Water quality modeling on the Skunk River

by

Gerald Douglas Groth

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INTRODUCTION

Overview

The State of Iowa has established water quality criteria for Iowa's rivers and streams to ensure that water of a specified quality will be available for its expected use. Maintaining these criteria may require the limitation of wastewater effluent discharges to these waterways, which otherwise may violate the established limits. The State of Iowa follows a Waste Load Allocation (WLA) procedure in setting these limits which may use mathematical models in its implementation. The mathematical models simulate water quality in a stream or river in response to an expected waste discharge.

Since 1975, the State of Iowa has utilized one rather simple water quality model to simulate the criteria most often violated in the State. The model was developed by Stanley Consultants, Inc. in 1975, and is characterized by its use of the modified form of the Streeter-Phelps equation for calculating dissolved oxygen deficits.

The ability of this model to accurately simulate water quality has been questioned because of its simplistic nature. As a result, the accuracy of the Waste Load Allocations have also been questioned.

WLAs which are too conservative may result in the design, construction, and operation, of treatment facilities in excess of what is actually needed. Baumann (Department of Civil Engineering, Iowa State University, personal communication, 1983) estimated the present worth costs of these excess facilities to be as high as 100 million dollars.

Initial Objectives

The initial objectives of the study were:

1. To establish how the State of Iowa models water quality parameters,

2. To perform a sensitivity analysis on the model to find out how model input will affect model output, and

3. To apply the model to the Skunk River at Ames (where a new treatment facility will be constructed around 1987) and to the Des Moines River, southeast of the City of Des Moines.

These initial objectives were altered however, during the course of the research and resulted in establishing a set of revised objectives. A major reason for the revision occurred in 1983, when it was discovered that the State of Iowa had changed their modeling procedure. The changes involved several modifications to the original water quality model, as well as, proposing the ultimate use of a more sophisticated model. Other reasons for revising the objectives were varied, but generally were made to arrive at more rewarding results.

Revised Objectives

The revised objectives of this thesis are:

1. To document how the State of Iowa models, or proposes to model, water quality parameters,

2. To perform a sensitivity analysis on the original and modified form of the less sophisticated water quality model,

3. To establish an initial data base for future in-depth modeling

exercises,

4. To confirm and further explain previously obtained sampling data, and

5. To apply the model to the Skunk River at Ames with sampling data for model calibration or curve fitting.

The first four revised objectives are completely addressed in this thesis. The fifth revised objective, however, is only partially addressed, due to the complexities involved in calibration and verification of a model.

RESEARCH PLAN AND METHODOLOGY

The following plan of study was undertaken to satisfy the revised research objectives and consisted of five separate steps.

The first step in the research plan involved the evaluation of the basic characteristics of the Skunk River basin. The evaluation primarily consisted of reviewing published material on the Skunk River, with additional data collection when necessary. The basin characteristics evaluated in the study included geological, physical, hydrological, limnological, historical and other general aspects. A map of the Skunk River basin near Ames is shown in Figure 1 and delineates major items of interest, which are referenced throughout this thesis.

Additional data collection involved several field reconnaissance trips down the Skunk River, discharge measurements using one of the Civil Engineering Department's current-meters, and the analysis of river mileage and slope from 7 1/2 minute USGS (United States Geological Survey) contour maps.

Field reconnaissance for familiarization with the study area typically involved driving to river access points and observing items of interest. These items included depth and width of flow, overland surface flow contributions, and percentage of ice cover. The field reconnaissance also included a canoe trip from just north of U.S. Highway 30 bridge (south of Ames) to Cambridge, Iowa. The canoe trip and the current-meter discharge measurements required the use of additional help due to the difficulties involved in transporting and using the necessary

