment M-4 the 2 foot long diffuser described in Chapter 4 was used. It was laid on the bottom of the model pond in the appropriate location to satisfy geometrical similarity.

Figure 31 for Experiments P-2, M-12, and M-4 shows the typical humps in the C-diagram that result from the gross circulation patterns that are developed by the inlet flow jet. Visual observation of the model experiments verified the general counter-clockwise circulation patterns around the pond that were observed in Experiments 3 and 4 of Chapter 3. It was found that the degree of short circuiting with inlets, such as exist in pond D, is affected by the Reynolds number and also the model inlet design.
The M-4 represents a good model representation of pond D. The C-diagrams for P-2 and M-4 are quite similar and the values of the dimensionless mean, dimensionless beginning time and time to peak, peak concentration, and the plug flow deviation parameter are all within 10 percent of each other. Dead space differs by 14 percent for P-2 and M-4. This difference is felt to be within the realm of experimental reproducibility.

In verifying Experiment 4 on pond E the same problem of modeling the inlet flow pattern was encountered. Pond E has two 30 inch inlet pipes that lay on the bottom of the pond discharging the flow south from the northwest corner of the pond (see Figure 4).

The method of modeling the inflow was the same as for pond D except that the 2 foot diffuser was placed in the proper location for geometrical similarity of pond E.

Figure 32 gives the C-diagrams for Experiment 7, P-5, on pond E and the verification Experiment, M-6. Table 1 gives the dimensions, hydraulic data, and the pertinent parameters determined from the experimental results of P-5 and M-6. The dimensionless mean, \( \bar{\theta}_{pf} \) and peak concentration are fairly close in numerical value. The time to peak and beginning time of the tracer is off considerably along with the dead space quantity.

In modeling ponds D and E, the main problem seems to be that of inlet design. The dimensionless concentration versus time curves are
Figure 3.2. C-diagram for Experiments for P-5 and M-6.
very sensitive, especially the front portion, to inlet effects. It has been observed that undiffused inlet flows such as exist in ponds D and E, create pond-wide circulation patterns which have a direct influence on the degree of short circuiting. This same type of behavior took place in the model studies to varying degrees depending on the type of inlet used. It was found that the more the incoming flow was diffused, the closer the model could be made to resemble the actual pond results. The difference that exists in the results for P-5 and M-6 is attributed mainly to that of inlet design. The effect of decreasing the flowrate ratio between model and prototype for pond E (i.e. $Q_m < Q_m$ computed from Reynolds number criterion) probably had an effect also on the results of Experiment M-6 in relation to P-5.

**Density stratified flow**

The model experiment to verify Experiment 6 on primary pond A-I was designed according to the densimetric Froude number criterion. If the model experiment had been designed according to the densimetric Froude-Reynolds number criterion, very large quantities of salt would have been needed to give the proper density ratio between model and prototype. Therefore a lower flowrate was chosen but still resulting turbulent flow, so as to reduce the amount of salt needed.

Figure 33 gives the C-diagrams for Experiment P-3 in pond A-I, and the model verification Experiment, M-7. Table 2 gives the pertinent data and experimental results for these two experiments where the
Figure 33. C-diagrams for Experiments P-3 and M-7.
Table 2. Model and prototype pond data and experimental results for stratified flow experiments.

<table>
<thead>
<tr>
<th>Experiment No.</th>
<th>Depth (ft)</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Q (gpm)</th>
<th>Re No.</th>
<th>$\bar{t}$ (min)</th>
<th>$\bar{\theta}_c$</th>
<th>$\theta_b$</th>
<th>$\theta_{pk}$ (c/c_o)</th>
<th>$\bar{\theta}_pf$</th>
<th>$\bar{V}_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-3(pond A-1)</td>
<td>5.5</td>
<td>1421</td>
<td>2912</td>
<td>4765</td>
<td>690</td>
<td>595.4 hr.</td>
<td>.586</td>
<td>.017</td>
<td>.092</td>
<td>1.78</td>
<td>.596</td>
</tr>
<tr>
<td>M-7</td>
<td>2.5</td>
<td>20</td>
<td>40</td>
<td>50</td>
<td>362</td>
<td>299.2</td>
<td>.706</td>
<td>.086</td>
<td>.108</td>
<td>1.73</td>
<td>.443</td>
</tr>
</tbody>
</table>

Modeling Parameters

\[
F_\Delta = \frac{V}{\sqrt{g_d \Delta \rho / \rho}}
\]

\[
\left(\frac{\Delta \rho}{\rho}\right)_r = \frac{Q_r^2}{L_r d_o^3} = 8.38
\]

\[
t_r = \frac{L_r^2 d_o}{Q_r} = 1/130
\]

\[
L_r = 1/75
\]

\[
(d_o)_r = 1/2.2
\]

\[
Q_r = 1/95.3
\]
inflow was more dense than the receiving fluid. It is easy to recognize from Figure 33 and the results in Table 2 that the model did not fit the prototype experimental data very well. There are a number of possible reasons for this lack of fit of model and prototype experiments. Some of these are:

1. Reynolds number equality was not followed. With the large model that was used in the model experiments, a reduction in the flow-rate was a necessity due to the large amounts of salt that would have been needed to give the proper density differences. It is not known exactly what effect this had except that the Reynolds number should have been greater which would have resulted in a higher turbulence level of the flow.

In visually observing Experiment M-7, streaks of tracer were noticed during the experiment. These streaks of concentrated tracer show up at the outlet concentration versus time curve as peaks. The obvious lack of mixing which did occur in the pond A-1 after a certain time is probably due to low turbulence level of the model experiment, particularly near the bottom where the tracer was highest in concentration.

2. The data on pond A-1 were corrected due to the loss of tracer that occurred during Experiment 6 (Chapter 3). Therefore it is felt that the model data are probably as reliable as the prototype data.

3. Another possible reason is attributed to the approximate modeling of density currents and stratified flow situations with salt water to
give the necessary density difference. But this was necessary, as it was impossible for the water to be heated sufficiently to give the required density difference to satisfy the densimetric Froude number criterion.

4. The last reason might be due to errors in flowrate measurement, measurement of depth and temperature, fluorometer calibration curves, variations in inlet flow patterns and others which are involved in experimental data taking.

In summarizing, it is concluded that existing pond behavior can be verified and simulated in the lab for situations involving plain or constant density flows as long as the inlet flow pattern can be closely approximated. (Proper simulation of prototype inlet design.)

It is not known how well density-stratified flows in stabilization ponds can be modeled. To be able to determine the validity of the modeling laws, better data must be taken on existing ponds to be used to compare with the hydraulic model. The prototype data, on density stratified flows, taken in this study were not adequate for a proper accounting of the modeling laws. The main reasons for the lack of reliable data for density stratified flows in this study were due to two main factors: (1) The large size of the primary ponds (A-1 and A-2 of Figure 9) which posed problems for obtaining data due to the very long theoretical detention times, and (2) the environmental and hydraulic changes that occurred. These two factors actually had an effect on the experiments that were performed. (Refer to Experiments 5 and 6, Chapter 3.)
Plain flow (constant density)

Experimental reproducibility

A number of experiments were performed on the hydraulic model to determine the extent of the reproducibility of the results to obtain an idea of the reliability of the model data. For this series of experiments, including the experiments on depth and Reynolds number variations, the model was designed as outlined in Chapter 5.

Figures 34, 35, and 36 show the C-diagrams for the experiments performed with the hydraulic and geometric conditions given in Table 3, groups A, A-1, and A-2 respectively. Table 3 also gives the values for the important parameters that were computed for each experiment. The C-diagrams for these sets of experiments, exhibit quite a bit of variability particularly between $\theta = 0.0$ to $\theta = 1.0$. The parameter variation tabulated in Table 3 is not as extreme as the C-diagrams seem to indicate. If the lack of exact reproducibility is assumed to be of the order of 10 percent, then the variability about the dimensionless mean, $\bar{\theta}_c$, plug flow deviation parameter, $\bar{\theta}_{pf}'$, and dead space, $V_d'$, are within the realm of experimental reproducibility. It obviously takes more pronounced variations in the C-diagram to greatly effect these parameters.

