TREATMENT AND DESIGN CRITERIA OF A THREE-CELL SUBMERGED TUBE AERATION SYSTEM AT THE BROWNFIELD WASTEWATER TREATMENT PLANT

by

RICHARD M. OLLER, B.S.

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CHAPTER I

INTRODUCTION

In 1977 the City of Brownfield, Texas received a Step 2 Plus 3 (design plus construction) Construction Grant from the Environmental Protection Agency. This grant was for design and construction of a new wastewater treatment plant. The selection of a treatment process was based on secondary treatment criteria and on a limitation of available land space for the new facility. An aerated lagoon system using submerged tube aerators was chosen to replace the Imhoff tank primary treatment system.

Construction of the new system was completed in 1982. The aeration system which was installed was supplied by Hinde Engineering Company from Chicago, Illinois. The plant design included a new main lift station; inflow measuring device; comminutor; non-aerated, rectangular grit chamber and three aerated ponds arranged in series.

The wastewater in Brownfield is a typical domestic wastewater with no industrial contributors. The domestic water is primarily from the Canadian River System and is supplemented by local wells. Typical values of hardness in the influent to the plant are 300 to 500 mg/l, as $CaCO_3$ hardness. Alkalinity in the influent is in the approximate range of 160-200 mg/l.

The design influent flow rates to the plant were estimated by recording the time of pump operation at two lift stations in the city. These recordings were compared to existing population census data. Design flow rates were based primarily on estimated population projections from an arithmetic plot of past population census reports.

The design population was established as 12,500 persons. The design period was based on a 20-year design life. This allows plant capacity to be sufficient until the year 1995. The estimated design flow is 1.25 million gallons per day (1).

Wastewater characteristics were not determined for treatability. No historical data existed for flow, organic loads, suspended solids, dissolved oxygen, or any other quantitative or qualitative parameter. Therefore, assumed values stipulated by the Texas Department of Water Resources (TDWR) in their publication entitled, Design Criteria for Sewerage Systems (44) were used for design purposes. The values used in the actual design of the wastewater system were 200 mg/1 BOD_5 and 200 mg/1 suspended solids.

In selecting the most feasible treatment alternative, the following criteria had to be met: (1) the new facility had to be located out of the 100 year flood plain; (2) a rather simplistic treatment system to minimize operations was needed; and (3) the City wanted to locate the plant on available land, already in ownership by the City. These three requirements led to the selection of the three-cell submerged tube aeration system. The limited available land space was the primary factor for this choice.

The treatment plant receives water from a main lift station built in this project. The wastewater enters the treatment plant via an open channel. (See Figure 1.1.) The water is channeled through a continuously operating, 180 degree, sweep-comminutor, to a non-aerated, rectangular grit chamber. Wastewater flows over the grit chamber weir to a collection trough leading to a splitter box. The influent flow can

be divided in this splitter box, directed either to pond 1 or to a transfer structure located in the common dike between ponds 1 and 2. The normal flow path is into pond 1. Pond 1 is designed for a 9-day detention time, and ponds 2 and 3 both have a 4.5 day detention time (1).

Air is provided by one or more of four positive displacement blowers and is delivered to the ponds by a network of slotted polyethylene tubes, weighted and laying on the bottoms of all the ponds. These tubes are spaced close together at the influent end of pond 1. The spacing of the tubes gradually increases toward the effluent end of pond 1. The same arrangement scheme is repeated in ponds 2 and 3. The blowers produce 375 scfm at 12 psig and 60^{o}F. The system generally requires three of the four blowers provided to supply the required oxygen for aeration.

The lagoons are operated as a partially mixed treatment system which allowed substantial savings in the sizing of the blowers and energy costs to the City. The organic removal efficiency requirement is 85 percent (1). A total of 18 days' detention time was required with 15 percent excess volume added to pond 1 for sludge storage (1).

The plant at the time of initiating this study (November, 1982) was not consistently producing an effluent with an organic removal efficiency of 85 percent. Therefore, a reason or reasons exist for the inconsistency in the observed organic removal efficiency.

The primary objective of this study was to identify and suggest corrections for the operational problems associated with the submerged tube aeration system installed in the Brownfield, Texas municipal

wastewater treatment facility. To accomplish the primary objective, the following secondary objectives had to be realized:

1. to compare the assumptions used in design of the system with organic removal rates realized in the operation of the system; and
2. to examine the operational characteristics of submerged tube aeration devices and a system utilizing these devices during the various seasons.

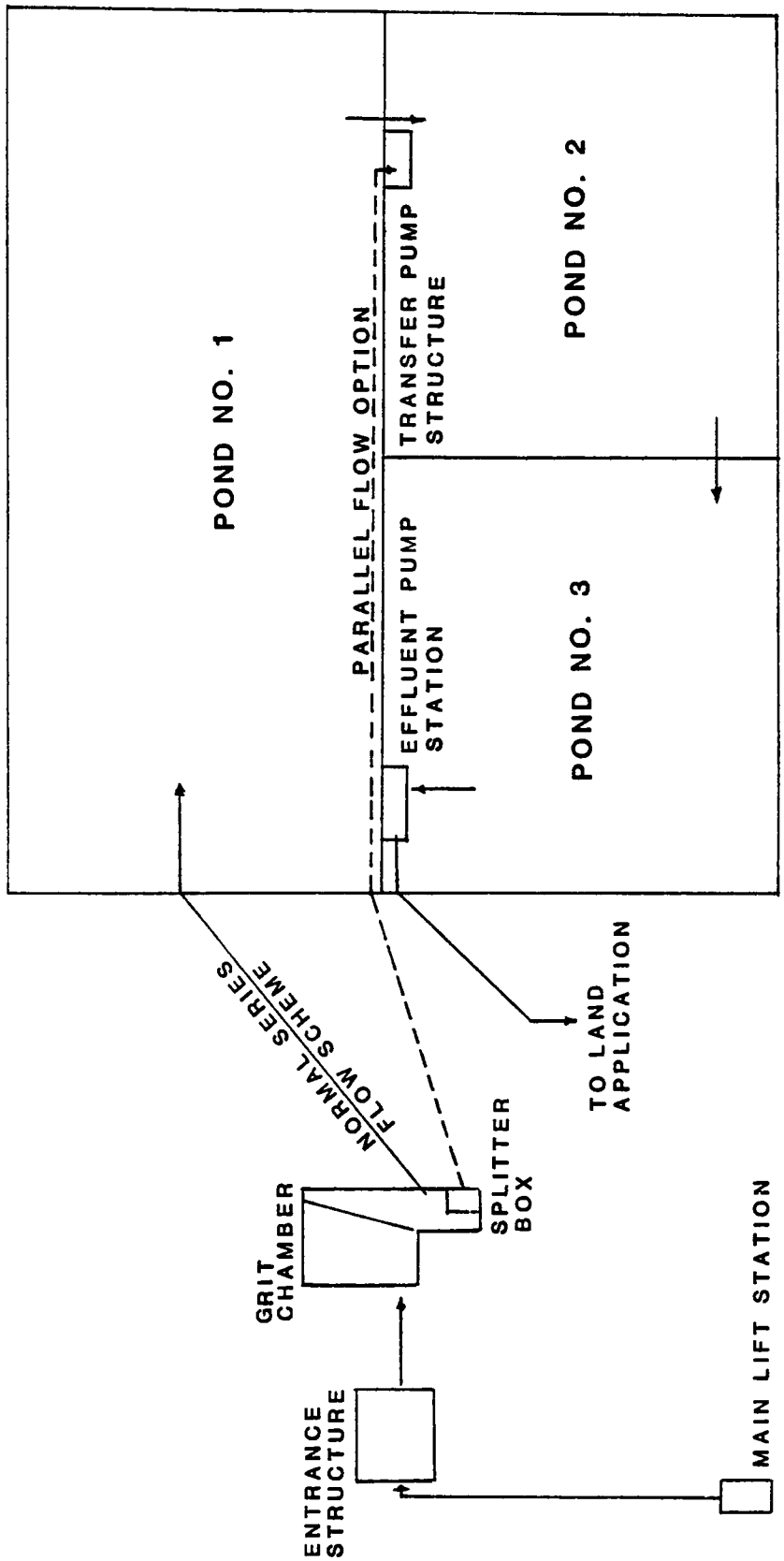


FIGURE 1.1: Schematic flow diagram.

CHAPTER II

BACKGROUND AND REVIEW OF LITERATURE

Aerated Lagoons in Wastewater Treatment

Background

Wastewater treatment has been practiced for over 3000 years. The first treatment facility in the United States, from a historical basis, was in San Antonio, Texas, in 1901 (2). This operation employed a system of ponds in the primary treatment process. Wastewater treatment ponds since then have been used to treat many different types of waste. Industries have utilized stabilization ponds to treat various waste streams. Municipalities in all types of climatic zones continue to use stabilization ponds to treat domestic wastewater.

As knowledge of the operational processes involved with ponds was gained, different pond configurations and applications surfaced. Pond systems are versatile in their treatment applications. They can be used alone or in conjunction with other treatment processes (2). General depths for wastewater ponds are five to twelve feet.

The Manual of Wastewater Operations (7) differentiates ponds from lagoons by defining ponds as having regular and controlled shapes, depths and marginal areas, specifically designed as treatment devices. Lagoons are constructed using natural terrain and dams to form basins whose shapes, depths, and marginal areas are not controlled. From those applications of ponds to wastewater treatment, the concepts of the complete-mix and partial-mix systems developed. Operational characteristics and design equations, along with the necessary design assumptions, require investigation to insure proper application of the

known principles in a specific design situation.

Types of Aerated Lagoons

The two types of aerated lagoons prevalent today are the complete-mix and partial-mix aerated lagoons. The main difference between the two is in the degree of mixing provided for the lagoon contents. Complete-mix lagoons are those that maintain settlable solids in suspension. Partial-mix lagoons maintain only a portion of the suspended solids in suspension (2). The latter system is also known as a facultative aerated lagoon. The differentiation between the two types is shown by the power level to maintain the suspended solids in suspension. Design values for complete-mix systems are 30 hp/10^6 gallons (3), whereas partial-mix lagoons require approximately 1/6 the power requirement of the complete mix system (2). The power requirement for the partial-mix system is to insure adequate dissolved oxygen concentrations throughout the cross section of the lagoon.

The relative advantages of one type of lagoon over the other are based on the amount of land area available on which to locate the specific system. Complete-mix systems can be located on smaller tracts of land as opposed to the partial-mix system. The difference is due to the power requirements for mixing in the two systems. Therefore, a trade off exists between the two systems. The partial-mix system has better oxygen transfer efficiency but requires more land area due to the lower organic surface loading rates.

Two types of aeration processes can be employed in either of the two types of lagoons. Mechanical aeration devices are generally used in a complete-mix process, with diffused air being the predominant type for

partial-mix systems. The mechanical type of aerator agitates the wastewater and air is absorbed into the water droplets from the atmosphere. Diffused air is introduced into the wastewater near the bottom of the reaction basin, bubbled up through the liquid with very fine bubbles to achieve a higher degree of oxygen transfer.

Design Equations for Aerated Lagoons

The United States Environmental Protection Agency (EPA), has funded most of the recent studies that have been conducted on the design of aerated lagoons. One of the most current research articles completed to date is entitled "Design Approach To Dual-Power Aerated lagoons," by Linvil G. Rich (3). Rich has introduced a procedure for the design of complete-mix and partial-mix aerated lagoon systems which utilizes the biokinetic coefficients as the main design parameters. Other factors relating to operation, performance, geometry, number of cells in series, power requirements and effluent quality, BOD_5 and suspended solids are discussed. Rich concludes that further research is needed to better evaluate the parameters used and to test the assumptions made in applying his procedure (3).

System design, as outlined by Rich (3), starts by determining the influent flow rates to the treatment process. Secondly, effluent limitations imposed by the regulatory agency must be identified. From this point, design process assumptions must be made or an extensive wastewater characterization must be completed on the wastewater to be treated. Typical assumed values for BOD_5 and suspended solids are 200 mg/l each for domestic wastewater with no industrial discharges. Rich (by an example problem) has shown that a system design based on the

above loadings, utilizing a completely suspended cell with partially suspended cells following, produces a high-quality effluent. In addition this combination required less land area than a partially suspended cell system. Average values of total BOD_5 and total suspended solids (TSS) in the effluent were approximately 35 mg/l. The combination in which a complete-mix cell is followed by a partial-mix cell produces a lower soluble BOD_5 concentration in the effluent than does a single cell of either type (3).

Design Factors

Aeration system design for aerated ponds follows the basic activated sludge aeration design process. Performance of the aerated pond process is typically characterized by effluent measurements of soluble 5-day 20°C biochemical oxygen demand (BOD_5) and total suspended solids (TSS) (2). Total 5-day 20°C biochemical oxygen demand $(BOD_5)_t$ is a function of the soluble 5-day 20°C BOD and volatile suspended solids concentration (2,3). The $(BOD_5)_t$ (1) is defined as follows:

$$(BOD_5)t = (BOD_5)s + 0.85\ (X_a)n \qquad (1)$$

where:

$(BOD_5)t$, $(BOD_5)s$ = total and soluble 5-day 20°C BOD (mg/l) respectively

X_a = average concentration of biomass (mg/l) in system effluent

n = number of equal-sized, partially suspended cells in series

In a partial-mix system only enough oxygen is provided for oxidizing the influent BOD. In aerated lagoon systems, a long solids retention time

(SRT) is needed to achieve the BOD removal efficiencies required. Typically, a complete-mix, first-stage cell is followed by a lower energy level cell to allow settling to occur. The latter system, a partial-mix, low-energy system, has a minimum of two cells following the primary cell. The organic solids that settle are stabilized by anaerobic action in the sludge zone. Due to the anaerobic stabilization in these systems, partial-mix systems are frequently referred to as a facultative aerated pond system (2).

First-order biokinetics and plug-flow hydraulics more closely predict the performance of partial-mix systems (2). Design for the partial-mix systems is driven by the complete-mix systems. The key factor in the design of the partial-mix system is selection of the reaction rate coefficient (K_{pm}) (2). The "Ten State Standards" recommends K_{pm} values of 0.276 $days^{-1}$ at 20°C and 0.138 $days^{-1}$ at 1°C (47). Using these two values, a temperature coefficient, θ, with a value of 1.036 is calculated. Boulier and Atchison (5) have recommended K_{pm} values of 0.2 and 0.3 at 20°C and 0.1 to 0.15 at 1°C (5). Reid (2) recommended K_{pm} values of 0.28 at 20°C and 0.14 at 0.5°C, based on research conducted on aerated ponds in central Alaska employing submerged tube aeration.

Manufacturers' literature (4, 29, 34) suggests $K_{pm} = K_e$ values corresponding to Reid's findings for domestic sewage. Therefore, ranges are found for the terms of 0.1 to 0.15 at 0.5°C and of 0.2 to 0.3 at 20°C. Hinde Engineering uses 0.14 for domestic sewage if a specific K value is not specified (4). K values are governed by water temperature. The effect of temperature on K values (2) can be predicted by:

$$K_D = K_{20^oC} (1.036)^{T-20} \quad (2)$$

where:

K_D = reaction rate at design temperature ($days^{-1}$)

K_{20} = reaction rate at 20^oC = 0.276 $days^{-1}$

T = temperature of pond water (^oC)

Rich has concluded that K_d is derived from the following equation (3):

$$K_d = K_{d20} (1.05)^{T-20} \quad (3)$$

At temperatures greater than 20^oC, the values for K_d did not differ significantly from those at 20^oC.

Rich further concludes that the most important parameters guiding the design of aerated multicellular lagoons are biokinetic coefficients. He recommends that bench scale testing be done for 30-day periods in the dark at room temperature to assess the biokinetic coefficients and adjust them to meet the waste stream's conditions.