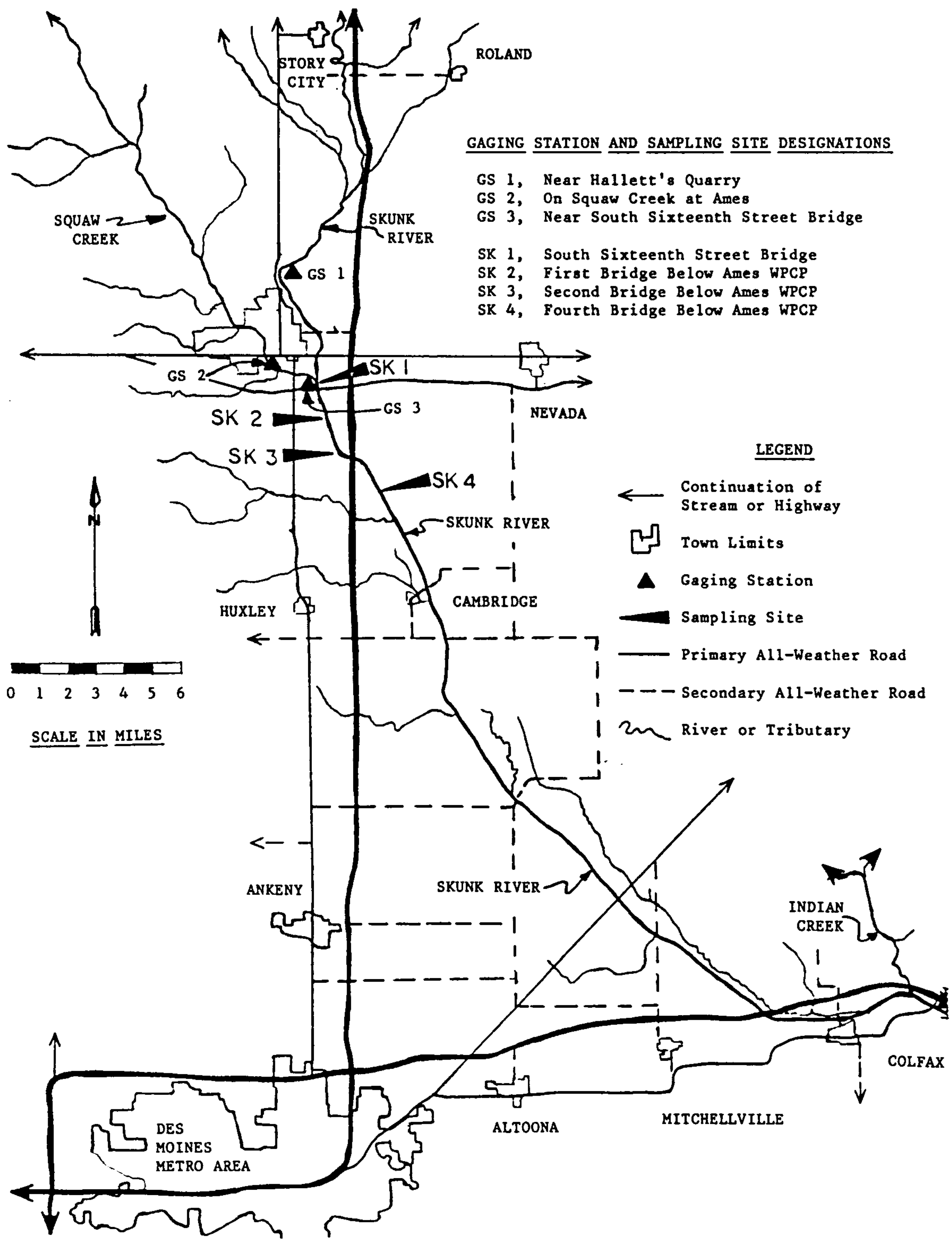

Figure 1. Vicinity map of study area

equipment. All other field reconnaissance was accomplished without assistance. Notes and pictures accompanied most field reconnaissance trips.

Discharge measurements were made using a 6-cup current-meter manufactured by "W. and L. E. Gurley" (Model number 622). The procedures followed in taking the current-meter discharge measurements are described in detail in several hydrology textbooks, such as Linsley et al. (1979) or Chow (1964). Briefly, however, the measurements were made by lowering the current-meter with an attached weight and cable into the flowing water from the bridge sampling site. Revolutions of the conical cups were timed and a corresponding velocity established from a rating table. Discharge was established by multiplying the average velocity, obtained at six-tenths depth, by the width of measurement (usually 5 to 10 ft). Water depths were indirectly measured knowing the length of cable played out from the water surface to the river bottom. Time was measured using a wind-up stop watch.