There are a number of factors that have an influence on reproducibility and are important to consider in determining the reliability of the
Figure 34. C-diagrams for Experiments D-53, D-54, and D-56.
Figure 35. C-diagrams for Experiments R-21, R-23, and R-24.
Figure 36. C-diagrams for Experiments O-6, O-61, and O-62.
Table 3. Model pond data and experimental results for constant density experiments.

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<td>.816</td>
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data. Following is a list of the most important ones:

1. Residual eddies or currents resulting from filling the model pond prior to each experiment.

2. Variations in flow patterns that depend, to a considerable degree, on the length of time the model was run for stabilization purposes before experiment was started.

3. Errors in flow measurement at both the inlet and outlet of the model pond.

4. Temperature variations of fluid during the experiment. The temperature changes affect density changes of the fluid that could result in density currents. Also, the concentration readings obtained with the fluorometer are dependent upon the temperature and temperature fluctuations affect the fluorometer readings.

5. Errors in the calibration curves for the fluorometer.

6. Errors in geometrical measurements, particularly depth measurements.

7. Errors due to random fluid behavior in inlet and outlet flows and random large scale turbulence of flow within the pond.

Figure 37 gives the C-diagrams for two separate experiments that were performed to obtain an idea of how factor (2) could affect the results of the experiments performed. The results are tabulated as group A-3 of Table 3. A simple explanation of the differences that exist can be given as follows: In order for the eventual flow patterns to stabilize,
Figure 37. C-diagrams for Experiments D-55 and D-56.
sufficient time must pass for the moving fluid particles to exert shearing stresses to effectuate movement of the adjacent fluid particles and they in turn exert shearing stress on other adjacent fluid particles until all the fluid within the pond is moving as it eventually would after a long period of time. Experiments D-55 and D-56 were not started until the fluid within the pond had stood overnight to allow for the currents and eddies, resulting from the filling procedure, to die out. The difference between these two experiments was that experiment D-55 was run at the proper flowrate for 20 minutes before injecting the tracer into the pond while D-56 was operated for over 2½ hours before dye injection. The resulting C-diagrams verify the explanation above. Experiment D-55 had a higher peak concentration and a greater mass of dye leaving the pond at the beginning than did D-56, indicating a more serious short circuiting problem. This is because insufficient time had elapsed during the stabilization phase to effectuate total fluid movement within the pond which resulted in a reduced area of flow and subsequently higher concentrations of tracer leaving the pond.

In evaluating the importance of changes in pond geometry or the hydraulic characteristics of the ponds and their effects on the hydraulic efficiency, all experimental variability must be considered and taken into account.
Effect of depth

A number of experiments were performed to evaluate the effect of changing the depth with all other variables held constant. Group A-I and Group B of Table 3 are the results of these experiments. Figure 38 shows the plot of these parameters versus the depth to width ratio $d_o/W$, for each experiment. From the plots, as the $d_o/W$ ratio increases, i.e. as the depth increases, $\bar{V}_d$ increases, $\bar{\theta}_{pf}$ increases, and $\bar{\theta}_c$ decreases, which all indicate less hydraulic efficiency and subsequently less biological conversion of organic matter. There is considerable scatter but the data plotted in Figure 38 suggest a trend probably exists.

Effect of Reynolds number

Groups A, A-I, and C of Table 3 contain the results of the experiments conducted to show the effect of Reynolds number. The Reynolds number was defined and used as

$$R_e = \frac{\bar{V} d_o}{\nu} = \frac{Q}{W \nu}$$

in which

- $Q$ = flowrate
- $W$ = width of pond
- $d_o$ = depth of pond
- $\nu$ = kinematic viscosity

Figure 39 shows $\bar{V}_d$, $\bar{\theta}_{pf}$, and $\bar{\theta}_c$ plotted against the Reynolds number. When considering the possible variability just due to experimental error,
Figure 38. Plot of $\bar{\theta}_c$, $\bar{\theta}_p$, and $\bar{V}_d$ vs. $d_0/W$ for D-series experiments.
Figure 39. Plot of $\overline{\theta_c}$, $\overline{\theta_{pf}}$, and $V_d$ vs. $Re$ for R-series experiments.
Figure 39 shows essentially no relationship for these parameters as a function of Reynolds number. Dead space and $\bar{\theta}_{pf}$ show a slight decrease as $Re$ increases which means greater efficiency of operation. At higher Reynolds numbers, this would be expected due to greater turbulence and larger diffusivity coefficients which result in greater mixing and interchange of the fluid particles.

Effects of inlets and outlets

Two types of outlet configurations were tried along with a number of inlet variations. Figure 40 shows the inlet and outlet configurations that were used in this study. D and E of Table 4 give the results of these experiments.

Figure 41 illustrates the C-diagrams for the experiments of group D. From this figure and from Table 4 it appears that O-5 is the best design for maximum hydraulic efficiency. It has the highest dimensionless mean retention time and the lowest values for $\bar{\theta}_{pf}$ and dead space which indicate maximum efficiency. Figure 42 shows the C-diagrams for the experiments of Group E of Table 4. From these results and in visually evaluating the C-diagrams of Figure 42, it is not readily evident which is best, but probably I-5 and M-4 would give maximum hydraulic efficiency.

Length to width ratio effects

Figure 43 shows how the model was modified to determine the effects of L/W ratio changes on the hydraulic efficiency of ponds. The
Figure 40. Inlet and outlet configurations employed.
Table 4. Model pond data and experimental results for constant density experiments.

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Figure 41. C-diagrams for Experiments O-1, O-2, O-3 and O-5.
Figure 42. C-diagrams for Experiments M-3, M-4, O-4 and I-5.
Figure 43. Model modifications to determine length to width ratio effects.
G-series utilized a movable wall as shown in Figure 43(a). Figures 43(b) and 43(c) show the two configurations employing baffles that were used to increase the L/W ratio. The results of these experiments are tabulated in Table 4 as Group F. Figure 44 gives the experimentally determined C-diagrams for these experiments. In referring to the C-diagrams and the tabulated results for Experiments G-2, G-3, G-4, G-5, and D-54, an increase in the hydraulic efficiency occurs for an increase in the L/W ratio. The experiments employing baffles show a marked increase in the hydraulic efficiency as measured by the values of $\bar{\theta}_c$, $\bar{\theta}_{pf}$, and $\bar{V}_d$. Short circuiting with the baffled experiments was not as severe as with the G-series experiments. The direct comparison of the experiments of the C and G series cannot be made because of the different kind of flow that exists in the baffled system. There exists complex secondary currents in the corners of baffled flow which undoubtedly have an effect on the C-diagrams and the hydraulic flow parameters. The significance and importance of the L/W ratio are still very evident in the tracer results. There was a significant increase in the hydraulic efficiency as shown by the values of $\bar{\theta}_c$, $\bar{\theta}_{pf}$, and $\bar{V}_d$, and the C-diagrams for Experiments C-1 and C-2. The relationship between these parameters the C-diagrams and the L/W ratio is intuitively clear. As the L/W ratio increases, the conditions for plug flow are closer to being satisfied and as an end result, the efficiency of the treatment increases.
Figure 44. C-diagrams for experiments with different length to width ratios.
Discussion of plain flow experiments