Rich points out that the performance of multicellular aerated lagoons are influenced greatly by the number of cells in series, the power level in each, the geometry of the cells, the suspended solids carried over from each preceding cell, benthal solids decay, sludge accumulation, soluble BOD_5, oxygen demand and the power requirements for the cells. Rich's research has concluded that the best arrangement is a completely mixed first cell, followed by three partially mixed cells. The recommended geometry is an inverted truncated pyramid approximately three meters deep at the center with 3:1 side slopes. Power levels should be greater than or equal to 6 W/m^3 for the completely mixed cell

and approximately 1 W/m^3 for the partially suspended cell.

Rich contends that retention times frequently are too long and cause an increase in BOD in the effluent. Rich recommends that retention times be only long enough to provide for the BOD removal required and the deposition and storage of solids. Additional depths of lagoons beyond three meters, or approximately ten feet, for increased storage of solids are not conducive to improved performance. In fact, it has been shown in such designs to increase the soluble BOD_5 in the effluent (3).

Detention Time

The average detention time of wastewater within one cell in a series of lagoons is determined for the worst possible environmental conditions--the coldest expected (predictable) operating temperatures. This is required since biological activity, and corresponding reaction rates, are slowed with decreases in pond water temperatures. The required detention time (4) can be calculated by:

$$t = (n/K_D) [(C_o/C_n)^{1/n} - 1] \quad (4)$$

where:

t = detention time (days)

n = number of cells in series

K_D = reaction rate for design temperature ($days^{-1}$)

C_o = influent BOD_5 (mg/l)

C_n = effluent BOD_5 from the nth pond in an n-pond series (mg/l)

$t_1 = t_2 = t_3 = t_4 \ldots = t_n$

Typical depths for partial-mix ponds range from 1.5 to 4.5m (5 to

15 feet). A 3-meter depth (10 feet) is the average generally cited (2, 3).

Oxygen Requirements

The design of the partial-mix system is the same as for a complete-mix design. The Mancini and Barnhart equation for oxygen requirements (5), is stated as follows:

$$N = \frac{N_a}{\alpha \left[\frac{C_{SW} - C_L}{C_S} \right] (1.025)^{T_W - 20}} \qquad (5)$$

where:

N = equivalent oxygen transfer to tapwater at standard conditions

N_a = oxygen requirement to treat the wastewater (kg/hr)

$\alpha = \frac{\text{oxygen transfer in wastewater}}{\text{oxygen transfer in tapwater}}$ (usually 0.9)

C_L = minimum D.O. concentration maintained in the waste (usually 2 mg/l)

C_S = oxygen saturation value of tapwater at 20°C and one atmosphere pressure (9.17 mg/l)

T_W = wastewater temperature (°C)

$C_{SW} = (C_{SS})P$ = oxygen saturation value of the waste (mg/l)

$\beta = \frac{\text{wastewater oxygen saturation value}}{\text{tapwater oxygen saturation value}}$ (usually 0.9)

C_{SS} = tapwater oxygen saturation value at temperature, T_W (mg/l)

P = ratio of barometric pressure at plant site to barometric pressure at sea level (assume 1.0 for the elevation of 100 m)

Maximum oxygen requirements must be provided during the summer months. Hinde Engineering (4) suggests an equation for oxygen requirements similar to the Mancini and Barnhart equation. Hinde's air supply equation is:

$$FTR = LTR\ [(\ C_{DC} - DO/C_{DL}\ @\ 20^{o}C\ \&\ \text{sea level})\ (\theta)^{T-20}\ ] \qquad (6)$$

where:

FTR = Field transfer rate for O_2 (mg/1)

LTR = O_2 transferred under laboratory conditions - standard conditions = Salinity-Surface tension correction factor which usually ranges from 0.95 to 1.0

β = Beta, salinity-surface tension correction factor (usually 0.95-1.0)

C_{DC} = O_2 saturation concentration at design temperature and water level mid-depth (mg/1)

DO = average operating O_2 concentration (usually given as 2 mg/1)

θ = Temperature correction factor (usually 1.024)

T = Design maximum water temperature (^{o}C)

α = O_2 transfer correction factor for dirty water as recommended by Hinde to be in the range of 0.85 to 0.95.

The minimum design oxygen requirement suggested by Hinde Engineering (7) is:

$$O_2 = 2\ X\ IBOD_5 \qquad (7)$$

where:

$IBOD_5$ = Influent BOD_5

Applying the Hinde Engineering oxygen requirement equation requires some manipulation of the variable C_{DC}. The Hinde system uses a submerged-tube, fine-bubble diffuser. The air bubbles are formed at the bottom of the lagoon and released to travel to the water's surface. Because the bubbles are exposed to a constantly varying external water pressure and the solubility of oxygen is directly related to this pressure, the C_{DC} value must reflect a pressure increase. The mid-depth value must be used for C_{DC} or significant errors in calculation of the field transfer efficiency will result (2, 3, 4). Table 2.1 gives the C_{DC} values recommended by Hinde Engineering. The C_{DC} values (8) in the table were estimated using the following equation:

$$C_{DC} \text{ (mid-depth)} = C_S \, [P + 0.5X/760] \tag{8}$$

where:

C_s = the standard methods DO saturation value at design temperatures (mg/l)

P = barometric pressure at site (mm Hg)

X = Hydrostatic head above air release point (mm Hg)

Aeration requirements determined by other researchers have utilized different equations to determine the oxygen needs. Assuming nitrification does not occur, the maximum rate at which oxygen will be required in a complete-mix cell is estimated using the following equation (3):

$$RO_2 = 4.16 \times 10^{-5} \, r \, [1.47Q(S_0-S_1) - 1.42QX_1] \tag{9}$$

where:

The term 1.47 is used in the equation to convert the BOD_5 terms to first-stage values.

The 1.42 is an assumed oxygen-equivalent ratio for biomass.

The term 4.16×10^{-5} is a conversion factor changing g/d to kg/hr.

Other terms in the equation are as follows:

Q = wastewater flow rate (m^3/day)

r = ratio of maximum oxygen requirement to the average requirement (mg/l)

RO_2 = oxygen requirement rate (kg/hr)

S_0 = total BOD_5 of influent wastewater (kg/hr)

S_1 = soluble BOD_5 in effluent of completely suspended cell (mg/l)

X_1 = biomass concentration in completely suspended cell (mg/l)

To use the oxygen requirement rate equation, X_1 must be known. The biomass concentration can be estimated by:

$$X_1 = Y(S_0-S_1)/1 + k_d t_s = FY(S_0-S_1) \quad (10)$$

where:

Y = growth yield (dimensionless)

k_d = specific decay rate (day^{-1})

t_s = solids age (days)

F = solids decay factor

Combining equation 9 by Rich for the oxygen requirement rate, and equation 10 for the biomass concentration yields:

$$RO_2 = 4.16 \times 10^{-5} [1.47QY(S_{j-1}-S_j) + 1.42V_j K_d X_j] \quad (11)$$

where:

Other terms are as previously defined, and (subscript (j) identifies the cell in question)

Rich (3) cites the importance in accounting for the benthal oxygen demand. He concludes that the benthal demand is exerted at the sludge-water interface. However, the decomposition of solids in the bottom of the partial-mix pond will release soluble organics that exert a delayed oxygen demand.

Rich recommends that the following equation be used to estimate the rate of benthal oxygen requirement:

$$RO_2 = 4.16 \times 10^{-5} A_j B\theta^{T-20} \quad (12)$$

where:

A_j = bottom area of each partially suspended cell (m^2)

B = rate of benthal oxygen demand (g O_2/m^2-day)

θ = temperature sensitivity coefficient (1.05)

T = cell water temperature (^oC)

Uniform dissolved oxygen concentration throughout the cross-section of a cell is the key to proper treatment. To estimate the oxygen transfer required the following equation is utilized by Hinde Engineering (4):

$$SCFM = \frac{O_2 \text{ required (lbs/day)}}{\frac{1440 \text{ min}}{\text{day}} \times \frac{0.75 \text{ lbs.}}{\text{cf. air}} \times 0.232 \times FTR} \quad (13)$$

where:

SCFM = standard cubic feet per minute (air flow rate)

0.232 = % O_2 in air at standard conditions

FTR = Field transfer rate of O_2 (lbs/min)

In addition to the above equation, benthal-oxygen requirements must be satisfied by the aeration equipment. This benthal demand is exerted at the sludge-water interface in partial-mix cells. To estimate the benthal oxygen demand, the following equation has been proposed (3):

$$RO_2 = 4.16 \times 10^{-5} A_j B \theta^{T-20} \quad (14)$$

where:

B = rate of benthal oxygen demand (g O_2/m^2-day)

Note: To convert from g O_2/m^2-day to lbs. O_2/ft^2-day, multiply (B) by 2.02×10^{-4}.

To maintain effective dissolved oxygen dispersion, a fluid turnover time of 15-20 minutes or less has typically been recommended by Rich (3). System turnover rate can be estimated by (2):

$$T_{to} = \frac{1440\ V_t}{L\ K_p} \quad (15)$$

where:

T_{to} = fluid turnover time (minutes)

V_t = total system volume (million gallons)

L = length of aeration tube (ft)

K_p = pumping rate (MGD/ft)

1440 = minutes per day

Hinde Engineering (4) recommends a pumping rate of 0.086 MGD/Ft. of

aeration tubing, a design turnover time of 15 minutes, and an air feed rate of 2.7 scfm/200 ft. of aeration tubing. The scfm recommended by Hinde (4) to maintain proper mixing can be calculated by:

$$SCFM = \frac{(1440)(V_t)(0.027 \text{ scfm/ft.})}{T_{to}\, K_p} \tag{16}$$

$$= \frac{(1440)(V_t)(0.027 \text{ scfm/ft.})}{(15 \text{ min.})(0.086 \text{ MGD/Ft.})} \tag{17}$$

$$= (30.14)(\text{Total System Volume}) \tag{18}$$

Again, Hinde (4) recommends the design air requirement is the larger of the two calculated system parameters (oxygen transfer and mixing). In general, the air required to maintain uniform oxygen dispersion will dictate the design air requirement (4).

The interaction of all of the design factors such as reaction rates, temperature, mixing detention times, turnover rates and oxygen transfer rates are important in the design of an aerated pond system. The key difference between a complete-mix system versus a partial-mix system lies in the power requirement of the aeration equipment. Additional power is required with the complete-mix system to totally suspend the biomass in the cell. However, the foregoing equations used to determine the design factors are applicable to both type systems.

Sludge Accumulation

Sludge is a key parameter to be dealt with in a partial-mix system. The majority of the sludge material settles out in the first partial-mix cell. As solids settle out in the first partial-mix cell, the power

level (H_p/ft^3) in the cell will increase as a result of a reduced water volume. When the power level increases above a threshold level, some of the solids will remain in suspension and will be carried over to the second partially suspended cell, where the power level will be such that the settlable solids will settle (3). This phenomenon typically occurs with mechanical surface aerators. Diffused aeration, as used in partial-mix systems, provides enough oxygen for the oxygen demand, and the power input into the pond will not typically cause an increase in solids suspension.

The objective, then, is to predict solids accumulation in the first partial-mix cell. The decomposition of the bottom sludge enables the prediction of annual carryover of sludge for each cell in the system. Therefore, solids loading rates should not exceed the rate at which solids decomposition occurs over the annual cycle (2, 3).

The loading of biodegradable solids on the first partial-mix lagoon can be estimated by (3):

$$L = \frac{YS_0Q}{nA_j[1 + k_d(V/Q)_1]} \tag{19}$$

where:

L = biodegradable solids loading on partially suspended cells (g/m^2-day)

A_j = bottom area of each partially suspended cell (m^2)

k_d = specific decay rate (day^{-1})

$(V/Q)_1$ = hydraulic retention time in completely suspended pond (days)

n = number of equally sized, partial-mix cells in series

S_0 = total BOD_5 influent wastewater (mg/l)

Y = growth yield coefficient (dimensionless)

Q = wastewater flow rate (m^3/day)

For systems where no sludge carryover occurs on an annual basis, the following equation can be used to estimate the nonbiodegradable fraction (3):

$$V = \frac{365\ Q\ (X_{oi} + 0.23\ YS_o)}{xp} \tag{20}$$

where:

X_{oi} = concentration of inert fraction of solids in influent wastewater (mg/l)

x = weight fraction of solids in sludge

p = water density (g/m^3)

The above equation is at best an approximation for sludge accumulation. Each system must be evaluated individually to predict the sludge accumulation occurring within the primary pond.

It has been suggested that one can expect a complete destruction of such solids over an annual cycle. However, little information is available on the decomposition rates of deposited solids under conditions in which solids are continually being added to the deposits (3).

The accumulated sludge deposits in a partially suspended pond exerts a benthal oxygen demand on the system. Pond oxygen requirements must take into account this benthal demand. Rich suggests a value of

approximately 60g O_2/m^2 per day (approximately 0.01 lbs. O_2/ft^2-day) (3). Manufacturer's literature suggest a value of 0.25 lbs. O_2/lb. BOD applied (4, 29, 34).

Temperature, as noted previously, has an effect on the dissolved oxygen concentration. However, in relation to benthal oxygen demand, little information is available. Rich again recommends a value of 1.05 for the sensitivity coefficient to relate the temperature to the specific decay rate, k_d (3). Additional research relating benthal oxygen demand to temperature is necessary to quantify a design value.

Hinde Engineering has provided a typical materials balance for a 1 MGD plant (4). This materials balance is shown in Figure 2.1. Table 2.2 is provided to show the correlation between project design life and required storage volume in days for accumulated sludge. This table presents a method of predicting detention time for stabilization and digestion of sludge.

Suspended Solids, Removal Efficiencies

Removal of settlable solids is another parameter used in determining the degree of treatment and water quality. For these reasons, regulatory agencies require measurement of suspended solids in the influent and effluent streams.

Aerated lagoons can remove a large portion of the suspended solids from wastewater. However, specific parameters such as detention time, turnover time, temperature, light penetration and nutrients, must be controlled to effectively prevent growth of algae. Rich (3) cites several factors that influence removal of suspended solids by gravity settling. These are: (a) fraction of suspended solids that are

nonsettlable, (b) level of turbulence in cell, and (c) surface overflow rate from cell.

The major component of suspended solids found in the effluent streams from aerated ponds is algae. Oxidation ponds predominantly have mixtures of multicellular greens (Chlorophyta) and blue-greens (Cyanophyta) (4). Sedimentation rates for algae are typically low.

The fraction of suspended solids which are nonsettlable can be controlled by providing conditions that will reduce the growth of algae (2, 18). Since light is a necessary constituent for algae growth, increasing the pond depth will reduce light penetration and thereby reduce the growth of algae. Long retention times also enhance the growth of algae; therefore, systems should not be oversized where suspended solids concentrations must be controlled.

Rich (3) reports that there is a specific threshold level of turbulence that maintains a specified weight fraction of suspended solids. To control the level of suspended solids removal the aeration power must be kept below the threshold level of a complete-mix cell. Rich recommends a pond or cell three meters in depth (9.75 feet) with a detention time of one day, which will yield a surface overflow rate of 3 m^3/m^2 per day (75 gal./ft^2-day). This rate is low enough under quiescent conditions to remove almost all the suspended solids.

The subsequent concentration of algae in the effluent from a aerated pond system is also dependent on the time of year. Warm water temperatures and long daylight hours increase the amount of algae present in the water. Photosynthesis is the process by which the algae utilize the sun's radiant energy to fix atmospheric CO_2 and thereby

provide the reducing power to convert CO_2 to organic compounds (46). Some species of bacteria found in lagoons, as well as green plants, are capable of utilizing photosynthesis for plant maintenance.

Algae can be unicellular or multicellular, motile or immotile (46). Algae are typically autotrophic and thereby utilize inorganic nitrogen rather than atmospheric nitrogen. The inorganic nitrogen found in a typical domestic wastewater are in the form of nitrate or ammonia. Some algae species utilize amino acids in place of nitrate or ammonia for their inorganic nitrogen source. All of the inorganic compounds necessary for algal survival are photosynthetically converted into cellular organic materials, with oxygen produced as a by-product (46).