The discharge measurements were intended to document the linear trend of groundwater contributions to the Skunk River below Ames, which was assumed by Dougal (1969). Appendix A contains the data obtained from four discharge measurements performed on July 28th, 1983 with the assistance of C. S. Oulman. The river mileage referenced in Appendix A can be obtained from the mileage comparison in Table 6. The four sites are also shown in Figure 1 as SK1, SK2, SK3, and SK4, respectively.

Mileage between sampling points was checked using the 7 1/2 minute topographic contour maps. The maps have a scale of 1"=2000' with contour intervals of 10 feet and are commonly referred to as "quadrangle" maps.

Seven maps were obtained for the analysis and covered the Skunk River area from South Ames to just east of Colfax, Iowa. Names of the quadrangle maps used were the Ames East, Huxley, Elkhart, Loring, Altoona, Mitchellville, and Colfax, Iowa maps. River slope was checked by dividing elevation differences with river mileage.

River mileage was measured between sampling points designated in Dougal's (1969) study, from Ames to Colfax, using an engineer's scale in most instances. The sinuous portion of the river immediately below Ames, was measured with an Alvin map wheel.

The second step of the research plan involved the collection of sampling data. Sampling data were obtained to update previously collected data and to further explain stream phenomena. Specific objectives of the additional sampling were to obtain dissolved oxygen (D.O.), organic and inorganic nitrogen, carbonaceous, and chlorophyll (or pigment) data.

Four separate sampling trips were taken during the research period. The frequency and number of sampling trips were primarily influenced by the river's discharge rate, since periods of <u>low</u> flow were of significant interest. Discharge rates in excess of 150 to 200 cubic feet per second (cfs), south of Ames, were considered to be limiting flow rates for sampling.

Table 1 lists the dates and scope of each of the four sampling trips. Low flow conditions with ice cover were not encountered during the research period.

D.O. diurnal studies consisted of collecting D.O. samples at least once every two hours at each of two locations on the river. One sampling

Table 1. Dates and objectives of water quality sampling trips

Trip Dates and Time	Approximate Discharge at South Sixteenth Street Bridge[a]	Sites Sampled	Research Objectives
7/28/83 from 8:30 a.m. to 2:00 p.m.	182 cfs	SK1, SK2, SK3, and SK4	Current meter discharge measurements and preliminary D.O. profile study
8/4/83 from 9:50 a.m. to 8/5/83 at 2:40 a.m.	156 cfs	SK1 and SK2	D.O. diurnal study
9/14/83 from 1:40 p.m to 2:45 p.m.	41 cfs	SK1, SK2 SK3 and SK4	D.O., organic and inorganic nitrogen, carbonaceous BOD, and chlorophyll profile study
10/6/83 from 6:05 a.m. to 10/7/83 at 1.35 a.m.	109 cfs	SK1, SK2, SK3 and SK4	Complete profile study as indicated above plus D.O. diurnal for SK1 and SK3

[a]Obtained from analysis of combined upstream gaging station preliminary discharge as presented in the evaluation of discharges section of this thesis.

site was chosen upstream of the Ames Water Pollution Control Plant (WPCP) discharge location, in a relatively "clean" stream environment, with the other site chosen downstream from the plant, near the critical oxygen deficit location. This provided an indication of the diurnal effects both upstream and downstream of a pollutant source.

All D.O. samples were obtained using a D.O. dunker lowered from a bridge sampling site into the main portion of the river's flow. The D.O. dunker contained space for two 300 milliliter (ml) BOD (Biochemical Oxygen Demand) bottles, permitting each to be filled twice by overfilling from the bottom of the bottles. Two D.O. samples were obtained from most sampling sites using only one bottle per filling. The remaining water was used for subsequent testing or returned to the river. D.O. analyses were done using the Azide modification to the Winkler D.O. method as described in Standard Methods by American Public Health Association (APHA, 1981). Samples for D.O. determinations were immediately fixed in the field prior to titration, which was either performed in the field or back in the laboratory.