It was mentioned previously that it apparently takes significant changes in geometry or in the hydraulic characteristics to greatly affect the parameters $\bar{\theta}_c$, $\bar{\theta}_{pf}$, and $\bar{V}_d$. In reviewing the results of the model experiments and also the existing pond experiments P-1, P-2, and P-5 for plain flow tabulated in Tables 3 and 4, it is recognized that there is not a great deal of variability between the experiments, excluding C-1 and C-2. The values of $\bar{\theta}_c$ for most of the experiments varied between $\bar{\theta}_c = .50$ to $\bar{\theta} = .65$, for $\bar{\theta}_{pf}$ between $\bar{\theta}_{pf} = .50$ to $\bar{\theta}_{pf} = .65$ and for dead space between $\bar{V}_d = .45$ to $\bar{V}_d = .60$. For one type of design in comparison with another there might be an increase in $\bar{\theta}_c$ of 30 percent and a decrease of $\bar{\theta}_{pf}$ and $\bar{V}_d$ of 30 percent. This would probably not result in a straight increase of 30 percent in treatment efficiency as the efficiency would be determined by Equation 28 with $c/c_o$ for both pond designs. But Experiments C-1 and C-2 show an increase of 90 percent and 44 percent respectively in the mean detention time in comparison with $\bar{\theta}_c$ for Experiment G-2. For $\bar{\theta}_{pf}$, Table 4, indicates a decrease of 57 percent and 36 percent for C-1 and C-2 respectively. For $\bar{V}_d$ a decrease of 79 percent and 39 percent for C-1 and C-2 is also shown in Table 4. This means that the L/W ratio seems to be the most important design parameter affecting the hydraulic characteristics of ponds. This is a logical result. As baffles are placed in ponds, the average length of travel of a fluid particle is increased and the width of flow is decreased. As the L/W ratio increases, the conditions for one-dimensional or plug
flow are closer to being satisfied. The conditions for plug flow are desirable for maximum treatment efficiency, provided sufficient mixing occurs between the inflow and the microorganisms in the pond to cause treatment to occur. Since an intimate contact of degradable organic matter and the microorganisms is necessary, the desirable flow situation would be where the inflow laterally and vertically mixes with the ambient fluid then flows as a slug towards the outlet. This would result in minimum values for $\bar{\theta}_{pf}$ and $\bar{V}_d$ and a maximum value for $\bar{\theta}_c$ which in turn would increase the treatment efficiency for a given pond design.

**Stratified flow (variable density)**

The model was designed as was described in Chapter 4. Prior to the placement of dye for each experiment, the flowrate into and out of the model was set at the proper value for a certain length of time to allow for the flow to stabilize. The stabilizing time for the stratified flow experiments was considerably less than the stabilizing time for the constant density experiments. The reduced time was necessary due to the problem of salt buildup or washout that would occur for long periods of model operation prior to dye injection. The stabilizing time was arbitrarily chosen between 15 and 30 minutes depending on the flowrate.

Table 5 gives a list of the density stratified flow experiments with the experimental results that were performed. The density of the inflow of group A was greater than the density of the pond fluid while the reverse was true for the experiments listed in group B. Figure 45 illustrates
Table 5. Model pond data and experimental results for stratified flow experiments.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Depth (ft)</th>
<th>Q (gpm)</th>
<th>Re</th>
<th>$\tau_t$ (min)</th>
<th>$\bar{\theta}_c$</th>
<th>$\theta_b$</th>
<th>$\theta_{pk}$</th>
<th>$\bar{\theta}_{pf}$</th>
<th>$\bar{v}_D$</th>
<th>$F_\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Group A $\rho_{in} &gt; \rho_{pond}$</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-1</td>
<td>2.5</td>
<td>50</td>
<td>362</td>
<td>299.2</td>
<td>.706</td>
<td>.086</td>
<td>.108</td>
<td>.443</td>
<td>.422</td>
<td>.00281</td>
</tr>
<tr>
<td>S-2</td>
<td>1.5</td>
<td>95</td>
<td>666</td>
<td>94.5</td>
<td>.547</td>
<td>.127</td>
<td>.198</td>
<td>.590</td>
<td>.602</td>
<td>.0183</td>
</tr>
<tr>
<td>S-6</td>
<td>1.5</td>
<td>80</td>
<td>598</td>
<td>112.2</td>
<td>.648</td>
<td>.107</td>
<td>.123</td>
<td>.530</td>
<td>.538</td>
<td>.00975</td>
</tr>
<tr>
<td>S-7</td>
<td>1.5</td>
<td>78</td>
<td>583</td>
<td>115.1</td>
<td>.621</td>
<td>.108</td>
<td>.138</td>
<td>.531</td>
<td>.581</td>
<td>.0120</td>
</tr>
<tr>
<td><strong>Group B $\rho_{in} &lt; \rho_{pond}$</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-3</td>
<td>1.5</td>
<td>80</td>
<td>598</td>
<td>112.2</td>
<td>.590</td>
<td>.079</td>
<td>.125</td>
<td>.564</td>
<td>.590</td>
<td>.0200</td>
</tr>
<tr>
<td>S-4</td>
<td>1.5</td>
<td>80</td>
<td>598</td>
<td>112.2</td>
<td>.540</td>
<td>.060</td>
<td>.097</td>
<td>.523</td>
<td>.586</td>
<td>.0150</td>
</tr>
<tr>
<td>S-5</td>
<td>1.5</td>
<td>80</td>
<td>598</td>
<td>112.2</td>
<td>.497</td>
<td>.057</td>
<td>.072</td>
<td>.655</td>
<td>.627</td>
<td>.00975</td>
</tr>
</tbody>
</table>
Figure 45. C-diagrams for Experiments S-2 and S-4 (density stratified flow) and D-55.
some typical C-diagrams for two of the density stratified flow experiments. Also shown in this figure is the constant density Experiment D-55. In all three of these experiments the flow was allowed to stabilize for about 20 minutes before dye injection. These curves show that in general when the density of the influent is greater than in the receiving body, the values of the time to peak ($\theta_{pk}$), beginning time, ($\theta_b$), and $c/c_0$ are greater than the corresponding values when $\rho_{in} < \rho_{pond}$. The reasons for these differences are given as follows: When $\rho_{in} > \rho_{pond}$, the inflow tracer or waste material flows along the bottom of the pond due to its higher density. The movement through the pond is slower due to the lower velocities that occur near the pond bottom due to the influence of the bottom boundary of the pond. This causes the tracer to take a longer time to reach the outlet. The efficiency of mixing in the lower depths of the pond is less due to reduced turbulence level and subsequently the tracer or waste material is higher in concentration ($c/c_0$)$_{pk}$ when it leaves the pond. The reverse is true when $\rho_{in} < \rho_{pond}$. The tracer is concentrated in the higher velocity region of the pond, with respect to depth, and it thus reaches the outlet in a shorter time interval. In this region of flow there is greater mixing efficiency due to the higher turbulence level which exhibits itself in greater turbulent diffusion of the tracer or waste material. The greater turbulent diffusion may explain the lower peak concentration at the outlet. When $\rho_{in} = \rho_{pond}$, the C-diagram and the values of $\bar{\theta}_c$, $\bar{\theta}_{pf}$, $\theta_b$, $\theta_{pk}$, and $\bar{V}_d$ should be somewhere between
the values of these same parameters for the two types of density-
stratified flows. Referring to the results of Experiment D-55 recorded
in group A-3 of Table 3 and the results of the density-stratified flow
experiments recorded in Table 5 and shown in Figure 45, this seems to
be true.

Figures 46 and 47 are photographs of the dye tracer as it flows
out of the diffuser for $\rho_{\text{in}} < \rho_{\text{pond}}$ and $\rho_{\text{in}} > \rho_{\text{pond}}$ respectively. These
photographs visually show the tracer-flowing near the bottom or near the
surface of the pond for the two cases of density-stratified flow. Figures
48 and 49 are photographs taken from the side and above the diffuser
respectively when $\rho_{\text{in}} > \rho_{\text{pond}}$.