Algae and bacteria are generally accepted as being beneficial to pond treatment systems. Bacteria are required to break down the complex organic compounds. Algae can utilize some of the minerals liberated in bacterial degradation. Most importantly, the algae provide supplemental oxygen to the treatment system. This is one source of natural dissolved oxygen in an aquatic system.

In partial-mix systems using submerged-tube aeration, algal blooms are reduced due to the mixing within each cell. Also, the depths of each cell, approximately 10 to 13 feet, prohibit sunlight penetration beyond approximately 10 inches. Retention times should be limited to 4 to 5 days, for series cell operation. As additional cells in series are added to the treatment system, more of the limiting nutrients for algae growth are utilized and organism growth is reduced. These limiting nutrients are phosphorous and nitrogen, with phosphorous being the key nutrient for growth. Each additional pond in series removes more

phosphorous through algae uptake, thereby reducing the available limiting nutrient in succeeding ponds and in turn reducing algae growth (46).

Algae removal from the pond effluent can be accomplished by chemical treatment and filtration. Chemicals typically used for algae control are chlorine and copper sulfate. Killing algae by chemical addition exerts an additional BOD load on the system or receiving stream. Filtering the effluent will remove suspended solids; however, it causes a maintenance problem due to the clogging of the filter or screening device.

In addition to being a continuing maintenance problem, the retained solids must be disposed of. The literature indicates that typically disposal of algae and suspended solids from pond effluents is done by land application. However, landfilling is also common. With either practice, it is the generator's responsibility to prove that the solids are not hazardous waste, therefore requiring special disposal techniques.

Hinde Engineering reports that their Air-Aqua systems predominantly have single-celled planktonic greens (Chlorella, Scenedesmus and Pediastrum are typical species). These algae species are not essential for treatment in mechanically aerated systems. Hinde Engineering found that the artificially induced circulation from aeration equipment eliminates the propagation of multicellular algae species. Series lagoon operation, as cited, reduces algae growth. Air-Aqua systems claim a removal rate of 98 percent or higher with clarification or filtration for suspended solids (4).

Nutrients in Domestic Wastewater

Nutrients such as nitrogen, phosphates, sulfur, iron, carbon, hydrogen and oxygen are the principal organic nutrients that can be utilized by bacteria and are generally found in a typical domestic wastewater (46). Nitrogen and phosphorous are the primary nutrients of concern in the stabilization pond or aerated lagoon treatment system since they are usually the limiting elements in algae growth. Both nitrogen and phosphorous can undergo changes when passed through a stabilization pond system (4). Metcalf and Eddy (46) state:

> Nitrogen can undergo a number of chemical and physical processes, including settling (in organic particulate form), assimilation into algae cells, ammonification (conversion of organic nitrogen to ammonia), nitrification, and dentrification. Phosphorous is removed by assimilation into algae cells and by precipitation.

Alkalinity is the key to phosphate concentration variations. During daylight hours the alkalinity levels in the pond waters will increase and cause phosphate to precipitate. Night brings lower levels of alkalinity and will cause some of the phosphorous present in the wastewater to dissolve from the sediment. It has been determined that typically the effluent phosphate concentration is less than one-half of the influent concentration (46).

In a wastewater stream, organic nitrogen and ammonium nitrogen are the nitrogen forms found in the influent stream. A simple nitrogen cycle is presented in Figure 2.2. The organic nitrogen is contained in fecal matter and other organic suspended solid materials. Microbial activity then converts this organic-N to ammonia and the ammonium ion. The ammonium ion is then nitrified to nitrite and then nitrate. The organisms responsible for nitrification from the ammonium ion to

nitrites are Nitrosomonas, and those which convert nitrites to nitrates are Nitrobacter. The overall reaction by equation is (2):

$$NH_4^+ + 2O_2 \rightleftharpoons NO_3^- + 2H^+ + H_2O \quad (21)$$

The nitrate and ammonium ions present in the water can be utilized, as mentioned previously, by other organisms. This is called assimilation and produces cell protein and other nitrogen-containing compounds. The nitrate is converted to nitrite and then to nitrogen gas. Several bacteria are responsible for this process, namely, Pseudomanas, Micrococcus, Achromobacter, and Bacillus (2). The dentrification process in equation form is (2):

$$6NO_3^- + 5CH_3OH \rightleftharpoons 3N_2 + 5CO_2 + 7H_2O + 6OH^- \quad (22)$$

Changes that can occur with the different forms of nitrogen present in wastewater ponds under aerobic conditions are shown in Figure 2.3.

Hinde Engineering Company, the manufacturer of the submerged tube aeration system, claims nutrient removal is obtained through nutrient uptake by organisms and precipitation (4). Ammonia, when present in the cells at a temperature of 20°C and a neutral pH, is converted at 85 to 90 percent efficiency. When the temperature ranges are below 5°C or above 40°C, pH is not neutral and removal rates are much lower (46). If the nitrogen and phosphate maximum effluent concentration limits are 5 mg/l or below, then additional treatment equipment is required.

pH and Alkalinity Influences

The effect of pH and alkalinity on the biological environment within pond systems can be explained by the following equations (2):

$$CO_2 + H_2O \rightleftharpoons H_2CO_3 \rightleftharpoons HCO_3^- + H^+ \quad (23)$$

$$M(HCO_3)_2 \rightleftharpoons M^{++} + 2HCO_3^- \quad (24)$$

$$HCO_3^- \rightleftharpoons CO_3^= + H^+ \quad (25)$$

M = metal ion

$$CO_3^= + H_2O \rightleftharpoons HCO_3^- + OH^- \quad (26)$$

$$OH^- + H^+ \rightleftharpoons H_2O \quad (27)$$

The above equations describe the basic carbonate buffering system which controls the hydrogen ion concentration or pH in the lagoon or pond. Figure 2.4 shows a typical relationship between pH, CO_2, HCO_3, CO and OH (2).

Photosynthesis reactions affect the total equilibrium of the system. Algae remove carbon dioxide during metabolic processes from the dissolved phase, causing the equations to produce the products on the left-hand side (2). This tends to decrease the hydrogen ion concentration and raise the pH. By raising the pH, bicarbonate alkalinity is decreased. Decreasing the bicarbonate alkalinity pushes equation 21 to the left and equation 22 to the right, thereby decreasing total alkalinity. Photosynthesis, hence, will tend to decrease alkalinity and reduce carbonate hardness in the wastewater.

Calcium Carbonate Scale Formation

Scale formation around the orifice of the submerged tube is a characteristic specific to a partial-mix system. This scale is formed at the orifice by the release of fine air bubbles into the water above. The fine air bubbles cause an oxidation reaction to occur in the water. Specific compounds formed from metal ions and $CO_3^=$ or HCO_3^- are

precipitated by this oxidation reaction. The following equation shows the actions of these ions:

$$3Ca^{++} + H_2O + (3O_2) + CO_2 \leftrightharpoons HCO_3 + CaCO_3 + O_2 \quad (28)$$

A precipitate of $CaCO_3$ occurs readily in waters that have a pH of 7.0 to 9.0. The hardness of the water, measured as mg/l of $CaCO_3$, is very significant in the amount of scale that forms around the slits in the submerged tubing.

Water with high concentrations of carbonate hardness are assumed to cause frequent scale formation around the orifices. Hinde Engineering recommends the periodic use of a hydrogen chloride gas cleaning system to feed the resultant acidic solution into the aeration piping to dissolve the scale formation on the orifices (4). Scale formation causes increased back pressures on the blowers, resulting in a decrease in system efficiency.

Pond Geometry

Raymond A. Ferrara and Donald R.F. Harleman (35) analyzed various pond geometries with hydraulic design models to predict an efficient hydraulic design. Their findings from dye tracer investigations indicate that "none of the ponds achieved the efficiency of plug flow." Treatment performance in a pond system is dictated by both the hydraulic movement pattern and the biological and chemical treatment processes in the system. Ferrara and Harleman found that "the plug-flow model overestimates treatment performance." A complete-mix system more nearly approximated the simulated organic removal rates. They also concluded that small length to width ratios, 1:1, did not have high organic

removal efficiencies (35).

It can be assumed that sludge accumulation would be increased in an installation with length-to-width ratios smaller than 3:1. Again, the geometry of these ponds approximates the plug-flow pattern. Plug-flow analysis has been shown to grossly overestimate BOD removal rates in these systems. Rich found that if the length/width ratio exceeds 2:1, then the power level for aeration needs to be increased to ensure uniform distribution of dissolved oxygen (3).

The Texas Department of Health has stipulated for aerated facultative ponds that the length-width ratios "should be 3:1" (42). The flow shall be oriented to the length from inlets at one end to outlets at the opposite end. Other configurations are optional with approval from the Department. The directional layout of the primary cell is recommended to be in the same direction as the prevailing winds, with the inlet being oriented on the lee side of the prevailing winds. Reasons for the inlet works being placed on the lee side of the ponds with respect to the prevailing winds is to reduce the amount of scum and floatables transferred into adjoining cells.

Other researchers have recommended that the aerated ponds be rectangular in shape and use a length-to-width ratio in the range of 2:1 to 4:1. The general side slopes of the ponds is typically 3:1, (horizontal to vertical). The pond configuration recommended in the Hinde Engineering Design Manual is a rectangular shape which utilizes common sides to form the cell system (4). In almost all cases, state regulation dictates the physical parameters of the ponds. The Environmental Protection Agency recommends a length-to-width ratio of at

least 3:1 (2). In almost all sources reviewed, it was found that plug-flow hydraulics better defined the BOD_5 removal rates with first order reaction kinetics. This is due in part to the physical design being composed of rectangular basins with 3:1 (length to width) ratio, and because in most installations the ponds are designed in series.

Current Operation of the Brownfield Plant

The Brownfield Wastewater Treatment Plant consists of a three-cell aerated lagoon system with "no-discharge" permit for the effluent. This means that final disposal of all effluent is accomplished by applying it to adjacent cropland as irrigation water. A schematic diagram of the plant is shown in Figure 2.5.

Raw sewage from the city is collected in a new lift station constructed in conjunction with the new treatment plant. The sewage is then lifted into an open channel which flows to the preliminary treatment units. The preliminary treatment unit consists of a comminutor with bar screen bypass and a non-aerated, rectangular grit chamber. After passing through the grit chamber, the wastewater enters pond 1 through two submerged inlets located approximately 3.0 feet above the bottom of the cell. The influent organic loading to cell 1 was estimated to be 200 mg/l BOD and 200 mg/l total suspended solids (1).

Influent flows through cell 1 are based on plug-flow theory, and aeration is provided throughout the cross section of the cell. A greater degree of aeration is provided at the influent end of cell 1. Air volumes per unit volume of liquid are subsequently decreased as the wastewater flows toward the effluent end.

"Step-aeration" is also provided in cells 2 and 3 in the same

manner as described for cell 1. Submerged polyethylene tubing traverses the cross section of each cell. This tubing has small slits or "orifices" located on the top side of the tube and a lead weight running the entire length of the tube on the bottom side. The tubing is closely spaced at the influent end of cell 1. The spacing between succeeding tubes increases toward the effluent end of each cell. Aeration for the three cells is provided by the use of three 375 scfm Sutorbilt 608V-4500 positive displacement blowers.

Wastewater flows through cells 2 and 3 in the manner described for cell 1. The effluent is collected in a concrete structure housing three vertical turbine irrigation pumps and two recirculation pumps. The effluent pump structure is shown in Figure 2.6. The treated wastewater is pumped to city-owned land for use as irrigation water.

Holding ponds for effluent storage are typical in the design of a land application system. However, due to space limitations and the additional cost, no holding pond was provided. The holding pond volume was theoretically accounted for by designing additional storage in the three cells of the treatment system. This appeared to be a simpler and least expensive way of providing storage for non-effluent disposal days.

The treatment system is generally run in a series mode. However, flexibility is provided by the piping and valving as shown on Figures 2.7 and 2.8, the entrance structure and transfer pump structure, respectively. By the use of valves, the influent flow can be split and the plant can be operated in parallel. Lagoons 2 and 3 float together (same depth in both ponds) and combined have the same volume as 1. Normal operating depth was initially set in the design phase at 14 feet.

Detention time for flow-through is 18 days, 9 days for cell 1, and 4.5 days each for cells 2 and 3, at the design depth of 14 feet.

During the study period, the plant was treating approximately 0.5 mgd. The plant is designed for 1.25 mgd. Occasionally, flows of 0.9 to 1.0 mgd were seen during the summer months. An additional sludge storage volume of approximately 15 percent of the total bottom area of pond 1 was provided.

The municipal water supplied to the Brownfield area has a $CaCO_3$ hardness of 500 mg/l. The higher the hardness concentration, the greater the potential for scale formation in the submerged tube system. Increased frequency of cleaning of the tube system is required with the increase in scale formation.

Some 60 pounds of anhydrous hydrogen chloride is fed into the aeration system to reduce calcification of the orifices and thereby improve overall system efficiencies. The associated cost for HCl gas is approximately $150 per cylinder. The effective gas weight per cylinder is 60 pounds and the Brownfield system requires cleaning at least twice per month.

Problems at Beginning of Study

Brownfield BOD_5 removal efficiencies recorded prior to the study ranged from 0 to 87 percent. Suspended solids removal efficiencies varied with the season of the year and ranged from 0% to 41% in summer and 65% to 87% in winter. Typical average effluent values for BOD_5 and TSS were 65 and 115 mg/l, respectively (10).

Odors had been a particular problem with this plant during the first year of operation, and have been primarily due to equipment

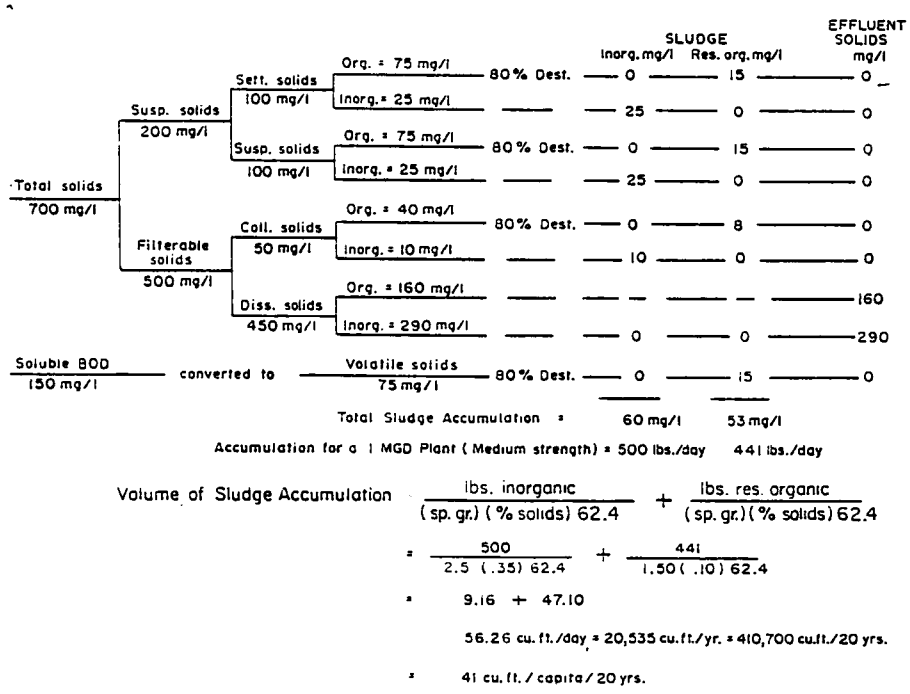
failures. Blowers have continually experienced high operating temperature gradients (175^{o}F) across the blower, from inlet to outlet. This was believed by the author to be responsible for the numerous blower failures.