D.O. profile studies were conducted to examine the river reach below the treatment plant. Four sampling sites were chosen in the profile studies, beginning 0.37 miles above the Ames WPCP effluent discharge (sampling site SK1) to about 5.0 miles below the discharge (sampling site SK4). These sites are shown in Figure 1 and described in more detail in Table 6. The D.O. profile studies were sampled in a downstream direction and were collected in a manner identical to the diurnal studies. The time elapsed for sample collection during the profile studies were generally less than one hour.

Two of the four sampling trips were primarily used to obtain organic and inorganic nitrogen data. Other useful data obtained at that time included carbonaceous BOD, air and water temperatures, chlorophyll, and dissolved oxygen data.

Grab samples were obtained in a manner similar to the D.O. profile studies utilizing the D.O. dunker from the desired sampling location. With the exception of the D.O. samples and temperature data, all other analyses were performed by the Engineering Research Institute's Analytical Services Laboratory (ASL). Table 2 shows the analytical methods employed for each sampled parameter.

Samples were returned to the laboratory for analysis as soon as possible after the last site. Typically this was less than an hour and a half after beginning the sampling run. In spite of the brief time delay, the samples were stored on ice until they were returned to the ASL. The NH_3 samples were additionally preserved with sulfuric acid to a pH less than 2 as prescribed in Standard Methods (APHA, 1981).

Chlorophyll data were obtained by filtering a known quantity of water though a 0.45 micron Millipore filter, subsequently drying the filter with desiccant, and then freezing the filter paper for a period of at least seven days. A $MgCO_3$ solution was added to each water sample prior to filtration.

BOD tests measured carbonaceous BOD only, as nitrification was inhibited in all the samples. Samples for BOD determinations were stored in a dark incubator for a period of 5 days at $20^{o}C$.

The third step of the research plan involved documenting how the State modeled water quality parameters and how they subsequently

Table 2. Methods used in the analysis of water quality parameters

Water Quality Parameter	Method	Reference
Biochemical Oxygen Demand	Incubation, 20°C, 5 days nitrification inhibited	SM[a], p. 483
Kjeldahl nitrogen	Phenate method, semi-automated block digester	MCA[b], p. 351.2
Ammonia nitrogen	Phenate method, automated	MCA, p.350.1
Nitrite and nitrate nitrogen	Cadmium reduction method, automated	MCA, p.353.2
Phytoplankton pigments	Spectrophotometric determination	SM, p. 950, 953
Temperature	Mercury-filled Celsius thermometer, -20 to +110°C	SM, p. 124
Dissolved Oxygen	Winkler, azide modification	SM, p. 390

[a] Standard Methods for the Examination of Water and Wastewater (APHA, 1981).

[b] Methods for Chemical Analysis of Water and Wastes (EPA, 1979).

established WLAs. This step involved researching various State documents and meeting with State employees.

The sensitivity analysis was a fourth step of the research plan and involved computer input of various parameter values to see how model output was affected. An interactive program written in BASIC computer language was set up on Iowa State University's VAX computer system. Terminals in Town Engineering were used in accessing the computer program. This program was also transferred to the Civil Engineering's Apple II+ computer and saved on diskette for future use.

Model calibration or "curve fitting" of the obtained data consisted of the fifth research step and was performed in a manner similar to the previous sensitivity analysis step. This step however, had a fixed set of output conditions, which were to be arrived at through input manipulation.

REVIEW OF LITERATURE

Mathematical Modeling in General

Mathematical modeling involves the use of mathematical relationships to simulate or describe observed phenomena. Water quality models attempt to reproduce observed conditions in a river or stream system, but are incapable of predicting the exact response of a stream. The exact response is nearly impossible to obtain due to the difficulties involved in understanding all of the natural processes in a river system and the inherent inability to account for all of the processes, even if they were completely understood. Consequently, inexact models are used to simulate only those factors having the greatest impact on the water quality components being modeled.

While models are developed from analysis of past data, they may be used to predict what stream quality parameters will be like under future stream and effluent conditions. The Waste Load Allocation (WLA) process uses these predictions to establish effluent quality limitations for various parameters, while maintaining minimum water quality criteria.

A complete review of all the models and modeling techniques available for predicting stream response to effluent dischargers would be too complex and lengthy for this thesis. Consequently, only a review of the major historical and local studies will be presented. Prior to that, however, a theory review of those parameters which have the greatest impact on stream modeling in the Skunk River near Ames will be presented.