The effect of the densimetric Froude number on the results is shown
in Table 5. The important parameters listed in Table 5 are shown in
Figures 50, 51, and 52 plotted versus the densimetric Froude number,
$F_{\Delta}$. Figure 50 shows $\bar{\theta}_{\text{pf}}$ and $\theta_{\text{b}}$ for both density difference situations,
plotted against $F_{\Delta}$. The curves for $\bar{\theta}_{\text{pf}}$ vs. $F_{\Delta}$ for $\rho_{\text{in}} > \rho_{\text{pond}}$ are
close to bracketing the values of $\bar{\theta}_{\text{pf}}$ for Experiments D-55 and D-56
when $F_{\Delta} = \infty$ ($\rho_{\text{in}} = \rho_{\text{pond}}$). Figure 51 shows similar curves for $\bar{\theta}_{\text{c}}$
and $\theta_{\text{pk}}$ plotted against $F_{\Delta}$. These curves also bracket the values of
$\bar{\theta}_{\text{c}}$ for Experiments D-55 and D-56 when $F_{\Delta} = \infty$. Figure 52 shows $\bar{v}_{\text{d}}$
plotted against $F_{\Delta}$ with the two curves bracketing $\bar{v}_{\text{d}}$ for Experiments
D-55 and D-56 when $F_{\Delta} = \infty$. It is not known exactly why the curves for
$\rho_{\text{in}} > \rho_{\text{pond}}$ and $\rho_{\text{in}} < \rho_{\text{pond}}$ for $\bar{\theta}_{\text{c}}$, $\bar{\theta}_{\text{pf}}$, and $\bar{v}_{\text{d}}$ cross but if these
Figure 46. Photograph of tracer discharging from diffuser for density stratified flow $\rho_{\text{in}} < \rho_{\text{pond}}$.

Figure 47. Photograph of tracer discharging from diffuser for density stratified flow $\rho_{\text{in}} > \rho_{\text{pond}}$. 
Figure 48. Photograph of tracer discharging from diffuser for density stratified flow $\rho_{\text{in}} > \rho_{\text{pond}}$.

Figure 49. Photograph of tracer discharging from diffuser for density stratified flow $\rho_{\text{in}} > \rho_{\text{pond}}$.
Figure 50. Plots of $\bar{\theta}_{pf}$ and $\theta_{pk}$ vs. $F_\Delta$ for $\rho_{in} > \rho_{pond}$ and $\rho_{in} < \rho_{pond}$.
Figure 51. Plots of $\bar{\theta}_c$ and $\theta_b$ vs. $F_\Delta$ for $\rho_{in} > \rho_{pond}$ and $\rho_{in} < \rho_{pond}$.
Figure 52. Plots of $V_d$ vs. $F_\Delta$ for $\rho_{\text{in}} > \rho_{\text{pond}}$ and $\rho_{\text{in}} < \rho_{\text{pond}}$.
curves are extended to infinity they should be bracketed by Experiments D-55 and D-56. Experiments D-55 and D-56 were run to obtain an idea of the influence that the stabilizing time would have upon the results.

This has been discussed before under the section on experimental reproducibility. The values of \( F_\Delta \) greater than 0.015 represent stratified flows of low density differences. It is felt that when \( F_\Delta \) is greater than 0.015 that there is little change in the parameter values or in other words that there is a constant relationship between \( F_\Delta \) and the parameter values. If this is the case then the difference between the values of \( \bar{\theta}_c \), \( \bar{\theta}_{pf} \) and \( \bar{V}_d \) when \( F_\Delta > 0.015 \) for \( \rho_{in} > \rho_{pool} \) and \( \rho_{in} < \rho_{pool} \) might be due to differences because of lack of reproducibility in the model.

More experiments with a greater range of densimetric Froude numbers should have been made to verify this explanation. Also, more experiments should have been performed to obtain a better idea of the effect of stabilization time on the results. Due to the lack of time and the large amounts of salt required to make the density experiments, this was not done in this research.

In summarizing, it is felt that the results contained herein for different degrees of stratified flow are good considering the variability due to lack of experimental reproducibility and the approximations that were made. There appears to be no choice but to accept the results with certain approximations when using salt (mass transfer) as a method of creating density stratified flows. The effect of both the use of salt to
create density differences and the short stabilization time is to probably shift the curves of $\bar{\theta}$, $\bar{\theta}_p$, and $\bar{V}_d$ vs. $F_\Delta$ either up or down depending on the flow situation but it is felt that the general relationship with $F_\Delta$ is valid. The existing ponds in the Logan pond system have values of $F_\Delta$ that are within the range of the values of $F_\Delta$ that were used in these experiments and therefore the results should apply to existing ponds.

**Expected Treatment Efficiency For Selected Experiments**

In the design of a waste stabilization pond, it would be desirable to be able to predict the percent conversion of the waste material to stable end products during its residence in the pond. Equation 27 or Equation 28 of Chapter 2 can be used for this purpose, providing the age distribution function, $c/c_o$, the theoretical detention time, $\bar{t}$, and the first order reaction coefficient, $K$, are known. Since all pollutant remaining in the pond longer than two detention times is ignored in computing $N_2/N_o$, the quantity $(1 - N_2/N_o)$ represents the fraction of waste material that has undergone reaction.

Equation 28 consists of two parts: 1) the residence time distribution of the fluid particles $c/c_o$ as a function of $\theta$ and 2) the biological first order reaction equation, $N/N_o = e^{-K\bar{t}\theta}$ as a function of $K$, $\bar{t}$, and $\theta$. To determine the effect of the hydraulic design on the treatment efficiency, $K$ and $\bar{t}$ must be held constant. Since the extent of reaction of a waste is highly dependent on $\bar{t}$ for a given $K$, the detention time
used in Equation 28 must be the same for all pond designs for a meaningful comparison. The $c/c_o$ data as a function of $\theta$ are then sufficient to determine the overall treatment efficiency for each pond design and by comparing the treatment efficiencies for these designs it will be possible to determine the most efficient hydraulic design.

Table 6 gives $N_2/N_0$ for some of the experiments discussed earlier. The theoretical detention time used to determine $N_2/N_0$ was the same for all the experiments listed in Table 6. This makes it possible to compare the pond designs on the same basis even though each experiment had a different theoretical detention time. The values of $K$ used in Equation 28 that were selected were based on the average values (Table 7) presented by Fair, Geyer, and Okun. Also, included in Table 6 are the values of $N_2/N_0$ for plug flow which are used to compare with the treatment efficiencies of the other experiments. Computer program N-2 in the Appendix was used to determine the treatment efficiency.