TABLE 2.1

MID-DEPTH VALUE FOR C_{DC}
mg/1 at 20°C

Site Elevation (ft.)	5' Water Depth	10' Water Depth	15' Water Depth
Sea level	9.89	10.59	11.28
500	9.72	10.42	11.11
1000	9.56	10.25	10.95
1500	9.39	10.09	10.78
2000	9.23	9.93	10.62
2500	9.07	9.77	10.46
3000	8.92	9.61	10.29
3500	8.76	9.46	10.15
4000	8.61	9.30	10.00
4500	8.46	9.16	9.85
5000	8.31	9.00	9.70

SOURCE: Hinde Engineering Company, Engineers Design Manual for the Air-Aqua Lagoon Systems, Hinde Engineering Company (1980).



$$\text{Volume of Sludge Accumulation} = \frac{\text{lbs. inorganic}}{(\text{sp. gr.})(\%\text{ solids})\,62.4} + \frac{\text{lbs. res. organic}}{(\text{sp. gr.})(\%\text{ solids})\,62.4}$$

$$= \frac{500}{2.5\ (.35)\ 62.4} + \frac{441}{1.50\,(.10)\,62.4}$$

$$= 9.16 + 47.10$$

$$56.26 \text{ cu. ft./day} = 20{,}535 \text{ cu. ft./yr.} = 410{,}700 \text{ cu. ft./20 yrs.}$$

$$= 41 \text{ cu. ft./capita/20 yrs.}$$

FIGURE 2.1: Materials balance format.

SOURCE: Hinde Engineering Company, Engineers Design Manual for the Air-Aqua Lagoon Systems, Hinde Engineering Company (1980).

TABLE 2.2

SLUDGE VOLUME ALLOCATION
(DAYS DETENTION)

Suspended Solids (mg/l)	Sludge Concentration (mg/l)		10-Year Project Life	20-Year Project Life
	Inorganic	Organic		
200	60.0	53.0	1.54	3.08
250	75.0	66.25	1.93	3.85
300	90.0	79.50	2.31	4.62
350	105.0	92.75	2.70	5.39
400	120.0	106.0	3.08	6.16
450	135.0	119.25	3.47	6.93
500	150.0	132.50	3.85	7.70
600	180.0	159.0	4.62	9.24

SOURCE: Hinde Engineering Company, Engineers Design Manual for the Air-Aqua Lagoon Systems, Hinde Engineering Company (1980).

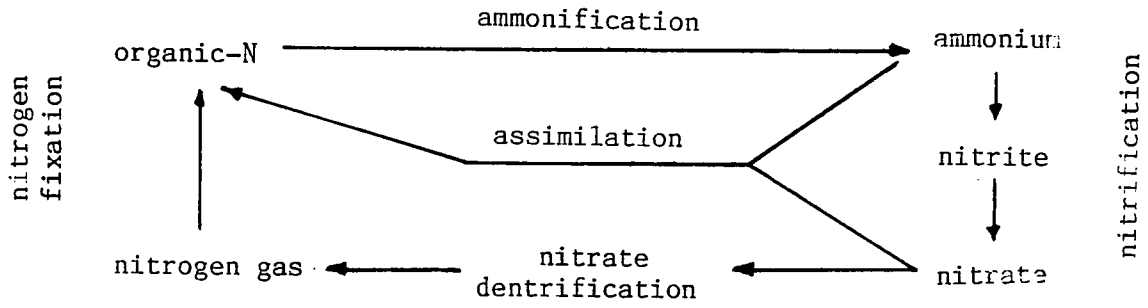


FIGURE 2.2: Nitrogen cycle.

SOURCE: United States Environmental Protection Agency, Design Manual: Municipal Wastewater Stabilization Ponds (Oct. 1983), p. 13.

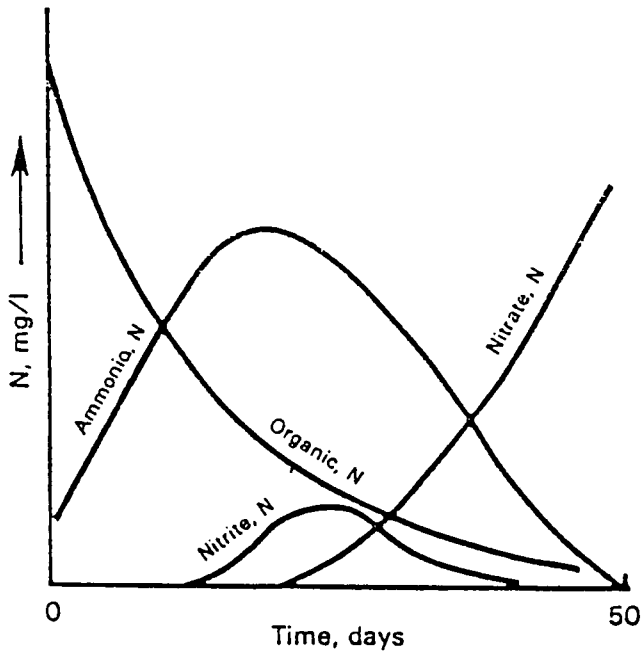


FIGURE 2.3: Changes occurring in forms of nitrogen present in a pond environment under aerobic conditions.

SOURCE: United States Environmental Protection Agency, Design Manual, Municipal Wastewater Stabilization Ponds (Oct. 1983), p. 18.

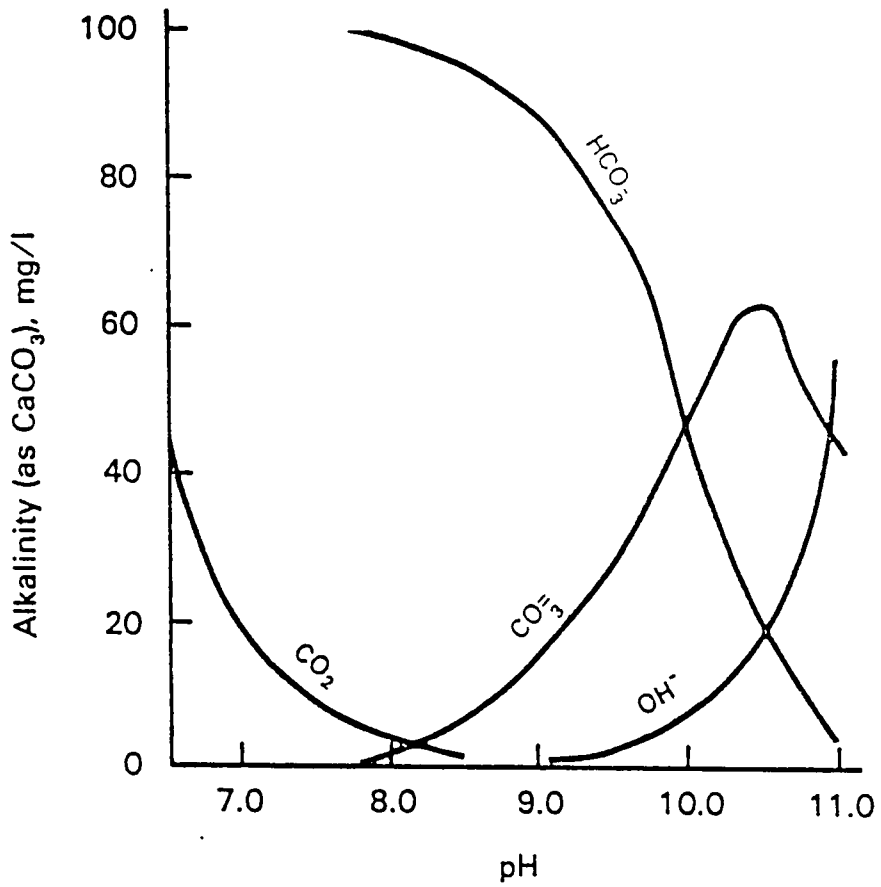


FIGURE 2.4: Calculated relationship among pH, CO_2, $CO_3^=$, HCO_3^-, and OH^-.

SOURCE: United States Environmental Protection Agency, Design Manual: Municipal Wastewater Stabilization Ponds (Oct. 1980), p. 15.

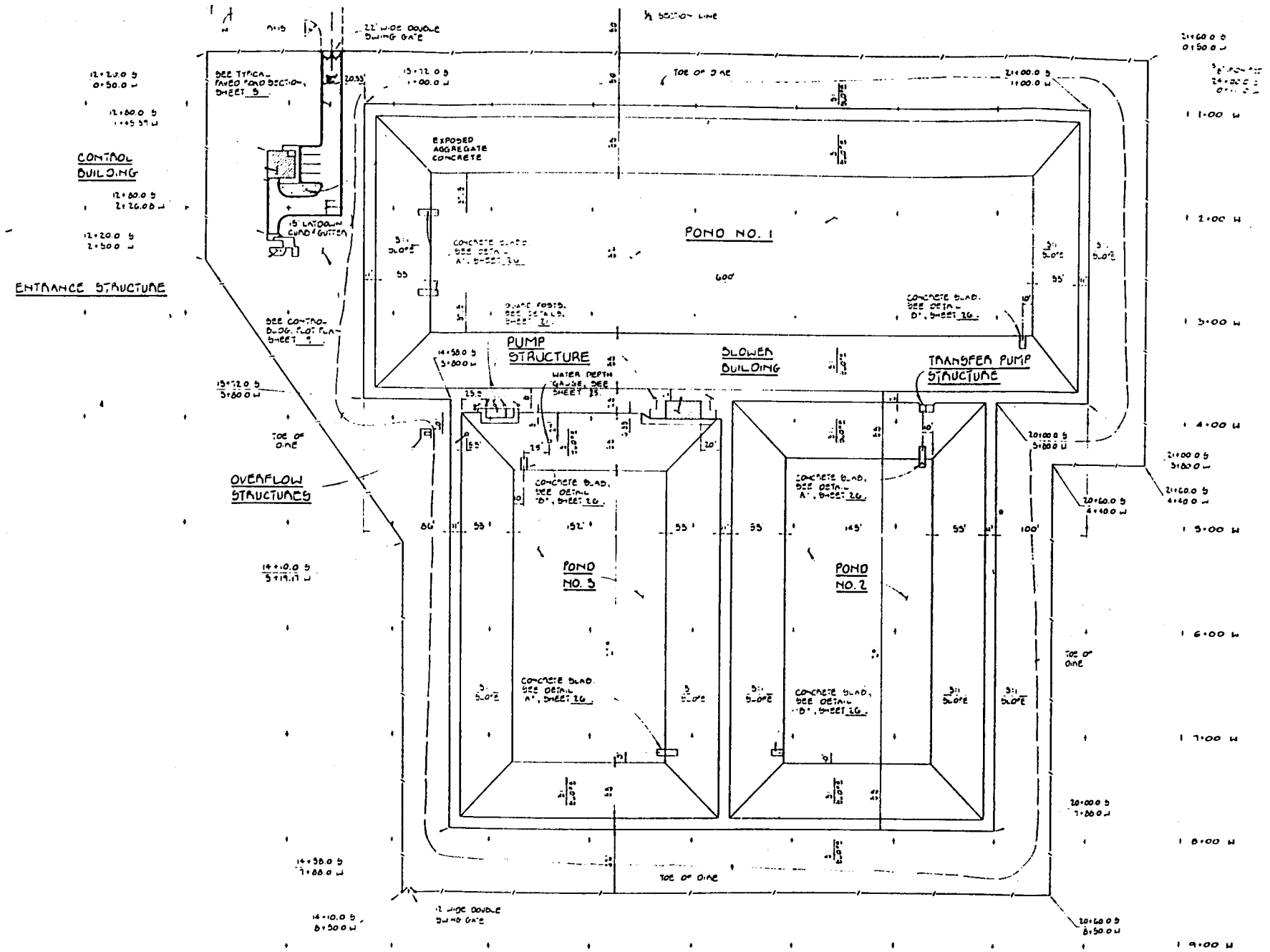


FIGURE 2.5: Plan view of the Brownfield wastewater treatment plant.

SOURCE: City of Brownfield Wastewater Treatment Plant Plans, Parkhill, Smith & Cooper, Inc., 1980.

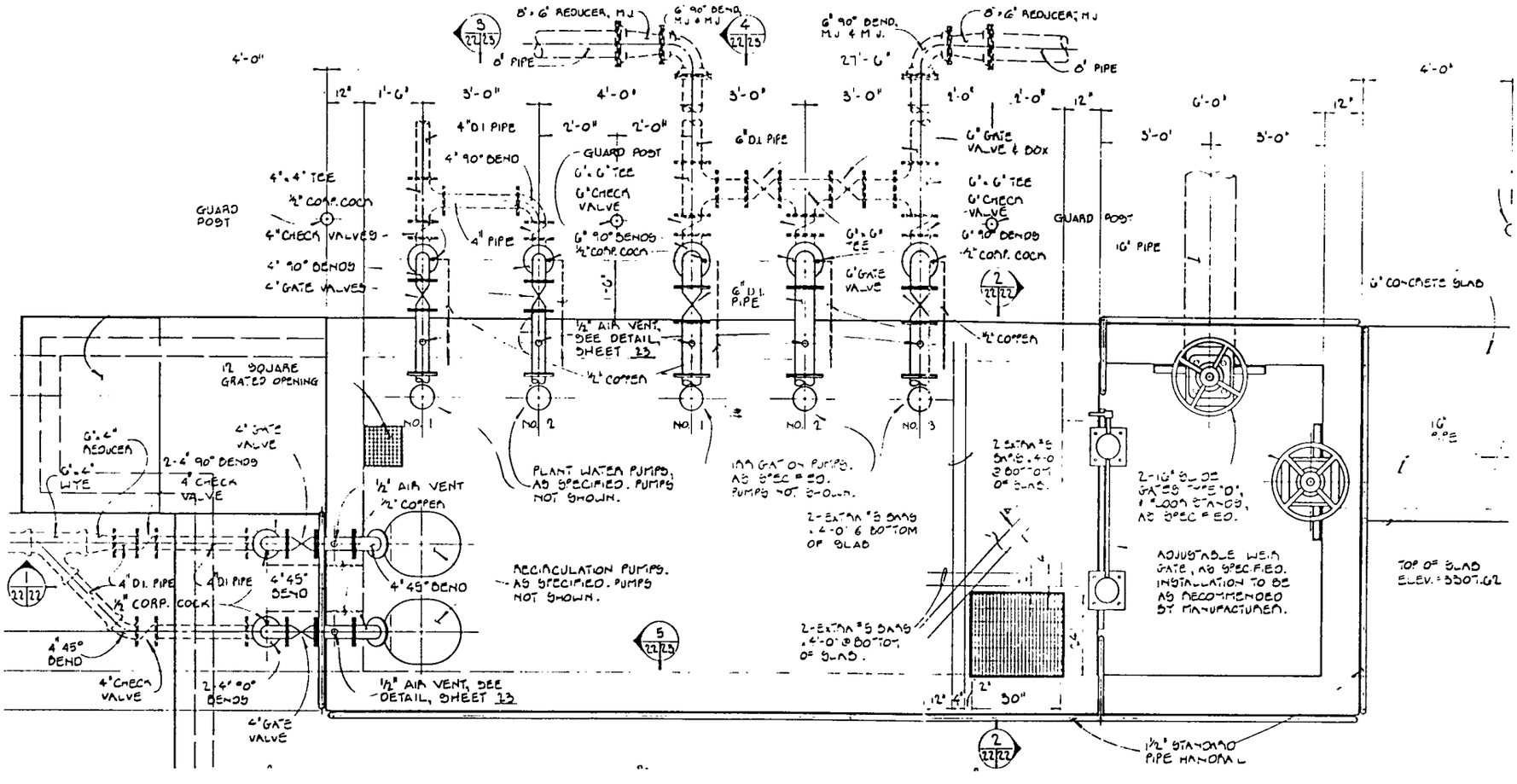


FIGURE 2.6: Plan view of pump structure.

SOURCE: City of Brownfield Wastewater Treatment Plant Plans, Parkhill, Smith & Cooper, Inc., 1980.