Theory Review

Four main areas to be covered in the theory review include reaeration and oxygen resources, biochemical deoxygenation, biostimulation and algal uptake, and ammonia toxicity.

Reaeration and oxygen resources

Streeter (1924) listed three sources of oxygen supply to a stream, where oxygen was used in aerobic decomposition of organic material. These three sources included:

1. Oxygen from dilution waters,
2. Oxygen from reaeration by the atmosphere, and
3. Oxygen from biological reoxygenation (or photosynthesis).

The amount of oxygen in dilution waters must be considered when modeling. TenEch Environmental Consultants, Inc. has found that dissolved oxygen concentrations in receiving waters can play an important role in establishing Waste Load Allocations (TenEch, 1978a).

The oxygen concentration of dilution waters primarily depends on the solubility of oxygen, if the water is in a relatively unpolluted state. Babbitt and Baumann (1958) listed several factors which affected the solubility of oxygen in water and its associated rate of replenishment. These factors included temperature, atmospheric pressure, turbulence, percentage of oxygen in the atmosphere, exposed water surface area, salinity, concentration of dissolved solids, photosynthetic activity, and pollution effects.

The solubility of oxygen is also used in establishing rates of reaeration (or reoxygenation), as shown below:

$$dC/dt = K_2(C_s - C)$$

where,

C = concentration of oxygen (mg/l),

C_s = saturation concentration of oxygen at a given temperature (mg/l),

K_2 = reaeration rate constant (1/day), and

t = time (days).

Various researchers have determined saturation values for dissolved oxygen (D.O.), often in contradiction with one another. The latest effort at the determination took painstaking measures to establish saturation-temperature relationships and may be found in Standard Methods (APHA, 1981). The book also lists the appropriate equations that are used in correcting the saturation value for atmospheric pressure and chloride (salinity) concentrations. An equation for the saturation value of dissolved oxygen at various temperatures was developed by Elmore and Hayes (1960) for zero percent salinity and one atmosphere of pressure. The equation is as follows:

$$C_s = 14.652 - 0.41022\ T + 0.0079910\ T^2 - 0.000077774\ T^3$$

where

T = water temperature in ^oC.

This equation has also been converted to temperatures in ^oF, as well as incorporating salinity effects (IDEQ, Iowa Department of Environmental Quality, 1976 and Zison *et al.*, 1978).

Reaeration in streams has been investigated by many researchers including Streeter (1924), Streeter and Phelps (1925), Theriault (1927), Fair and Geyer (1954), Langbein and Durum (1967), and Dougal (1969).

Reaeration studies prior to 1960, were principally performed on larger rivers with large pollutant loads. Commonly reported values for K_2 in these studies are presented in Table 3.

Table 3. Reaeration rate constants reported prior to 1960[a]

Stream type	K_2 at 20° C, base 10[b]
Small ponds and backwater	0.05 to 0.10
Sluggish streams and large lakes	0.10 to 0.15
Large streams of low velocity	0.15 to 0.20
Large streams of normal velocity	0.20 to 0.30
Swift streams	0.30 to 0.50
Rapids and waterfalls	0.50 and greater

[a]Source: Babbitt and Baumann (1958).

[b]K_2, base 10 = (1/ln 10) k_2, base e.

Most equations developed to predict reaeration rates involve flow velocity and depth as the main input variables. Other variables employed have included wind velocity, molecular diffusivity of oxygen, and kinematic viscosity, as summarized by Zison _et al_. (1978).

Langbein and Durum (1967) demonstrated that K_2 was influenced more by depth than by velocity. Their analysis resulted in an equation for K_2 as shown below:

$$K_2 = \frac{3.3\ v}{H^{1.33}}$$

where

v = mean velocity of the stream (feet/sec),

H = mean depth of stream (feet), and

K_2 = reaeration rate at 20°C, base 10 (1/day).