The model pond results for treatment efficiency do indicate the importance of proper hydraulic design of waste stabilization ponds. The best hydraulic design, that of large $L/W$ ratios, is further verified by the values of $N_2/N_0$ for Experiments C-1 and C-2. The trend of higher treatment efficiencies for higher $L/W$ ratios is evidenced here. Experiment C-1 shows 83 percent removal, C-2 shows 75 percent removal while most of the other experiments listed in Table 6 show 60-75 percent removal when $K = .40$. C-1 shows a treatment efficiency that is quite
Table 6. Treatment efficiency for selected experiments.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>t (days)</th>
<th>K=0.20</th>
<th>K=0.25</th>
<th>K=0.35</th>
<th>K=0.40</th>
<th>% Treatment Efficiency K=0.40</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-53</td>
<td>5.0</td>
<td>0.504</td>
<td>0.444</td>
<td>0.393</td>
<td>0.350</td>
<td>0.312 69</td>
</tr>
<tr>
<td>D-54</td>
<td>&quot;</td>
<td>0.466</td>
<td>0.413</td>
<td>0.369</td>
<td>0.331</td>
<td>0.298 71</td>
</tr>
<tr>
<td>D-56</td>
<td>&quot;</td>
<td>0.395</td>
<td>0.352</td>
<td>0.316</td>
<td>0.285</td>
<td>0.258 74</td>
</tr>
<tr>
<td>D-12</td>
<td>&quot;</td>
<td>0.455</td>
<td>0.416</td>
<td>0.381</td>
<td>0.351</td>
<td>0.323 68</td>
</tr>
<tr>
<td>O-6</td>
<td>5.0</td>
<td>0.442</td>
<td>0.390</td>
<td>0.345</td>
<td>0.307</td>
<td>0.273 73</td>
</tr>
<tr>
<td>O-61</td>
<td>&quot;</td>
<td>0.446</td>
<td>0.399</td>
<td>0.358</td>
<td>0.323</td>
<td>0.293 71</td>
</tr>
<tr>
<td>O-62</td>
<td>&quot;</td>
<td>0.410</td>
<td>0.361</td>
<td>0.320</td>
<td>0.285</td>
<td>0.255 74</td>
</tr>
<tr>
<td>O-2</td>
<td>5.0</td>
<td>0.495</td>
<td>0.452</td>
<td>0.415</td>
<td>0.383</td>
<td>0.355 64</td>
</tr>
<tr>
<td>O-3</td>
<td>&quot;</td>
<td>0.477</td>
<td>0.428</td>
<td>0.386</td>
<td>0.348</td>
<td>0.316 68</td>
</tr>
<tr>
<td>O-4</td>
<td>&quot;</td>
<td>0.618</td>
<td>0.575</td>
<td>0.539</td>
<td>0.507</td>
<td>0.479 52</td>
</tr>
<tr>
<td>O-5</td>
<td>&quot;</td>
<td>0.463</td>
<td>0.408</td>
<td>0.361</td>
<td>0.320</td>
<td>0.285 71</td>
</tr>
<tr>
<td>M-4</td>
<td>&quot;</td>
<td>0.480</td>
<td>0.429</td>
<td>0.386</td>
<td>0.349</td>
<td>0.318 68</td>
</tr>
<tr>
<td>I-5</td>
<td>&quot;</td>
<td>0.486</td>
<td>0.424</td>
<td>0.372</td>
<td>0.307</td>
<td>0.289 71</td>
</tr>
<tr>
<td>G-2</td>
<td>5.0</td>
<td>0.547</td>
<td>0.503</td>
<td>0.464</td>
<td>0.430</td>
<td>0.401 60</td>
</tr>
<tr>
<td>G-3</td>
<td>&quot;</td>
<td>0.517</td>
<td>0.461</td>
<td>0.418</td>
<td>0.381</td>
<td>0.349 65</td>
</tr>
<tr>
<td>C-1</td>
<td>&quot;</td>
<td>0.388</td>
<td>0.316</td>
<td>0.259</td>
<td>0.213</td>
<td>0.177 83</td>
</tr>
<tr>
<td>C-2</td>
<td>&quot;</td>
<td>0.443</td>
<td>0.380</td>
<td>0.328</td>
<td>0.284</td>
<td>0.246 75</td>
</tr>
<tr>
<td>S-2</td>
<td>5.0</td>
<td>0.453</td>
<td>0.409</td>
<td>0.372</td>
<td>0.339</td>
<td>0.310 69</td>
</tr>
<tr>
<td>S-3</td>
<td>&quot;</td>
<td>0.495</td>
<td>0.456</td>
<td>0.422</td>
<td>0.393</td>
<td>0.368 63</td>
</tr>
<tr>
<td>S-4</td>
<td>&quot;</td>
<td>0.418</td>
<td>0.376</td>
<td>0.341</td>
<td>0.310</td>
<td>0.284 72</td>
</tr>
<tr>
<td>S-6</td>
<td>&quot;</td>
<td>0.404</td>
<td>0.358</td>
<td>0.320</td>
<td>0.287</td>
<td>0.239 74</td>
</tr>
<tr>
<td>P-5</td>
<td>5.0</td>
<td>0.571</td>
<td>0.521</td>
<td>0.479</td>
<td>0.444</td>
<td>0.413 59</td>
</tr>
<tr>
<td>Plug Flow</td>
<td>5.0</td>
<td>0.368</td>
<td>0.286</td>
<td>0.223</td>
<td>0.173</td>
<td>0.135 86</td>
</tr>
</tbody>
</table>

Table 7. Values of K for wastewaters of various concentrations.a

<table>
<thead>
<tr>
<th>Type of Wastewater</th>
<th>K(day⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak wastewater</td>
<td>0.35</td>
</tr>
<tr>
<td>Strong wastewater</td>
<td>0.39</td>
</tr>
<tr>
<td>Primary effluent</td>
<td>0.35</td>
</tr>
<tr>
<td>Secondary effluent</td>
<td>0.12-0.23</td>
</tr>
<tr>
<td>Tap water</td>
<td>&lt; 0.12</td>
</tr>
</tbody>
</table>

aFair, Geyer, and Okun.
close to \((1 - \frac{N}{N_0})\) for plug flow. This points to the desirability of using baffled ponds to effectively increase the \(L/W\) ratio.

Equation 28 can be used to determine the treatment efficiency if the experimental age distribution function is known and a proper value of \(K\) can be determined. Equation 27 can be used if a mathematical model for the age distribution function can be developed.

**Application of the Finite Stage Model**

The purpose of this section is to illustrate the application of the finite stage model described in Chapter 2. This section will give some models developed from the experimental data resulting from this research. With a mathematical model for \(c/c_o\), then either Equation 27 or 28 could be used to determine the theoretical treatment efficiency of a proposed or existing pond.

The principles and the method discussed in Chapter 2 were applied to a few selected experiments to show the value and the application of the finite stage model for predicting the age distribution functions of waste stabilization ponds. A computer program was written to make the parameter evaluations for each experiment after which these parameters were used to generate residence time distribution data for each experiment for the purpose of comparing the actual experimental C-diagram with this derived mathematical model of the C-diagrams and the computed C-diagrams for the selected experiments. Figure 53 illustrates a
Figure 53. Experimental and theoretical C-diagrams for Experiments C-1 and C-2.

- $n = 2$
- $K_H = 0.30$
- $F_a = 0.3665$
- $F_b = 0.3665$
- $F_c = 0.2670$
- $K_H = 0.050$
- $F_a = 0.6534$
- $F_b = 0.0066$
- $F_c = 0.340$
relatively flat curve while Figure 57 shows the C-diagram for a steep curve. These figures illustrate the versatility of the finite stage model, and its ability to describe highly skewed distributions as well as relatively flat curves. The parameters for the model of each experiment are given with the figure for that experiment.

The finite stage models for the experimental C-diagrams were computed with the aid of computer program N-3 in the Appendix. This program uses the coefficients determined by Hovorka (1961) for his derived mathematical equation. The input to the program consisted of values for $K_H$ and $f_a$, for each integer value of $n$ and $F_c$, the delay time. The values for $f_a$ and $K_H$ were taken from the plots of $Y_{pk}$ vs. $X_{pk}$ for each value of $n$ in Hovorka's dissertation using the transformed values of $\tilde{\theta}_{pk}$ and $(c/c_0)_{pk}$ for each set of experimental data. Also, the experimental residence time curve (C-diagram) was input data to the program to determine the value of $n$, the number of model units. The output consisted of the derived model of the residence time curve with its appropriate values of $F_a$, $F_b$, $F_c$, $K_H$, and $n$. Each of the figures illustrating the experimental and derived C-diagrams give the computed values of these parameters. This mathematical model fits actual pond data, Figure 57, as well as the data generated with the use of the model pond.

To apply or use this mathematical model to describe the age distribution function and to determine the treatment efficiency, the values of
Figure 54. Experimental and theoretical C-diagrams for Experiment O-5.

- --- O-5
- model

n = 2
K_H = 0.30
F_a = 0.309
F_b = 0.5053
F_c = 0.185
Figure 55. Experimental and theoretical C-diagrams for Experiment R-21.
Figure 56. Experimental and theoretical C-diagrams for Experiment S-2.

- --- S-2
- --- model

$n = 2$

$K_H = 0.520$

$F_a = 0.2095$

$F_b = 0.6635$

$F_c = 0.1270$
Figure 57. Experimental and theoretical C-diagrams for Experiment P-5.