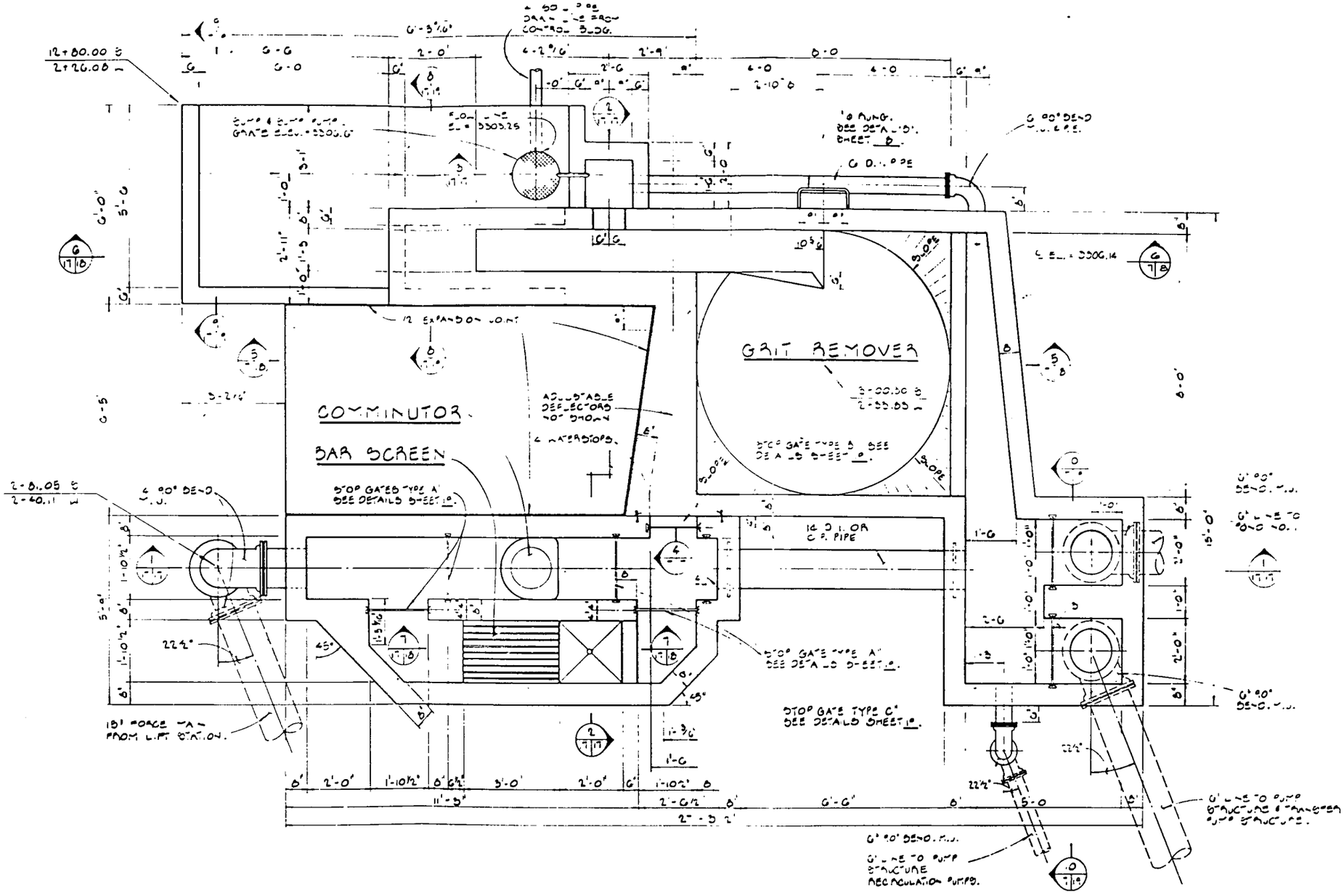


FIGURE 2.7: Plan view of entrance structure.

SOURCE: City of Brownfield Wastewater Treatment Plant Plans, Parkhill, Smith & Cooper, Inc., 1980.

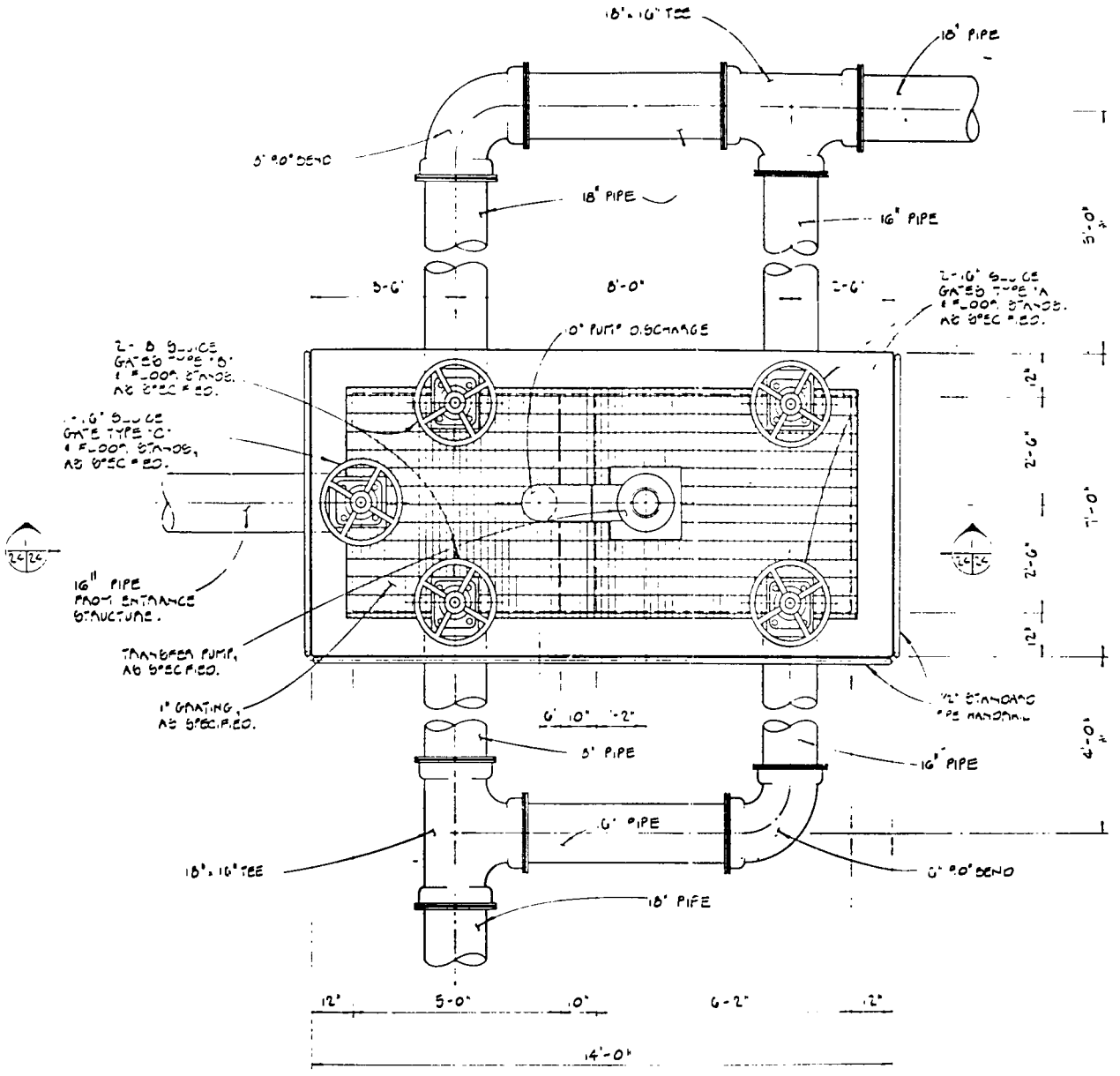


FIGURE 2.8: Plan view of transfer pump structure.

SOURCE: City of Brownfield Wastewater Treatment Plant Plans, Parkhill, Smith & Cooper, Inc., 1980.

CHAPTER III

DESCRIPTION OF EXPERIMENTAL PROCEDURES

Introduction

The study was concerned with the identification of the problems causing the poor effluent quality from the Brownfield wastewater treatment plant during its initial year of operation and the formulation of corrective procedures to rectify the operational problems discovered. The following tasks were performed to accomplish the project objective:

1. Characterize wastewater quality, wastewater flows, and energy needs over a calendar year.
2. Compare design criteria for the system with operational results obtained in task 1.
3. Examine operational and maintenance procedures used by municipal employees in operation of the plant during the study period.

Each task will be discussed in more detail in subsequent sections.

Additionally, the actual treatment results were compared to the theoretical results obtained using design equations used in the actual design of the plant. An attempt was made to determine if the design equations used did in fact mathematically predict the actual organic removal efficiencies of the treatment facility. Correlation of kilowatt-hours (Kwhrs) that were used per pound of BOD removed was examined. This phase required BOD_5 analysis as well as measurement of amperages from the blowers at varying water levels and organic loads.

A great deal of effort was also targeted at identifying operational procedures and performance, along with routine maintenance. This phase of the study was directed at predicting the staffing levels, operation

and maintenance required for a submerged tube aeration process.

All laboratory analyses were performed in the laboratory at the treatment facility. Chemicals used were reagent grade. Water used for standards and sample preparation was tap water which first passed through a Corning D-3 Demineralizer. Glassware was cleaned according to the recommendations set forth by The Texas Department of Water Resources (42), EPA's Methods for Chemical Analysis of Water and Wastes (48) and the 15th edition of Standard Methods for the Examination of Water and Wastewater (41).

The two operational modes (series and parallel) utilized in the normal operating procedure of the plant were evaluated to compare the removal efficiencies of design equations versus Linvil Rich's design equations. Theoretically, optimum removal efficiencies occur when the normal flow-through scheme is from pond 1 to pond 2 to pond 3.

BOD_5 samples were collected from the influent of the plant and at the discharge end of each pond. Dissolved oxygen measurements were taken at the same locations. The pond corners at the end of the pond to be sampled were selected as the most conservative points to measure dissolved oxygen concentrations, since there was no aeration or mixing provided in the corners.

Task 1

Sampling operations were conducted during periods when the plant was operated in both the series and parallel mode. In order to determine removal efficiencies in each mode, daily grab samples were taken at selected points throughout the plant. Laboratory analyses were performed in the laboratory at the treatment facility by city employees

using procedures outlined in Standard Methods (41) or approved by the Texas Department of Water Resources.

Sampling Locations

Sampling locations were selected as recommended in the facility Operation and Maintenance Manual. Theoretically, optimum removal efficiencies occur when the normal series mode of operation is used. BOD_5 samples were collected from the influent and effluent of each cell as recommended in the O&M Manual to compare the removal efficiency within each cell. Dissolved oxygen readings were also collected at the BOD_5 sampling points. The sampling points recommended in the O&M Manual for all organic samples were the corners of the cells and the grab samples collected were then combined prior to analysis as a composite sample. Figure 3.1 shows the suggested sampling points for the Brownfield plant. Table 3.1 lists the sampling locations and the types of analyses ran on the samples. The sampling schedule used is as shown on Table 3.2.

Sample Analysis

The laboratory used for the analysis is located in the control building and was constructed as part of the treatment facility. Chemicals used in the analysis were reagent grade. Dilution water used for the preparation of reagents, standards and samples were demineralized by filtering the water through a Corning D-3 Demineralizer. Major equipment used for performing the qualitative analysis are as listed:

1. BOD incubator
2. Portable YSI DO meter
3. Portable composite sampler with a battery-operated power supply
4. Portable Fisher Scientific pH, conductivity, and temperature meter
5. Electronic analytical balance
6. Corning Cartridge Demineralizer
7. Boat for collecting specific samples from the mid-sections of the cells.

Additional sampling equipment was required for specific sample collection. These items were constructed on-site and consisted of the following:

1. DO sample collection rod (used to collect samples at various depths and designed to prevent air bubbles from entering the sample bottle).
2. Sludge depth gauge rod (to measure the depth of sludge deposits in the cell bottoms).

The sludge depth rod did not work as planned. The depths of the cells and the material used in the construction of the rod made handling the rod in a boat awkward. Also, the depths of the cells caused the rod to deflect horizontally due to the buoyancy effect on the material. The sludge measuring device was redesigned, consisting of a series of interconnecting sections of pipe with a piece of 1/4" mesh screen wire end plate affixed to one end of one of the pipe lengths. This rod was constructed of 3/4" galvanized pipe. The rod was lowered to the top of the sludge deposits and the depth noted, then the rod was lowered to the bottom of the cell. The difference in the two measurements was the

calculated depth of the sludge deposits.

BOD_5

The test procedure used for determining BOD_5 was that published in Standard Methods (41) for the dissolved oxygen membrane (probe) method. BOD_5 dilutions were prepared using 5 to 10 percent of the sample with deionized water. Samples were taken and analyzed weekly. Immediate Dissolved Oxygen Demands (IDOD) were taken when the samples were collected. The dilutions were prepared by using a 1000 ml graduated cylinder and miscellaneous smaller-volume graduated cylinders. Dilution water, approximately 500 ml, was added to the 1000 ml graduated cylinder. The dilution water was agitated vigorously prior to sample addition. The percent sample dilution was added to the 1000 ml graduated cylinder. A plunger mixer was used to thoroughly mix the sample and dilution water. The prepared samples were then submerged in a large bucket filled with deionized water for a period of five days. The samples were removed on the fifth day and a YSI oxygen meter was used to obtain the dissolved oxygen reading. The oxygen depletion was recorded and the five-day BOD parameter calculated by the following equation:

$$BOD_5 \text{ (mg/l)} = (DO_i \text{ mg/l} - DO_5 \text{ mg/l}) \frac{1000 \text{ ml}}{\text{ml sample used}} \quad (29)$$

Where:

DO_i = initial dissolved oxygen

DO_5 = dissolved oxygen at the end of five days

Dissolved Oxygen Measurements

Dissolved Oxygen (DO) measurements were taken daily from the influent stream and from each pond influent and effluent. Grab samples were taken in such a manner not to introduce excessive oxygen into the sample bottle. A YSI membrane oxygen analyzer was used and corrected for temperature and elevation. Calibration of the meter was checked daily by the "air method." Dissolved Oxygen readings were recorded to assess the waste load in the system. These measurements allowed immediate determination of the required mode of aeration operation to maintain a minimum of 1 mg/l of DO in all the ponds at all times.

BOD_5 and DO analyses were performed for series and parallel operation. The same analytical procedures were used for both operational modes.

Temperature

Temperature was measured daily in the influent stream and at the influent and effluent of each pond. Mid-depth temperature readings as well as surface temperatures in each pond were made. A combination glass electrode was used for temperature readings. Temperature became very important in determining seasonal variations of oxygen holding capacity. Ambient atmospheric temperatures, along with physical weather conditions, were recorded daily.

pH

The pH of the influent stream and at the effluent end of each pond were recorded daily. The pH of domestic wastewater is normally between 6.8 and 7.6, and the ponds should stay within this range to provide a

good environment for microorganisms (4).

Alkalinity

Alkalinity of the influent was monitored weekly by taking grab samples. Alkalinity determinations were made with a Hach kit using a colormetric procedure. Alkalinity measurements were thought to be of significance in evaluating the calcification of the orifice opening in the submerged tubing, as recommended by Hinde Engineering.

Flow

Daily flow recordings were made using a digital totalizer with circular chart. Influent flow determines the hydraulic loading on the system. Excess flow, that flow over the average daily flow rate after a storm event, is an indication of infiltration/inflow. Flows were recorded for state permit requirements and to describe the overall treatment efficiencies as related to BOD_5 and suspended solids. Flow recordings also provide a means of determining the optimum mode of operation, series or parallel. Flows were used to determine the actual surface loading rates. When combined with DO measurements, the best operational mode and the need for supplemental aeration can be determined.

Kilowatts per Pound of BOD_5 Removed

To evaluate the effectiveness of oxygen input per kilowatt-hour (Kwhrs) per pound of BOD_5 removed, the BOD_5 concentrations and the amperage readings from the blowers were required. The blowers are positive displacement blowers and a direct correlation to BOD_5 levels (or removal efficiencies) is not possible. However, as depths within

the ponds increase, more static head must be overcome, and additional amps are required. Depth measurements were taken and evaluated with respect to amps and BOD_5 concentrations recorded at the time of sampling.