Other results obtained from Langbein and Durum (1967) included the following:

1. K_2 is less for large rivers than small rivers, despite the greater velocity of the large rivers,

2. K_2 decreases in the downstream direction at a 0.43 power of the discharge,

3. K_2 decreases at a specific location at a 0.13 power of the discharge,

4. Lesser stream slopes, characteristic of populated areas have low K_2 rates,

5. K_2 rates may increase slightly in pool sections, but decrease rapidly in riffles as the stage rises, and

6. Maximum assimilative capacity occurs in rivers or streams of intermediate size, such as those of sixth or seventh order.

Traditionally the reaeration rate has been evaluated by a mass balance approach where all the parameters except K_2 are measured, leaving K_2 to be arrived at by back calculation. Other methods used in evaluating K_2 include the productivity methods of Hornberger and Kelly (1975), which was adapted form the work of Odum (1956), and a radioactive tracer technique developed and applied by Tsivoglou (1972), Tsivoglou et al. (1965, 1968), and Tsivoglou and Wallace (1972).

The latter method was used by Foree (1976) in predicting reaeration in small streams. The following predictive equation for K_2, is commonly referred to as Tsivoglou's equation.

$$K_2 = C \triangle h/t$$

where

K_2 = reaeration coefficient at $20^{o}C$, base e (1/day),

C = Tsivoglou's gas escape coefficient (1/feet),

$\triangle h$ = change in water surface elevation (feet), and

t = time (days).

Wide fluctuations in diurnal oxygen levels have been attributed in part to photosynthesis (Goldman and Horne, 1983). Modeling photosynthetic oxygen production has been accomplished by either simulating algal growth (then relating oxygen production to the algal growth) or by simply including a term for the oxygen production without algal growth simulation.

The latter method has been employed by O'Connell and Thomas (1965) on the Truckee River near Reno, Nevada. Results of this study showed that diurnal oxygen curve analysis or direct measurement of net photosynthesis in algal chambers can be used to predict daily minimum D.O. concentrations in streams. O'Connor and Di Toro (1970) used an indirect method for simulating oxygen production by representing photosynthetic production with a half cycle sine wave. While both approaches have produced reasonable results, each one requires extensive stream sampling and may incorporate errors in the final value due to poor estimation of the other required parameters. Consequently, the modeling of algal growth has been seen as a method to estimate oxygen production.

Zison *et al.* (1978) summarized many of the approaches used to model algal growth. A complete review of algal growth modeling, however, is beyond the scope of this thesis. Briefly, the main factors found to influence algal growth were phytoplankton type, nutrient availability, light intensity, light duration, and temperature. Typically, Michaelis-Menton growth kinetics are used in this approach.

Biochemical deoxygenation

The fact that microorganisms are involved in the biochemical oxidation of organic compounds in wastewater was discovered by Dupre in France near the turn of the century (Phelps, 1944). Many British and American investigators have contributed to the knowledge of biochemical deoxygenation of receiving waters since then. The work of Hommon and Theriault in 1927 led to the conclusion that biochemical deoxygenation is a result of two separate stages of oxygen demand (Theriault, 1927). These two-stages are ideally shown in Figure 2 and consist of separate carbonaceous and nitrogenous oxygen demands.

Carbonaceous deoxygenation, as the name implies, involves the microbial decomposition of carbon containing organic matter. Nitrogenous deoxygenation involves the microbial breakdown of nitrogen containing compounds. These nitrogen containing compounds are sources of ammonia (NH_3 or NH_4^+) which can be oxidized to nitrite (NO_2) and