- --- P-5
- - - model

\[ n = 2 \]
\[ K_H = 0.780 \]
\[ F_a = 0.1266 \]
\[ F_b = 0.8474 \]
\[ F_c = 0.026 \]
\( \theta_b, \theta_{pk}, (c/c_{0})_{pk} \) and the experimental residence time distribution curve must be known. For a given rectangular pond design the results of the model studies reported in this chapter could be consulted to estimate values of \( \theta_b, \theta_{pk}, (c/c_{0})_{pk} \) and obtain a representative C-diagram. Mathematical models for many different pond designs could be determined from hydraulic model studies and then used to compute the expected treatment efficiency of existing or proposed waste stabilization ponds. Model studies on triangular and circular ponds with various inlet devices could also be made and mathematical expressions developed from the results for use in future pond designs.

**Discussion of Results**

The experiments that were performed both on the model and the existing ponds for verification purposes indicated a number of important pond design considerations. The experiments on ponds D and E and the model verification experiments for these two ponds revealed that inlets that create jet inflows considerably affect the flow patterns and have considerable influence on the degree of short circuiting. The typical flow pattern caused by this type of inlet (i.e. pipe laid in a corner on the bottom of a prototype pond) is a pond wide circulation current which results in short circuiting, the extent of which is dependent upon the Reynolds number. The C-diagram resulting from this type of circulation is a series of humps spaced at intervals equal to the time it takes for the
bulk fluid to circulate around the pond. The model experiments verify this behavior and indicate the degree that the resulting short circuiting is dependent on the type of inlet used.

The hydraulic model was effectively verified using Reynolds number equality for ponds D and E. The model design was somewhat arbitrary in that the inlet design was a trial and error procedure to obtain proper inlet flow patterns. It was found that the inlet flow had to be greatly diffused in order to approximate the prototype pond behavior.

The verification of the model for the density stratified flow in pond A-1 was not adequate due to the lack of data of sufficient detail or accuracy to describe occurrences in both the model and prototype.

In all of these experiments including those on the existing ponds, the front portion of the C-diagram was found to be quite sensitive to such factors as inlet types and locations, degree of stratification, Reynolds number, and environmental factors that have been ignored in this study. There are other factors that affect the shape of the C-diagram and also influence the magnitude of experimental error. One of the most important was the occurrence of residual eddies and circular currents that were developed in filling the model prior to each experiment. Particular care was taken to minimize this influence on the flow behavior by waiting a certain length of time to allow for the dissipation or die away of the currents and eddies. Then the model pond was put into operation for a length of time before each experiment was begun to allow the flow to
stabilize. The random nature of turbulence is another factor that influences the fluid flow behavior. Because of the randomness of turbulence which exhibits itself in rapidly varying velocities, it is reasonable to conclude that the gross fluid flow patterns are subsequently random in nature. This randomness of flow has a particular influence on the shape of the first portion of the dimensionless concentration versus time curves.

Other factors which influence the experimental error are (1) flow measurement, (2) fluorometer calibration, (3) variations in inlet flow, and (4) temperature measurements.

In reviewing the results of the experiments performed on existing ponds and the experiments on the model, much of the variability among experiments might be due to the random behavior of flow turbulence. The effects of changes in L/W ratio, depth, Reynolds number, densimetric Froude number, inlets and outlets, on the significant parameters \( \bar{\theta}_{pf'} \), \( \bar{\theta}_{c'} \), and \( \bar{V}_{d'} \), were not as large as would be expected even though some of these changes in hydraulic or geometric conditions greatly altered the shape of the C-diagram. There were some notable exceptions, however, with experiments employing L/W ratio changes and certain inlet and outlet types producing the most dramatic changes. The values of the parameters for Reynolds number changes were essentially constant while depth changes did influence the values of these parameters as much
as 30 percent but when the treatment efficiency was considered there was only about 6 percent difference among experiments involving depth changes. The density stratified experiments did indicate a definite relationship of these parameters including $\bar{T}_{pk}$ and $\theta_b$, with the densimetric Froude number. Certain inlet and outlet configurations were obviously poor designs for best hydraulic performance. But it was found that significant parameter change occurred as the L/W ratio became large. The L/W ratio is the most important single factor that effects dramatic changes in the important parameters used here.

The L/W ratio is also significant in the overall treatment efficiency. Significant increases in the calculated treatment efficiency were noted as the L/W ratio increased.

The finite stage model was shown to be an excellent model for representing the age distribution functions of waste water or fluid particles within a stabilization pond. It has been shown that the finite stage model for a flow system can probably be derived to represent a given flow situation with less variability than might occur due to the lack of reproducibility. In other words, this mathematical model is as reliable as the experimental data itself. The finite stage model has physical significance and is very versatile in representing highly skewed to relatively flat age distribution curves. This model can be used to predict expected treatment efficiency if the representative C-diagram is available and the mean detention time is known for the proposed pond design.
The data indicate that waste stabilization ponds should be constructed with large values of \( L/W \) ratio. This will give a maximum detention time and the minimum value for dead space which will result in greater treatment of the wastewater. Large values of \( L/W \) can be accomplished effectively and economically by baffling a large rectangular pond as has been shown in this study. Also, the data indicate that certain types of inlets and outlets and their placement such as Experiments O-4 and O-2, are obviously poor for maximum efficiency of pond operation and should not be used in constructing future wastewater ponds.
CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

Based on the experimental work conducted on existing waste stabilization ponds in the Logan pond system and on the hydraulic model, the following conclusions are made:

1. Short circuiting in existing ponds in the Logan pond system occurs to varying degrees depending on the environmental conditions, the hydraulic characteristics, and the geometry of the pond. The experiments on the Logan City ponds revealed that short circuiting can be greatly influenced by wind. The types of inlets or outlets and their location in the pond can also directly influence the degree of short circuiting, particularly if the outlet is downwind and the inlet is upwind of the prevailing winds. Density stratification also effects the flow patterns and the degree of short circuiting in wastewater ponds. Density differences between the inlet flow and the ambient fluid can cause the main flow to be in the upper or bottom half of the pond which results in more serious short circuiting because of the reduced effective volume of the pond.

2. The most important consideration in verifying the pond behavior for the unstratified flow situation is the variable behavior of the inflow jet. Proper inflow jet modeling is important if model behavior is to duplicate prototype behavior.
3. For duplicating pond behavior where stratified flow exists, no firm conclusion has been reached. More reliable prototype data must be obtained in order to fully verify the use of the densimetric Froude number as a modeling criteria. However, the preliminary results look promising.

4. Reynolds number changes had little effect on the significant parameters and on the age distribution functions as long as the flow was turbulent. This conclusion assumes that the inflow jet is well diffused.

5. The degree of depth distortion did have an influence on the values of $\theta_c$, $\theta_{p'}$, and $\bar{V_d}$. A change of about 30 percent was indicated by the data for $\theta_{p'}$, but in comparing treatment efficiencies, influence of depth resulted in only about a 6 percent change.

6. Inlet and outlet types and configurations had a significant effect on the hydraulic characteristics and subsequent treatment efficiency determinations. A change of as much as 42 percent was indicated by the data for the important hydraulic parameters, while a change of 19 percent in treatment efficiency was indicated for the two extreme conditions of inlets and outlets.

7. The length-to-width ratio $L/W$ has the greatest influence on the overall efficiency of waste stabilization ponds. Large $L/W$ ratios created by baffling were the best for maximum hydraulic performance.

8. The degree of stratification had an influence on the hydraulic performance. For an increase in the degree of stratification, i.e.,
decreasing densimetric Froude numbers, the influence on the hydraulic parameters and treatment efficiency depended on the type of stratification. When $\rho_{in} > \rho_{pond}$ the hydraulic performance improved. When $\rho_{in} < \rho_{pond}$ the hydraulic efficiency decreased.

9. The biological treatment efficiency can be predicted for a wastewater pond if a reliable value of $K$, the first order reaction coefficient, and a C-diagram are available.

10. The finite stage model represents well the age distribution function of a flow system. The model derivation depends on reliable values for $\theta_b$, $\theta_{pk}$, $(c/c_o)_{pk}$, and a representative C-diagram. The finite stage model can also be used for predicting pond performance for proposed designs.