Task 2

Design Analysis Procedure

The design analysis procedure consisted of the following:

1. identifying population and associated flows used in system design;
2. reviewing the design criteria required by the Texas Department of Water Resources;
3. reviewing the facility plan written for the Brownfield plant;
4. reviewing the assumptions made in the formulation of the design;
5. reviewing the equations used for determining organic loadings, reaction rates, temperature effects, removal efficiencies, oxygen transfer rates, and other parameters in the design;
6. evaluating the design equations used in the actual design versus other design equations found in the literature; and
7. identifying the differences between the design assumptions and design equations used for the Brownfield plant versus Rich's procedure.

The design analysis investigated all of the design procedures used in the actual design of the Brownfield plant. These results were then compared and contrasted to other similar designs and similar design equations, assumptions and procedures.

To better evaluate the operational efficiency of the system, the following parameters were calculated from the collected data:

k = BOD_5 removal reaction rate, L/mg-day (d^{-1})

F/M = food/microorganism ratio ($days^{-1}$)

SRT = Solids Retention Time (days)

MLSS = Mixed Liquor Suspended Solids (mg/1)

Y = Ultimate BOD (mg/1)

S_o = Influent Soluble BOD (mg/1)

L_T = Ultimate BOD @ Time (T) (mg/1)

L_1 = Ultimate One Day BOD (mg/1)

S_e = effluent soluble BOD_5 (mg/1)

Task 3

Operational Procedures

The operational procedures employed by operating personnel were outlined in the O&M Manual provided by the design firm during the construction of the plant. Analytical procedures and instruction were provided to all operations personnel for a period of approximately six months on an as-needed basis. Initial instruction was given for approximately one week on sampling, analysis and reporting. The analysis procedures followed were as published by the Environmental Protection Agency, National Pollution Discharge Elimination System (NPDES) in 1975 (49). Other references used for procedural instruction were Standard Methods (41), City of Brownfield Operation and Maintenance Manual (50), and Water Pollution Control Federation, Manual of Operation and Procedures (MOP-11) (51).

Maintenance Procedures

The required maintenance tasks for the plant were presented in the

Operational and Maintenance Manual (50) as being required on an as-needed basis. The only exception was with the blower operations. The blowers were originally set up on a daily maintenance schedule. Records were kept on inlet suction pressure, oil levels in the blowers and discharge pressures within the aeration system. The above maintenance routine was not altered from that presented in the Operations and Maintenance Manual.

Evaluation of O&M Procedures

Evaluations of the operational and maintenance procedures used by the municipal employees were based on the daily operational reports. These reports were primarily analytical work sheets, detailing BOD_5 and suspended solids determinations. Effluent quality was the basis of determining proper operations.

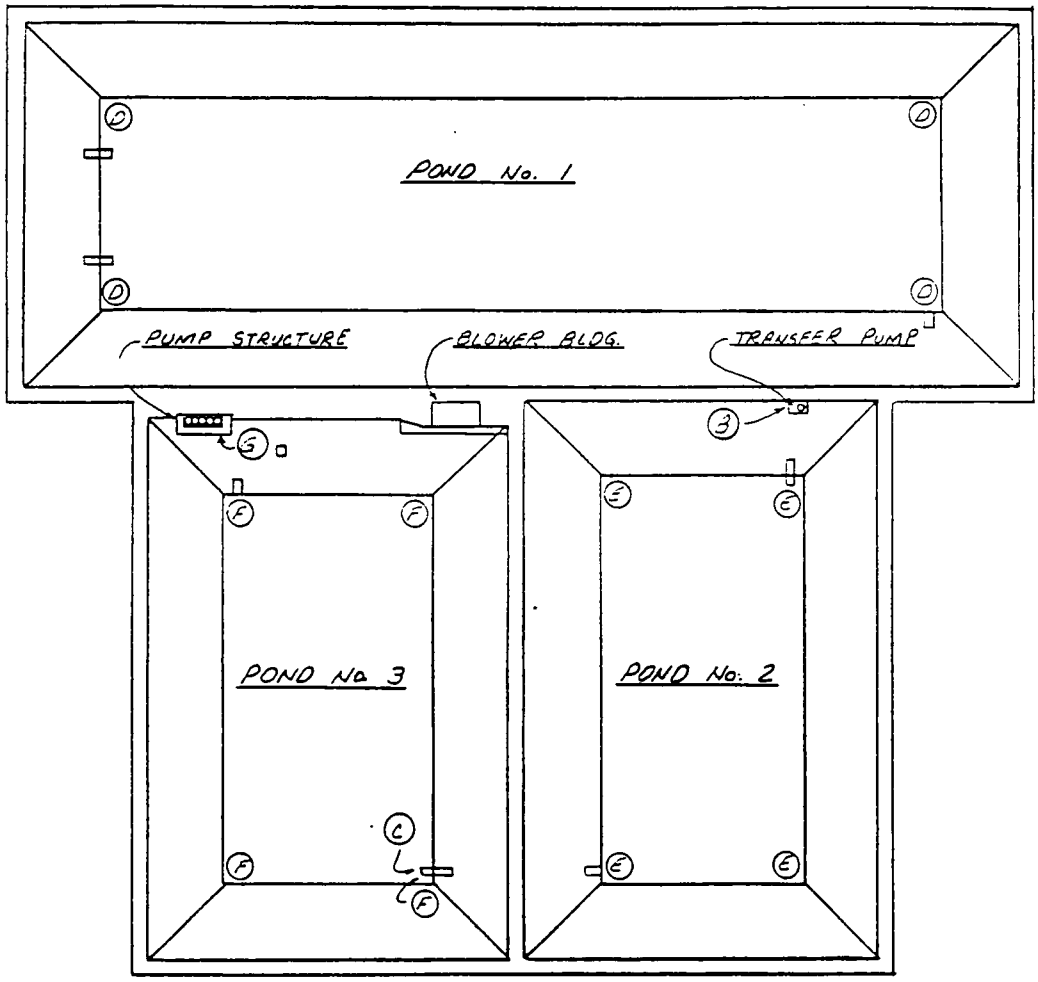


FIGURE 3.1: Suggested sampling points for the Brownfield treatment plant. (Letters in circles refer to Tables 3.1 and 3.2.)

SOURCE: Operations and Maintenance Manual for the City of Brownfield, Texas, Water and Environmental Technology, 1983.

TABLE 3.1

SAMPLING LOCATIONS AND TESTS

Location	Figure 3.1 Label	Frequency	Tests
Entrance Structure	A	Daily	Flow, Temp, pH
		1 per week	BOD_5, TSS[a]
Pond No. 2 Influent	B	Daily	Temp, DO, pH
Pond No. 3 Influent	C	Daily	Temp, DO, pH
Pond No. 1	D	[b]	Temp, DO, pH
Pond No. 2	E	[b]	Temp, DO, pH
Pond No. 3	F	[b]	Temp, DO, pH
Final Effluent	G	Daily	Flow, Temp, pH, DO
		1 per week	BOD_5, TSS[a]

[a]Run composite samples once per week. Run grab samples every third day. weekends included.

[b]Run grab samples once per week.

TABLE 3.2

REVISED SAMPLING LOCATIONS AND TESTS

Location	Figure 3.1 Label	Frequency	Tests
Entrance Structure	A	Daily	Temp, pH, DO
		2 per week	BOD_5[a,b]
Pond No. 2 Influent	B	Daily	Temp, pH, DO[b]
Pond No. 3 Influent	C	Daily	Temp, pH, DO[b]
Pond No. 1	D	Daily	Temp, pH, DO[b]
Pond No. 2	E	Daily	Temp, pH, DO[b]
Pond No. 3	F	Daily	Temp, pH, DO[b]
Final Effluent	G	Daily	Temp, pH, DO[b]
		2 per week	BOD_5, TSS[a,b]

[a]Run composite samples once per week.

[b]Run grab samples as indicated.

CHAPTER IV

RESULTS AND DISCUSSION

Criteria Used in Design of Brownfield System

The original design assumptions made for the Brownfield Treatment Facility are shown in Table 4.1.

These were obtained from assumptions based on the wastewater characteristics from similar towns in West Texas with similar population densities. The assumed design values for the different parameters were reviewed by the Texas Department of Water Resources and were approved with the acceptance of the Facility Plan. The Facility Plan is prepared as a design document for all treatment plants involved in the Construction Grants program. The treatment efficiencies for the Brownfield plant were stipulated by the Texas Department of Water Resources.

The design analysis phase examined the design procedures used in the actual design of the Brownfield plant. The comparative analysis of the design phase revealed the following:

1. Wastewater characteristics at the project site should be identified by sample collection and analysis for parameters such as: BOD_5, TSS, and TS. (For the Brownfield facility plan, it was assumed that the waste flow was a typical domestic wastewater with both BOD_5 and TSS concentrations of 200 mg/1. As pointed out by Rich (3), design of the multi-celled, multi-powered, aerated lagoons should be based on standard biokinetic reaction rates. To improve plant treatment efficiencies, both biokinetics and waste characteristics of the wastewater should be used in the design phase. Sample analysis from

pilot scale facilities should be performed to determine optimum k values for the particular waste to be treated.)

2. Baffles were not provided to dispense the effluent as it flowed from one cell to another.
3. Transfer of water from one cell to another was performed using interconnecting pipes placed within three feet of the bottom of each cell.
4. Geometry of the cells followed Rich's recommendations, the EPA's recommendations, and those contained in TDWR's Design Criteria for Sewerage Systems.
5. The design depth of the cells was increased to allow for storage of solids. (This practice is recommended by the manufacturers of the submerged tube aeration systems, but is not a common design recommendation in the literature.)
6. The reaction rates for K_d used in the design was a typical value found in the literature rather than one developed in characterization studies.
7. Recycle of the effluent from cell 3 to the influent of cell 1 was provided, but only approximately 1 percent of the average daily flow was capable of being recycled.

The Texas Department of Water Resources waste treatment permit requires that Brownfield's plant remove 85 percent of the influent BOD_5 concentration. Removal efficiency was calculated by use of the following equation:

$$\text{Removal Efficiency } (\%) = E = (100\ K_e T)/(1 + K_e T) \tag{30}$$

where:

K_e = reaction rate coefficient

T = detention time (days)

Since the removal efficiency was specified by the regulatory agency, the unknown parameter in the above equation is detention time. Detention times were assumed and equated with K_e to arrive at a value for $K_eT_{1,2,3}$. The above equation was iterated until the total efficiency for the combined system efficiency was greater than or equal to 85 percent. The following equations show the actual removal efficiency for winter and summer conditions:

BOD Removal, Winter Conditions:

Pond 1: $K_eT_1 = 0.18 \times 9.0 = 1.6$; $E_1 = (100 \times 1.6)/(1 + 1.6) = 61.5\%$ (1)

Pond 2: $K_eT_2 = 0.18 \times 4.5 = 0.8$; $E_2 = (100 \times 0.8)/(1 + 0.8) = 44.4\%$ (1)

Pond 3: $K_eT_3 = 0.18 \times 4.5 = 0.8$; $E_3 = (100 \times 0.8)/(1 + 0.8) = 44.4\%$ (1)

Effluent BOD = $195(1 - 0.615)(1 - 0.444)(1 - 0.444) = 23$ mg/l (1)

BOD Removal, Summer Conditions:

Pond 1: $K_eT_1 = 0.35 \times 9.0 = 3.2$; $E_1 = (100 \times 3.2)/(1 + 3.2) = 76.2\%$ (1)

Pond 2: $K_eT_2 = 0.35 \times 4.5 = 1.6$; $E_2 = (100 \times 1.6)/(1 + 1.6) = 61.5\%$ (1)

Pond 3: $K_eT_3 = 0.35 \times 4.5 = 1.6$; $E_3 = (100 \times 1.6)/(1 + 1.6) = 61.5\%$ (1)

Effluent BOD = $195(1 - 0.762)(1 - 0.615)(1 - 0.615) = 7$ mg/l (1)

The above equations were the results of an iterative process that was continued until the 85 percent removal limit was met.

In summary, the design cell volumes that were developed utilized the above detention times and assumed K_e values to obtain the desired

removal efficiency of BOD_5 and effluent concentrations of less than 30 mg/l.

The design developed for the Brownfield plant did follow the acceptable practices for the design of a treatment system of this type. Many of the design assumptions used in the design of the plant were recommended by manufacturers such as Hinde, for their specific system. The assumptions that were questionable when compared to design procedures proposed by others are: (a) characterization of the waste stream; (b) determination of the K_e value for the specific waste stream; (c) soluble BOD_5 from the degradation of solids in the anaerobic area of the primary cell; and (d) the additional depth allowed in all the cells for storage of solids.

Results of Operational Study

Treatment Operations

The actual influent BOD_5 and effluent BOD_5 concentrations, along with the percentage removed by the system, are shown in Table 4.2. These values are the monthly averages over the study period. The actual removal rates only approached the design removal efficiency of 85 percent in November, 1984. An 82% organic removal rate was observed during that month. All other months indicate lower removal efficiencies.

The organic removal efficiencies per pond are shown in Table 4.3. The calculated combined removal efficiencies for the ponds in series for the winter months are shown in Table 4.4, and for the summer months in Table 4.5. Variability in the seasons appeared to have only a slight effect on the performance of the system. As the weather warmed in

April, May and June, removal efficiencies varied, and were not predictable. Effluent quality typically deteriorated during these transitional months with warming ambient air, warmer water temperatures, windy conditions, and a colder under-layer of water in the ponds.

Soluble BOD_5 increased during the winter months when additional water was being stored in the system. Solids accumulated daily and the microbial activity was low, causing a lower overall organic release during the months of December, January, February, and approximately the first half of March. When the influent water began to warm, the soluble BOD stored in the sludge solids were released from the deposits as indicated on Tables 4.2 and 4.5. The released BOD_5 combined in the upper levels of the ponds with the influent BOD_5 that was being broken down by the increased microbial action, resulting in a much higher BOD than typically found in the influent stream. Higher BOD's were indicated in succeeding ponds.

Parallel operation was initiated when odor problems developed and the BOD_5 values began to increase in pond 3 over the values obtained in pond 2. The result of parallel operation was an improvement in the BOD_5 removal rate over series operation. Parallel operation of cell 1 with cells 2 and 3 did help the odor problem, particularly when there had not been sufficient aeration capacity to maintain a minimum dissolved oxygen concentration of at least 2.0 mg/l in all cells.

A summary of the data from the Brownfield system is shown in Table 4.6. The results show a tendency for the system to lose dissolved oxygen. This is believed to be due to the benthal demand resulting from sludges stored through the winter months. Recent findings at a

similarly designed plant in Levelland, Texas have indicated higher BOD_5 removal rates and total suspended solids removal rates by using a 1-cell stabilization pond approximately 4 to 5 feet deep following pond 3. The stabilization pond has no aeration, and effluent BOD_5 concentrations were recorded at 5 mg/l consistently throughout the summer and fall.

Table 4.7 sets out the calculated parameters to show the actual values obtained in operation versus assumed design values. Table 4.8 lists the design values recommended by Rich (3).

The actual values calculated for the different design variables shown in Table 4.7 do not exhibit enough difference in value to expect the plant to not perform according to the original design expectations. Design values for the kinetic parameters presented by Rich, Metcalf and Eddy, Oswald and others have the same range of values assumed in the design phase of the Brownfield plant. The variable K_d or K_p value of 0.18 is in the range found in the literature. Influent BOD_5, when compared to similar systems, typically has averaged below the design value of 195 mg/l.

Variables like F/M (food to microorganism) ratio and S_e (soluble BOD in effluent) values were not utilized in the design phase of the project because these values are not typically required in the design considerations as set out by the Texas Department of Water Resources. The contrasting differences in the DO concentrations for actual and design conditions are due to the prolonged period of blower failure.