11. The most significant conclusion of the research is that the hydraulics of waste stabilization ponds are important in determining the treatment efficiency of a pond. The hydraulics should be considered in proposed pond designs to insure maximum economy of construction and operation for maximum conversion of the waste material.

The following are recommendations for extensions of this study.

1. Obtain more reliable prototype data. This could be accomplished by performing tracer experiments on smaller ponds.

2. Obtain a considerable amount of tracer study data on model ponds utilizing different baffling schemes in increasing the effective L/W ratio. Also, obtain tracer data on odd shaped ponds such as (1)
triangular ponds, (2) circular ponds and others.

3. Determine the influence of the stabilizing time on the shape of the C-diagram for density-stratified flow experiments.

4. Obtain more data for a greater range of densimetric Froude numbers.

5. Determine mathematical equations for given pond designs from hydraulic model data and existing pond data for many different pond designs to be used for future pond designs for predicting treatment efficiency.
SELECTED REFERENCES


PROGRAM NI - CALCULATIONS FROM THE C AND F DIAGRAMS

C CALCULATION OF THE DIMENSIONLESS C-DIAGRAM AND F-DIAGRAM FROM THE
C EXPERIMENTALLY DETERMINED CONCENTRATION VS TIME CURVES FROM TRACER
C STUDIES. THE MEAN RESIDENCE TIME, PLUG FLOW DEVIATION PARAMETER,
C DEAD SPACE, VARIANCE, AND THIRD MOMENTS

INTEGER CC(250), C(250), TBAR(250), F(250)

INTEGER Y, DS

10 READ (5, 80) DATA

WRITE (6, 81) DATA

READ (5, 90) NN, N1, N2

READ (5, 91) (CC(I), I=1, N1)

READ (5, 92) D*PL*O*TPE*AK*WR*IE

READ (5, 93) DELT*DC*CO*VISC

DT = 6 T = 1 + N1

6 C(I) = CC(I)/CO

CDT = 7.48*D*PL*W/Q

DT = DELT/CDT

DC1 = DC/CO

M = N1 + 1

IF (M > N) GO TO 14

DO 5 J = M + NN

C(J) = C(J-1) - DC1

IF (C(J) < 0.0) GO TO 11

GO TO 5

11 N = J - 1

GO TO 13

14 N = NN

13 CONTINUE

C MEAN DETENTION TIME, PLUG FLOW DEVIATION PARAMETER, VARIANCE, AND
C THIRD MOMENTS FROM THE C-DIAGRAM

X = 0.0

SUM = 0.0

TH = 0.0

THD = 0.0

VAR = 0.0

DO 7 J = 1, N

X = X + C(J)

SUMC = SUM + C(J)

TH = TH + SUMC

THD = THD + SUMC*SUM

VAR = VAR + SUMC*SUMC

IF (J.EQ.N2) GO TO 30

31 TBAR(J) = SUM

GO TO 2

70 TX = TH/X

DTHM = 1 - TX

TH0 = THD/X

THDM = THD + TX*(2.0*TX - TX - 3.0*VAR/X)

THDI = 1 - (3.0*TH - 3.0*VAR + THD)/X

GO TO 31
SU M = SUM + DT
THM = TH/X
VARM = VAR/X - THM * THM
ATBAR = THM * CDT
RE = (272.816/W) * Q/VISC
TBGN = BIE N/C DT
THETPK = TPEAK/C DT
24 CONTINUE
C F-DIAGRAM CALCULATIONS
K = N + 1
L = N + 2
C(K) = C(N)
C(L) = C(N)
F(I) = 0.0
DO 3 I = 2, N
3 F(I) = DT * (.54166667 * (C(I) + C(I + 1)) - .04166667 * (C(I - 1) + C(I + 2))) + F(I - 1)
C DEAD SPACE CALCULATION
DSP = 1.0 - THM * F(N)
WRITE (6, 95) RE, CDT, ATBAR
WRITE (6, 96) TBGN, THETPK, THM, THM, DSP
WRITE (6, 97) TH00, THDM, TH1
WRITE (6, 79)
WRITE (6, 99) (C(I) * I = 1, N)
WRITE (6, 94)
WRITE (6, 99) (TBAR(I) * I = 1, N)
WRITE (6, 102)
WRITE (6, 99) (F(I) * I = 1, N)
CONTINUE
IF (Y.EQ.0.0) GO TO 12
Y = Y + 1
GO TO 10
12 CONTINUE
C FORMAT STATEMENTS
79 FORMAT ('/2X*DIMENSIONLESS C/O VALUES AS A FUNCTION OF DIMENSION
LESS TIME RATIO*')
80 FORMAT (A6)
81 FORMAT ('/2X*DATA SET*2X, A6/)
90 FORMAT (3I5)
91 FORMAT (8F10.5)
92 FORMAT (6F10.2)
93 FORMAT (F10.5, 3F10.7)
94 FORMAT ('/2X*DIMENSIONLESS TIME RATIO*')
95 FORMAT (2X*REYNOLDS NO=*, F8.2, 5X*THEOR MEAN=*, F8.3, 5X*ACTUAL MEAN
IN TWO DETENTION TIMES) =*, F8.3)
96 FORMAT (2X*DIM REG TIME=*, F8.5, 5X*DIM TO PEAK=*, F8.5, 5X*DEPARTU
RE FROM PLUG FLOW=*, F8.5, 2X*DIM MEAN(TWO DET TIMES) =*, F8.3, 5X*DE
AD SPACE =*, F8.5/)
97 FORMAT ('/2X*3P-MOMENT (ABOUT ORIGIN) =*, F8.5, 5X*3RD-MOMENT (ABOUT MEAN) =*, F8.5, 5X*3RD-MOMENT (ABOUT T/TPAR = 1.0) =*, F8.5/)
99 FORMAT (13F10.5)
102 FORMAT ('/2X*F-DISTRIBUTION VALUES*'/)
STOP
END
**PROGRAM**

**N7-TREATMENT EFFICIENCY CALCULATION**

**C** Program to determine reaction of a waste given the first order reaction coefficient, \( K \), and the dimensionless \( C/CO \)

**Diagram**

- **DIMENSION** \( C(200) \), \( CC(200) \), \( E(200) \)
- **REAL** \( K(5) \)
- **INTEGER** \( W, DS \)
- \( W = 1 \)
- \( DS = 7 \)
- \( KK = 5 \)
- \( TOET = 5.0 \)
- \( K(1) = 0.20 \)
- \( K(2) = 0.25 \)
- \( K(3) = 0.30 \)
- \( K(4) = 0.35 \)
- \( K(5) = 0.40 \)

1. \( L = 0 \)

   READ \((5, 80)\) DATA

   WRITE \((6, 81)\) DATA

   READ \((5, 90)\) \( NN, CDT \cdot DT \cdot CO \)

   READ \((5, 83)\) \((C(T), I = 1 \cdot NN) \)

   \( DT = D T / C D T \)

2. \( L = L + 1 \)

   **SUM** = 0.0

   DO 2 \( J = 1 \cdot NN \)

   \( CC(J) = C(J) / CO \)

   \( E(J) = \exp (- K(L) \cdot TOET) \cdot SUM \)

3. **SUM** = **SUM** + **DT**

   **TEF** = 0.0

   DO 4 \( J = 1 \cdot NN \)

   \( EAV = (E(J) + E(J+1)) / 2.0 \)

   \( CAV = (CC(J) + CC(J+1)) / 2.0 \)

4. **TEF** = **TEF** + **EAV** \( \cdot **CAV** \cdot **DT**

   WRITE \((6, 100)\) \( K(L) \cdot **TEF**

   IF \((L \cdot EQ. KK) \) GO TO 10

   GO TO 3

10 CONTINUE

   IF \((W \cdot EQ. DS) \) GO TO 15

   \( W = W + 1 \)