The SRT (solids retention time) value is the only variable that differs significantly from the original design assumption. Hinde Engineering (4) recommends this 20-year life design for SRT. Lasaire

(29), a similar manufacturer of the submerged tube aeration systems, is another source. The sludge mass in the bottom of the partially mixed aerated lagoons was shown to exert a higher benthal oxygen demand on the total aeration system than the original design allowed for. The SRT value then can be utilized in the design for a system with a 20-year life with no external sludge handling requirements. However, the benthal oxygen demand that is observed is often three to five times the influent BOD demand, and aeration facilities must be sized for this additional load which can vary throughout the year.

The design of the Brownfield facility was based on the removal efficiency dictated by the Texas Department of Water Resources. In this case 85 percent removal of the influent BOD_5 was required. The employment of a diffused submerged tube aeration system, as manufactured by Hinde Engineering Company, dictated the design to be a tapered aeration system, plug-flow type of hydraulic flow-through pattern. The tapered aeration system best describes the Hinde Engineering aeration system (4).

Blower Operations

Research by Rich (3), Oswald (44) and others, as well as manufacturers' literature (4, 29, 34), have indicated that depths within the ponds are critical to proper operation and removal rates. Hinde Engineering recommends that typical depths be within the range of 10-12 feet (4). The Brownfield system has a design operating depth of 14 feet. This additional depth reduced the overall surface area requirement but increased the static pressure for the submerged tube aeration system. The increased static pressure against the submerged

tubing may have increased the frequency of blower failures and/or partial shutdown of the aeration system. When blower failure occurred, the aeration tubing would fill with water. This phenomenon caused many tubes in the system to remain "waterlogged" for an extended period of time, further increasing static pressure against the blowers and reduced the amount of oxygen input into the cell(s).

The elevated static pressure resulting from increased depth in the ponds, "waterlogging" episodes, and possible calcification of the slits in the tubing eventually caused a high temperature gain across the blowers, and in doing so increased the potential for mechanical failure. This phenomenon was experienced with the subsequent results--a wastewater volume exhibiting a high oxygen demand and no means to supply the oxygen. The accumulated organic loads are never entirely compensated for, and therefore the system never completely recovers. Several alternatives were tried during these times of blower failure. The alternatives used were: (a) discharging effluent to other holding ponds; (b) discharging effluent via irrigation; (c) replacing the blower(s); and (d) splitting the flow between the primary cell number 1 and cells 2 and 3 (parallel operation mode). The results obtained with each option were:

Discharging Effluent to Other Holding Ponds

The additional holding ponds were from the original treatment system. The volumes available were less than the total volume from the new system. No means of discharging the wastewater was available from the holding ponds. Evaporation was the only way to empty the contents of all the ponds in the old treatment plant. Therefore, this

alternative was only good during the summer months and only on an emergency basis.

Discharging Effluent Via Irrigation

The predominant way of wasting water was to increase both the irrigation application frequency and the land area utilized for disposal. This was the most practical and easiest solution. There appeared to be no adverse effect on the land, crops or groundwater by using this alternative.

Replacing the Blowers

Replacing blowers was an expensive and time-consuming alternative and only proved practical when there were no blowers to supply the required volume of air.

Splitting the Flow Between the Primary Cell Number 1 and Cells 2 and 3

Operating the plant in parallel worked extremely well and actually produced a higher effluent quality than series operation. However, the operations personnel would not leave the plant in parallel operation for any length of time; therefore, this alternative could not be exploited to its fullest capability.

The optimum speeds for positive displacement blowers similar to the Brownfield installation are typically unknown by many engineers. The manufacturers recommend speed with rpm's in the range of 1200 to 2500. The design of positive-displacement blowers should be done with extreme care, such that an optimum rpm is selected that produces the required volume of air at the lowest possible temperature gain (27).

Operational Control of Treatment Process

The proper operation of the plant and its equipment is the determining factor to a successful operational design. All EPA-funded treatment facilities must be provided with an operations manual prepared under the direction of the design engineer for the specific plant. These manuals detail the type(s) of process(es) in the system, list the type and manufacturer of the equipment, and provide a verbal description of the required maintenance for the plant and equipment. Checklists for identifying and correcting the common ailments encountered in the specific treatment process employed should be included.

The levels of competency for the operating personnel of the facility are specified. The qualifications for the plant personnel are taken from the certification program from the State Health Department's requirement for certified operators. The design engineer is required by EPA mandates to provide operational assistance for a period of one year from the date of acceptance of the constructed facility and start-up operations.

The Brownfield plant personnel requirement as specified in the operations manual show a minimum level of competency for the chief operator to be a "C" certification. This is the average level for operators in the state of Texas, with "A" and "B" being higher levels of competency. Waste treatment plant operators are required to obtain professional course work specifically targeted to the operators' specific fields of practice. These courses, known locally as "short schools," are offered throughout the state to provide the necessary developmental courses to obtain certified operators. After obtaining

the necessary course work and experience record, the operator is tested by written exams to determine his level of competency.

Other requirements of plant operators who manage the system are:

1. Mechanical knowledge of the equipment, regular maintenance requirements and operational characteristics. Training should be provided to the operators by the equipment manufacturers during the start-up phase of the plant (22).
2. Self-disciplined work habits and documentation of the daily work performed (22).
3. The possession of management skills for the budgeting of monies and management of plant personnel.
4. Ability to perform all work requirements necessary within the plant.

All operations personnel should have a qualified person from whom to seek help during plant upsets.

Pre-Start-Up Phase

Operations at the Brownfield plant were initiated in November of 1982. The Operations and Maintenance Manual had been reviewed in detail prior to this point with the operators to familiarize them with the information in the manual and its location in the manual.

Requirements Placed on the Operator

The O&M manual specifically states that the operator completely understand the processes in the treatment plant (50). This is an attempt to alert the operations staff of their need to review the O&M manual and to consult others employed in similar operations to answer questions concerning the processes that are included in the treatment

system. The operators are required by state law to attend short schools for updating their technological competency.

The O&M manual also details the responsibilities of the operators and facility manager (50). The primary duty of the operations staff is to oversee and maintain the treatment facilities function of producing an effluent that meets or exceeds the permitted limits for the least operational cost. This in itself requires a thorough understanding of the design, design assumptions, budgeting constraints, personnel, state requirements and equipment capacities and capabilities.

The daily duties of the Brownfield wastewater treatment plant operators, outlined in the O&M Manual, are listed below:

1. Record daily influent flow rates, weather conditions and physical appearance of plant.
2. Perform routine mechanical equipment checks (i.e., pumps, blowers, comminuter, etc.).
3. Record inlet suction pressures and discharge pressures on the blowers.
4. Take qualitative samples and perform daily laboratory tests.
5. Record all laboratory results and file on the premises.
6. Record the duties performed daily.
7. Maintain the aesthetic appearance of the plant.

One addition to the daily maintenance routine was made. Inlet air filters were to be taken from their housings daily and cleaned and then placed back in their housings. Belt tensions were also checked on the blowers and discharge temperatures on the blowers were recorded. It was found after blower failures had occurred, that when the discharge

temperature exceeded 275°F, serious damage to the internal components of the blowers resulted. When temperatures exceeded 275°F the blower was taken off line for repair or replacement.

Operator Performance/Plant Performance Results

The effect of proper operator comprehension of the treatment process and monitoring of the process can be seen in the performance of the plant (meeting the permit requirements). The check points of the plant are the design removal efficiency determined by the consulting engineer and the regulatory agency. The permitted required performance of the Brownfield plant was for: (a) removal of 85 percent of the influent BOD_5; (b) 2 mg/l dissolved oxygen minimum concentration in each pond; (c) unlimited effluent suspended solids concentration; and (d) minimal odor emission from the treatment process. The quality data in Table 4.3 indicates that the plant has not met the design removal efficiencies.

A thorough design analysis was made of the blower system and sizing by Guy Mace of Semblex Corp., and the aeration system by Thomas N. Dobras, P.E. of Hinde Engineering Co., along with staff engineers from Parkhill, Smith & Cooper, Inc., in March of 1985. Upon completion of the analysis by these engineers, who were not familiar with the original design, it was determined that the design was very conservative and was capable of meeting permit limitations. These findings, then, indicate that operator performance was a primary source of poor plant performance.

Operator performance is a difficult item to quantify with numbers. In the study of the Brownfield plant, interviews, rather than numerical

evaluation, were used to determine the operators' knowledge of the treatment process and associated process control techniques. All of the interviewer's questions were derived from the O&M manuals provided to the operating personnel 30 days prior to the start-up of the plant. It was found that:

1. The operations personnel at the Brownfield plant did not understand the theory of the treatment process which was utilized in the treatment facility.
2. The operators did not understand the different valving techniques available to allow process flow changes.
3. Maintenance was not taken seriously and in many instances was totally neglected.
4. Most of the major qualitative control tests were performed only to provide information for report forms in order to comply with regulatory requirements.
5. Numerical values were recorded routinely with no recognition of what the numbers were indicating.

None of the operators in the Brownfield facility had a good working knowledge of the monitoring techniques or why certain parameters were being monitored. All of the operators, however, were certified by the Texas Department of Health as having successfully completed course work, which included a knowledge and understanding of laboratory techniques and testing procedures for wastewater treatment plants. It is apparent that a discrepancy exists between classroom instruction results and field operation performance.

The plant performance suffered due to the operator inconsistency in

monitoring and maintenance operations. To evaluate the performance of the treatment system, a plot of constituent parameters versus normalized flow was prepared as shown in Figure 4.1. Normalized flow is taken to be actual flow divided by design flow. Table 4.9 is a compilation of the typical data used in constructing operations performance curves. These plots can help the operator see if the plant is operating in a reasonable range based on design removal efficiencies. This becomes a tool to measure consistency, both in treatment plant reliability and operator control of the unit processes involved in the total treatment scheme.

Tables 4.10 and 4.11 show that the reliability of the Brownfield treatment system to meet design treatment quality was not accomplished routinely during the study period. Operational procedures that were utilized during the study period were the primary reason for the low reliability.

Another cause is the operators' inability to recognize that steady-state conditions are not constant at this plant. Dynamic changes occur regularly and no adjustments in the operations of the unit processes were attempted.

Dynamic changes in state are common in most waste streams, and to properly adapt the treatment system to meet these changes requires flexibility in the design of the different unit processes and a thorough understanding of the treatment process. The unit processes employed in the system must be capable of providing operational flexibility so that adjustments can be made for hydraulic detention time, solids retention time, air supply, and depth of fluid. The Brownfield plant can

accomplish all of the above with relative ease, with the exception of varying the air supply.

Researchers like Rich have indicated the different parameters that help in making the adjustments to the unit processes for dynamic changes. Hinde Engineering recommends the following parameters to be monitored for control of their system (4): (a) influent BOD_5; (b) dissolved oxygen concentrations in the treatment cells; (c) cell depth within the system; (d) blower inlet and discharge pressures; (e) orifice calcification; (f) quantity of recycle; and (g) effluent BOD_5.

Operations personnel at the Brownfield plant during the study period were not aware of the importance of these parameters, nor were they confident in their ability to control these parameters to produce the permit requirements. The overall operator performance was questionable when compared to the effluent quality produced.

The reliability of the system to produce an acceptable effluent is rated at 95 percent. This was verified at the Town of Pecos City's wastewater treatment plant, which is also a Hinde Engineering aeration system. Table 4.12 shows the operational performance of the Pecos plant. Plant performance is above average and this fact is reflected as shown in Table 4.12 with high BOD_5 removal rates, averaging approximately 65 percent for the month of June.

The Environmental Protection Agency has issued numerous publications specifically addressing staffing requirements and effluent limitations based on properly-operated wastewater treatment plants. The staffing requirement for the Brownfield plant was determined to be 1.1 men per week (1). This is not the actual staffing practiced. Two men

have been permanently assigned to the plant for the past year and one-half. The plant is over-staffed according to EPA criteria (1). The increased staffing had no noticeable effect on increasing effluent quality.

Predictive effluent quality was attempted by use of the EPA procedures (1). The method utilized by the EPA is shown in Figure 4.1. Normalized flow is unitless and is determined by equating actual average flow/average design flow (2). In data from the study results analyzed by this method, the effluent BOD_5's plotted were predominantly located in the "questionable operation" area. The conclusion drawn from this analysis is that many pertinent parameters, such as those following, are not accounted for: (a) soluble BOD_5 held in the sludge deposits; (b) varying organic loads in the effluent BOD_5; (c) short circuiting within the basins; (d) mixing caused by wind; (e) density currents and mechanical aeration; (f) varying temperatures; (g) food-to-microorganism ratios; and (h) operator error. This method should provide a reasonable estimate of effluent quality if the plant design criteria is known.

Specific Equipment Operation/ Maintenance Results

The Hinde Engineering submerged tube aeration system requires very specific maintenance performed on a regular continuing basis. The submerged tubes must be cleaned periodically by injecting anhydrous-hydrochloride into the aeration system. For West Texas water with high calcium carbonate concentrations, it was found that this cleaning process had to be done weekly. Brownfield's plant, during the study period, experienced periodic power loss to the blowers and interruption

of aeration. Power was lost five different times from January, 1983 to January, 1984. When these interruptions occur, water can enter the aeration tubing. This water can only be evacuated by supplying the submerged tubing system with the maximum volume of air capable of being produced from the battery of blowers.

Occasionally, alternating the air feed pattern from one side of the basin to the other by opening and closing air discharge valves within the blower building will force water in the tubes from one side to the other, eventually forcing the water out of the slits in the aeration tubing. This process can take up to several weeks to completely evacuate the water from the aeration tubing. The significance of water-logged tubing is an increase in the back pressure on the blowers. The increased back pressure will cause blower failure and inherently will cause a limited number of blowers to be on-line at any one time.

Routine maintenance of the blowers is required. This includes daily monitoring of oil levels, inlet suction pressures, discharge pressures, discharge temperatures and amperage/hour readings. All of these items should be recorded daily on a maintenance log. It is apparent from this study that monitoring instrumentation should be added to similar blower installations to continuously monitor discharge temperature, pre-set not to exceed 275^{o}F, and discharge pressure (< 12 psi). Values above these limits would automatically shut the blowers down.

Operational Consequences of Blower Failures

Blower failures caused operational failures with respect to the normal operation of the system. Specifically, the cells are designed to

provide the required treatment with three blowers providing continuous air supply to maintain aerobic conditions.

Brownfield's plant experienced blower failure within six months of start-up. The reported cause of complete blower failure was lack of proper maintenance. The internal parts of the blowers were reported to have sand and grit on them (27). The build-up of the sand and grit caused the seals to wear and allowed the oil to leak out, resulting in total failure of the units.

Operational problems in the treatment process resulted from the failure of the blowers. The problems that resulted in the treatment process were low DO concentrations in all the cells, high effluent BOD_5, development of odors due to lack of DO, formation of blue-green algae on one occasion, and an increase in soluble BOD_5 during the three months that the blowers were out of service.

To overcome the lack of adequate oxygen supply, operational changes were required. The changes instituted during this portion of the study period are listed below:

1. The plant was put into parallel operation to distribute the organic load.
2. Recycle was increased from the effluent of cell 3 to the influent stream.
3. Wash-down hydrants were fitted with spray nozzles and water from the cells was recirculated through the main irrigation pump structure and sprayed back into the air over the surface of the cells.

The result of these efforts indicated that the parallel operation provided the best treatment during emergency situations when adequate

air supply is not available. The parallel operation was instituted for a period of approximately two months. The results obtained are shown in Table 4.13. The operators did not allow the system to be operated in parallel for more than two or three days at a time during the two-month period. The data is not conclusive enough to predict that parallel operation will correct problems associated with the inability to supply the proper oxygen to the cells. However, for the period of parallel operation obtained in this study, parallel operation did improve the effluent quality over that realized with the normal series operation.

Parallel operation also did bring about an improvement in the odor condition. Again, the improvement of the odor condition was only for the two-month period. The odors from the cells were substantially reduced from those noticed before the plant was placed in parallel operation. The method used in determining the degree of noticeable odors was through a review of the reports received from the residents living in the community.

Dissolved oxygen increased with parallel operation. Dissolved oxygen levels showed an increase in cells 1 and 2 over the series operation.