   GO TO 1

15 CONTINUE

80 FORMAT \((A 6)\)

81 FORMAT \(/ / / / / / 2 X \cdot * \) DATA \( 5 F 10.6 \)

83 FORMAT \((8 F 10.6)\)

90 FORMAT \((I 5 \cdot 3 F 10.5)\)

100 FORMAT \((5 X \cdot * K(1ST ORDER REACTION COEFFICIENT) = * * F )0 \cdot 5 \cdot 5 X \cdot * N \) \( / \) \( \) 

   Format of untreated waste at the outlet = \(* * F 10.5 / \) 

   STOP
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PROGRAM N3-FINITE STAGE MODEL DETERMINATIONS

C PROGRAM TO MATHEMATICALLY MODEL A FLOW SYSTEM WITH A FINITE STAGE MODEL
C THE INPUT PARAMETERS ARE DIMENSIONLESS C-DIAGRAM, DEAD SPACE, BEG TIME
C OF TRACER TIME TO PEAK, PEAK VALUE OF C, AND K=F(N)

REAL K(5)
DIMENSION C(200), TBAR(200), X(200), Y(5,200), Z(5), CC(200), F

1AA(5), CD(200)
INTEGER W*DS
W=1
DS=6

30 READ (5,80) DATA
WRITE (6,81) DATA
READ (5,90) NN, FC, CD*T, DELT, CO
READ (5,91) (K(I), I=2,5)
READ (5,91) FFAA(I), I=2,5
READ (5,83) (C(I), I=1,NN)
DT=DELT/CDT
XC=1.-FC
SUM=0.0
DO 40 J=1,NN
CD(J)=C(J)/CO
TBAR(J)=SUM

40 SUM=SUM+DT

30 TRANSFORMATION OF C AND TBAR FROM C-DIAGRAM
DO 50 I=1,NN
DX=TBAR(I)-FC
IF(DX*GT.0.0) GO TO 51

50 CONTINUE

51 L=I-1
NNN=NN-L
DO 52 J=1,NNN
NK=J+L
XT(J)=(TBAR(NK)-FC)/XC

52 YT(J)=CD(NK)*XC

30 FINITE STAGE MODEL CALCULATIONS
N=1
10 N=N+1
FA=FFAA(N)
FB=1.-FA
T=(1.*K(N))/FA+K(N)/FB
R3=SUM((T-T-4.*K(N))/(FA*FB))
R2=5.*(T-R3)
R1=5.*(T+R3)
E1=R1+K(N)/FB
E2=R2*K(N)/FB
E3=F*A*R3*R3
A11=E1/(FA*R3)
A22=A11**2.0
B11=1./FA-A11
R22=P11*B11
A21=((1./FA)**2.0-A22-B22)/R3
R21=-A21
Z(N)=0.0
IF (N.EQ.2) GO TO 11
A33=A11*3.0/7
A32=-3.*A22*E7/E3
B33=R22/2.0
B32=-3.*B22*E1/E3
A31=(-A32-B32)/R3
B31=-A31
IF (N.EQ.3) GO TO 12
A44=A22*2.0/5.
A43=7.*A32*A11/3.0
A42=-3.*A32*(7.*E1+3.*E2)/(3.*E3)
B44=R22*2.0/6.0
B43=7.*B32*B11/3.
B42=-3.*B32*(7.*E2+3.*E1)/(3.*E3)
A41=-A42-B42)/R3
B41=-A41
IF (N.EQ.4) GO TO 13
A55=A44*A11/4.0
A54=5.*A43*A11/12.
A53=-5.*A43*(D3*3.*E2)/(4.*E3)
A52=5.*A32*(2.*E2*E2+4.*E1*E2+1*E11)/(3.*E3*F3)
B54=5.*R43*B11/12.
B53=-5.*B43*(-R3+3.*E1)/(4.*E3)
B52=5.*B32*(2.*E1*E1+4.*E1*E2+E2*E2)/(3.*E3*E3)
A51=-A52-B52)/R3
B51=-A51
IF (N.EQ.5) GO TO 14
11 DO 1 J=1,NNN
Y(2,J)=2.*((A21+A22*7.*X(T(J)))*EXP(2.*R1*X(T(J)))+(B21+B22*(2.*X(T(J))))
1 Z(2)=7(2)+Y(2,J)-Y(T(J)) ** 7.0
GO TO 10
12 DO 2 J=1,NNN
Y(3,J)=3.*((A31+A32*3.*X(T(J)))+A33*(3.*X(T(J))**2.0)*EXP(3.*R1*X(T(J))
2 Z(3)=7(3)+Y(3,J)-Y(T(J)) ** 7.0
IF (Z(3) .GT. Z(2)) GO TO 5
NP=NP+1
GO TO 6
5 NP=NP-1
6 CONTINUE
GO TO 10
13 DO 3 J=1,NNN
X4=4.*X(T(J))
Y(4,J)=4.0*(A41*A42*X4+A43*X4+X4*X4+X4*X4+X4*X4+3.*I)*EXP(X4*R1)+B41+B42
3 Z(4)=7(4)+Y(4,J)-Y(T(J)) ** 7.0
IF (Z(4) .GT. Z(NP)) GO TO 7
NP=NP+1
7 CONTINUE
GO TO 10
14 DO 4 J=1,NNN
X5=5.*X(T(J))
Y(5,J)=5.*(A51+A52*X5+A53*X5+X5*A54*X5**3.0*A55*X5**4.0)*EXP(X5*R1)+B51+B52
4 Z(5)=7(5)+Y(5,J)-Y(T(J)) ** 7.0
IF (Z(5) .GT. Z(NP)) GO TO 8
NP=NP+1
8 CONTINUE
GO TO 10
1) *(R51*B52*B53*B54*B55*B56*B57*B58*B59*B60)*EXP(X5*R2)
4) Z(5)=Z(5)+(Y(5,J)-YT(J))*Z(J)
IF (Z(5),GT,Z(NP)) GO TO 8
NP=N
8 CONTINUE
FBL=FAA(NP)*XC
FBL=(1.-FAA(NP))*XC
WRITE(6,92)
WRITE(6,93)NP,K(NP),FBL,FBL,FC
WRITE(6,101)
WRITE(6,102)
WRITE(6,100)YT(NP,J),J=1,NNN)
WRITE(6,102)
WRITE(6,100)XT(J),J=1,NNN)
DO 55 J=1,NNN
55 CC(J)=Y(NP,J)/XC
WRITE(6,105)
WRITE(6,103)
WRITE(6,100)CC(J),J=1,NNN)
WRITE(6,104)
WRITE(6,100)TBAR(J),J=1,NNN)
CONTINUE
IF (W.EQ.05) GO TO 15
W=W+1
GO TO 30
15 CONTINUE
C FORMAT STATEMENTS
80 FORMAT(A6)
81 FORMAT(///2X,*DATA SET*2X,AA/)
83 FORMAT(9F10.6)
91 FORMAT(4F10.5)
92 FORMAT(///2X,*TRANSFORMED THEORETICAL C-DIAGRAM BASED ON FINITE STAGE MODEL*)/
93 FORMAT(2X,*N=*I5,*2X,*K=,*F8.5,*2X,*FA=,*F8.5,*2X,*FB=,*F8.5,*2X,*FC=*
1.*F8.5/*)
100 FORMAT(13F10.5)
101 FORMAT(2X,*TRANSFORMED C VALUES AS A FUNCTION OF TRANSFORMED TIME RATIO(FINITE STAGE MODEL)*/)
102 FORMAT(2X,*TRANSFORMED TIME RATIO VALUES(FINITE STAGE MODEL)*/)
103 FORMAT(2X,*C VALUES AS A FUNCTION OF TIME RATIO(FINITE STAGE MODEL 1)*/)
104 FORMAT(2X,*TIME RATIO VALUES(FINITE STAGE MODEL)*/)
105 FORMAT(///2X,*THEORETICAL C-DIAGRAM BASED ON FINITE STAGE MODEL*/)
STOP
VITA

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