Operators must be able to recognize the problems associated with declining DO levels and respond to the changing situation. As described for this plant, series operation with low DO levels (below 1 mg/l) did show an increase when the plant was operated in parallel. The sooner the plant can be put into parallel operation, the more likely the plant operations will stabilize and respond positively by showing an increase in DO levels. Odors should diminish with parallel operation.

Additional research should be conducted utilizing parallel operation of the lagoons with low DO values in the cells to verify that this mode of operation will in fact support the results obtained at the Brownfield plant for the two-month study period.

TABLE 4.1

DESIGN ASSUMPTIONS FROM THE FACILITY PLAN
FOR THE BROWNFIELD TREATMENT PLANT

Parameter	Assumed Value
Influent BOD	195 mg/1
Influent Suspended Solids	200 mg/1
K_e @ 10°C	0.18
K_e @ 30°C	0.35
% BOD Removal Required	85%

TABLE 4.2

AVERAGE MEASURED BOD_5 VALUES FROM SERIES TREATMENT
IN mg/1 FROM JANUARY 1984 TO DECEMBER 1984

Month	Influent BOD	Pond 1 Eff. BOD	Pond 2 Eff. BOD	Pond 3 Eff. BOD	% Rem.
Jan. 84	98	131	85	62	37
Feb.	202	238	187	182	10
Mar.	216	209	149	118	45
Apr.	277	277	272	275	1
May	108	60	62	37	66
June	57	71	50	76	-33
July[a]	0	0	0	0	0
Aug.	115	96	55	98	15
Sept.	119	87	43	27	77
Oct.	189	103	63	46	76
Nov.	175	88	59	32	82
Dec.	246	180	145	117	53
Jan. 85					

[a]No data available.

TABLE 4.3

AVERAGE ORGANIC REMOVAL PERCENTAGES IN EACH POND IN SERIES OPERATION FROM DECEMBER 1983 TO DECEMBER 1984

Month	Influent BOD (mg/l)	Pond 1 BOD_5 (%)	Pond 2 BOD_5 (%)	Pond 3 BOD_5 (%)
Dec. 83	103	1	36	34
Jan. 84	98	-34*	35	27
Feb.	202	-18*	21	3
Mar.	216	3	29	21
Apr.	277	0	2	-1*
May	108	44	-3*	40
June	57	-25*	30	-52*
July	0	0	0	0
Aug.	115	17	43	-80*
Sept.	119	27	49	39
Oct.	189	46	39	27
Nov.	175	50	34	46
Dec.	247	27	19	19

*Indicates an increase in soluble BOD in the effluent above that of the influent to the pond in series.

TABLE 4.4

CALCULATED REMOVAL EFFICIENCIES FOR THE WINTER MONTHS

Month	Combined % Removal (Series Oper.)	Total Removal Eff. (Inf.-Eff./Inf.) X 100
Dec. 83	24	58
Jan.	9	37
Feb.	2	10
Mar.	18	45
Nov.	43	82
Dec. 84	21	53

TABLE 4.5

CALCULATED REMOVAL EFFICIENCIES FOR THE SUMMER MONTHS

Month	Combined % Removal (Series Oper.)	Total Removal Efficiency (Inf. - Eff./Inf.) X 100
June	-16	-32
July[a]	-	-
August	-6	14
September	38	77

[a]No data available.

Minus values indicate an increase in soluble BOD_5 in the effluent.

TABLE 4.6

SUMMARY OF TEST DATA
(AVERAGED FOR 12-MONTH PERIOD)

Flow MGD	774,871
Influent BOD_5, mg/l	159
Pond 1 BOD_5	137
Pond 2 BOD_5	103
Pond 3 BOD_5 (effluent)	93
Influent TSS, mg/l	110
MLSS, mg/l (Pond 1)	85
DO Influent, mg/l	1.5
DO Pond 1	0.4
DO Pond 2	1.2
DO Pond 3 (Effluent)	2.9
Temperature ^{o}C Influent	20.2
Pond 1	16.8
Pond 2	16.3
Pond 3	16.2

TABLE 4.7

COMPARISON OF DESIGN PARAMETERS,
ACTUAL VS. DESIGN

Parameter	Actual*	Design
k_p Winter	0.19	0.18
Summer	0.32	0.35
F/M	0.82	Unknown
Y	Unknown	Unknown
L_T	173.9	Unknown
L_1	106.5	Unknown
DO Levels	2.9	4.0
S_e	93	3.0
BOD_5	158.9	195
MLSS	173.9	Unknown
SRT	80.2	Unlimited; designed for 20 years of operation & storage

*All values are average values over the period of record.

TABLE 4.8

DESIGN VALUES RECOMMENDED BY RICH

Parameter	Recommended Value	Definition
K_p or K_d	0.12 to 0.2 $days^{-1}$	(specific decay rate)
	13 $days^{-1}$	(specific growth rate)
Y	0.5	(growth yield)
K_s	120 mg/l	(saturation constant)
BOD_5	pilot tests	
L_T	pilot tests	
S_e	pilot tests	
SRT	hydraulic retention time	

TABLE 4.9

NORMALIZED FLOW RATES

(1) 1984 Months	(2) Design Flow Mgd	(3) Actual Monthly Average Mgd	(3)/(2) Normalized Flow Mgd	Effluent BOD_5 Conc. mg/l
J	1.25	0.780	0.624	62
F	1.25	0.709	0.567	182.5
M	1.25	0.679	0.543	118
A	1.25	0.688	0.550	275
M	1.25	0.756	0.605	37.3
J	1.25	0.649	0.519	76.3
J	1.25	0.885	0.708	0
A	1.25	0.993	0.794	98.7
S	1.25	0.949	0.759	27.5
O	1.25	0.802	0.642	46.0
N	1.25	0.659	0.527	32.5
D	1.25	0.654	0.523	117.5

TABLE 4.10

JUNE 1984 TABULATION OF NORMALIZED FLOW AND EFFLUENT BOD_5 (BROWNFIELD, TEXAS)

(1) Day of Month	(2) Design Flow Mgd	(3) Actual Flow, Mgd	(3)/(2) (4) Normalized Flow, Mgd	(5) Effluent BOD_5 mg/l	(6) BOD_5 % Removal
1	1.25	0.840	0.672		
2	1.25	0.949	0.759		
3	1.25	1.035	0.878		
4	1.25	1.059	0.847	60	0
5	1.25	1.097	0.878		
6	1.25	1.073	0.858		
7	1.25	0.924	0.739		
8	1.25	1.055	0.844		
9	1.25	0.108	0.086		
10	1.25	1.113	0.890		
11	1.25	1.116	0.893	105	-57*
12	1.25	1.023	0.818		
13	1.25	1.049	0.839		
14	1.25	0.953	0.762		
15	1.25	1.044	0.835		
16	1.25	1.112	0.889		
17	1.25	1.050	0.840		
18	1.25	0.994	0.795	80	16
19	1.25	1.081	0.865		
20	1.25	1.074	0.859		
21	1.25	1.000	0.800		
22	1.25	1.090	0.872		
23	1.25	1.045	0.836		
24	1.25	1.083	0.866		
25	1.25	0.995	0.796	60	-1.0*
26	1.25	1.115	0.892		
27	1.25	1.113	0.890		
28	1.25	0.999	0.799		
29	1.25	1.050	0.840		
30	1.25	0.988	0.790		

*Minus indicates an increase in soluble BOD_5.

TABLE 4.11

ANNUAL BOD_5 REMOVAL RELIABILITY, 1984

	% of Time Less Than or Equal to 30 mg/l	% of Time Less Than or Equal to 40 mg/l
Pond 1	0	0
Pond 2	0	0
Pond 3	17	33

TABLE 4.12

JUNE 1984 TABULATION OF NORMALIZED FLOW
AND EFFLUENT BOD_5
(TOWN OF PECOS CITY)

(1) Day of Month	(2) Design Flow Mgd	(3) Actual Flow, Mgd	(3)/(2) (4) Normalized Flow, Mgd	(5) Effluent BOD_5 mg/l	(6) BOD_5 % Removal
1	1.5	0.526	0.351		
2	1.5	0.455	0.303		
3	1.5	1.201	0.801		
4	1.5	1.004	0.669	27.0	67
5	1.5	0.560	0.373		
6	1.5	0.568	0.378	29.0	63
7	1.5	0.443	0.295		
8	1.5	0.507	0.338		
9	1.5	0.450	0.300		
10	1.5	0.410	0.273		
11	1.5	0.436	0.291	29.0	58
12	1.5	0.561	0.374		
13	1.5	0.515	0.343	31.0	67
14	1.5	0.557	0.371		
15	1.5	0.579	0.386		
16	1.5	0.503	0.335		
17	1.5	0.576	0.384		
18	1.5	0.480	0.320	33.5	67
19	1.5	0.407	0.938		
20	1.5	0.375	0.250	26.0	77
21	1.5	0.248	0.165		
22	1.5	1.333	0.887		
23	1.5	2.083	1.388		
24	1.5	0.379	0.253		
25	1.5	0.356	0.237	27.0	70
26	1.5	0.397	0.265		
27	1.5	0.366	0.244	23.0	69
28	1.5	0.268	0.178		
29	1.5	0.414	0.276		
30	1.5	0.208	0.138		

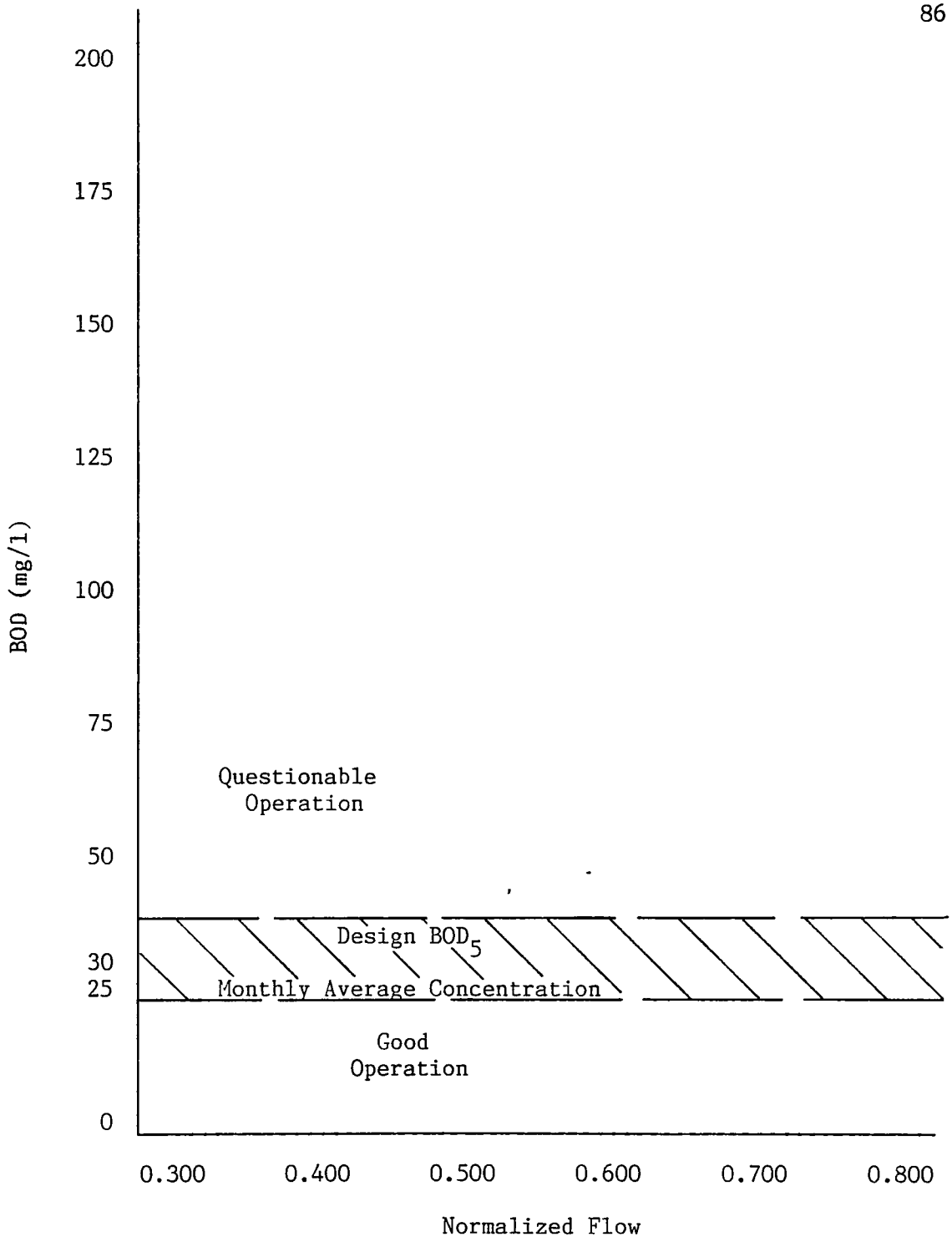


FIGURE 4.1: Normalized flow, mgd.

SOURCE: Adapted from United States Environmental Protection Agency, Estimate of Effluent Limitations to be Expected from Properly Operated and Maintained Treatment Works, 1979.

TABLE 4.13

AVERAGE MEASURED BOD_5 VALUES FROM PARALLEL OPERATION FOR A TWO-MONTH PERIOD, 1984 (BROWNFIELD, TEXAS)

Month	Influent BOD (mg/1)	Pond 1 Eff. Bond (mg/1)	Pond 2 Eff. Bond (mg/1)	Pond 3 Eff. Bond (mg/1)	% Rem.
October	186	NM	NM	43	77
November	196	NM	NM	28.3	85

NM = No measurement taken.

CHAPTER V

CONCLUSIONS

The following conclusions regarding the design equations utilized in the design of the Brownfield Wastewater Treatment Plant and the operational analysis of the plant are presented in this study:

1. The design equations utilized in the Brownfield design are acceptable equations for the design of a partially mixed three-cell aerated lagoon if the correct coefficients are used.
2. The size and operational capabilities provided in the design of the Brownfield wastewater treatment facility are sufficient to meet permit requirements if the operational and maintenance guidelines contained in the facility's Operation and Maintenance Manual are followed.
3. Changes in the operational procedures followed at the Brownfield wastewater treatment facility must be implemented if the facility is to meet permit requirements since the data recorded during the study period exhibited unacceptable levels of BOD_5 and TSS in the plant effluent when flows were less than half of the design flow.
4. The wastewater treatment system installed at the Brownfield facility is not a simple system and requires a greater degree of operator training and supervision than was provided during the start-up phase of the plant.
5. Low dissolved oxygen levels in the cells can be improved by changing from series operation of the cells to parallel operation.
6. The benthal oxygen demand in the cells of the Brownfield plant was increased by the greater sludge bed depths that resulted from

increasing the pond depths and decreasing the surface area required for the system.

7. Blower failure resulted from lack of maintenance to inlet air filters and increased discharge pressures, which increased the temperature across each blower from inlet to outlet.

Recommendations

As a result of this study, the following recommendations regarding a submerged tube, partially mixed treatment facility are made:

1. Design engineers should check on performance by being on-site for test runs during start-up operations and seeing that the operators follow a daily schedule for process control testing. Process training should be initiated prior to completion of the treatment facility and should emphasize the flexibility designed into the treatment facility.
2. Three-cell aerated lagoons should be supervised closely to ensure plant operations are within design expectations and that maintenance duties are detailed for all mechanical equipment.
3. The positive displacement blowers should be equipped with alarm sensors to shut down the individual units if pre-set values for temperature and back pressure are exceeded.
4. Wastewater streams should be characterized in the pre-design phase in place of assuming typical design concentrations for domestic wastewater published in the literature. Bench scale treatability studies should be conducted to verify the best treatment process for the level of funding provided.
5. Seeding of the primary cell at start up with waste-activated sludge

should be provided. This seed should be aerated a minimum of one week prior to accepting influent flow into the treatment facility.

6. Header piping should be affixed with screwed caps at the ends to allow purging of aeration header piping.

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