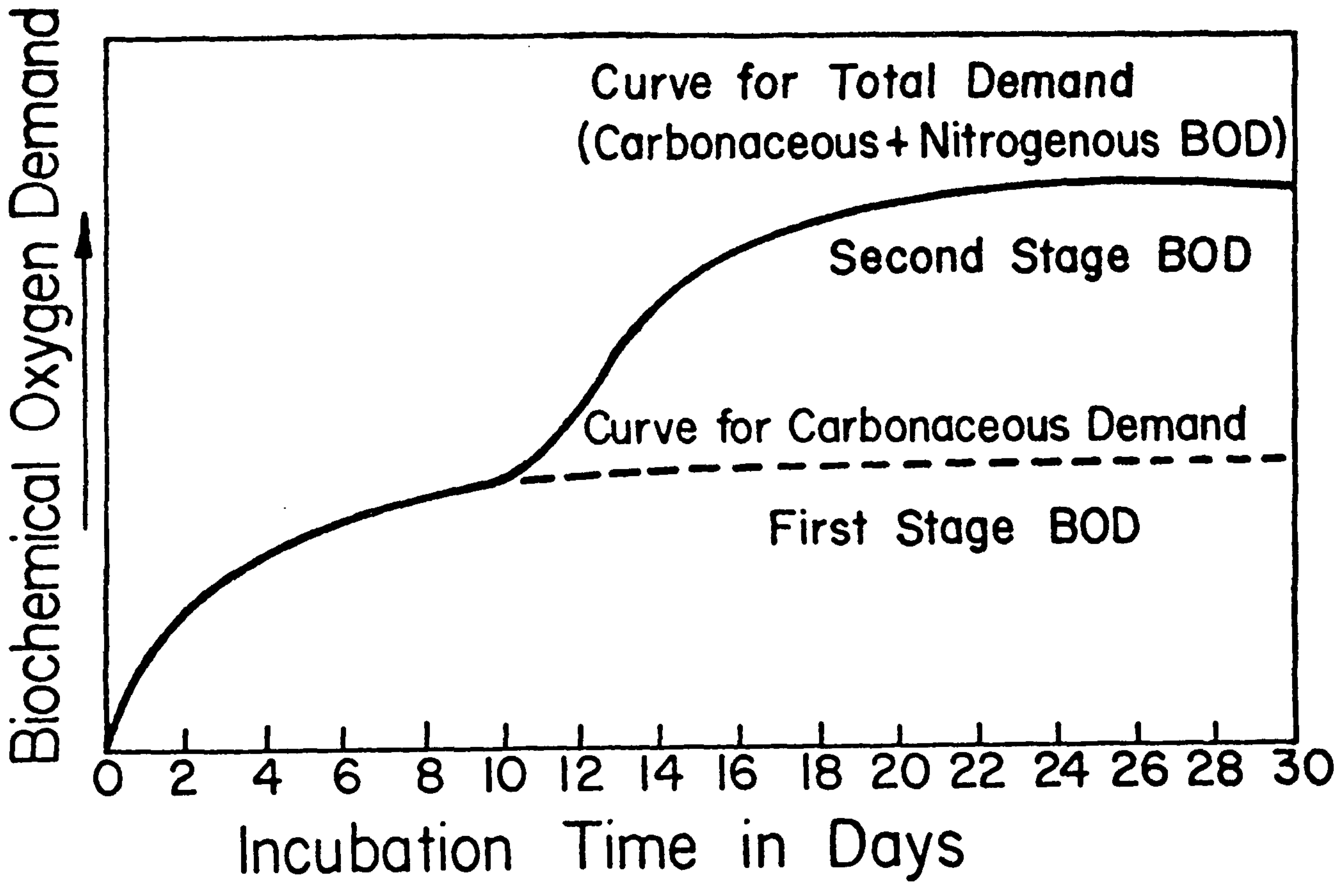

Figure 2. The two-stage curve for biochemical oxygen demand (Dougal, 1969)

nitrate (NO_3). The conversion step to nitrite and nitrate is commonly referred to as nitrification. The equations for nitrification along with the commonly accepted genera of bacteria accomplishing each transformation are:

$$NH_4^+ + 3/2\ O_2 \xrightarrow{\text{Nitrosomonas}} 2H^+ + NO_2^- + H_2O$$

$$NO_2^- + 1/2\ O_2 \xrightarrow{\text{Nitrobacter}} NO_3^-$$

A total equation showing the complete transformation is:

$$NH_4^+ + 2O_2 \xrightarrow{\text{(Both nitrifiers)}} NO_3^- + 2H^+ + H_2O.$$

Based on the last equation, 2 moles of O_2 are required to nitrify 1 mole of NH_4^+. Expressed in units of mass, [2(32)/14] on 4.57 mg of O_2 are required per mg of NH_4^+-N converted.

Gannon and Wezernak (1967) found that the theoretical value of 4.57 was too high, since the organisms can obtain oxygen from the synthesis of CO_2 in the atmosphere. Their studies indicated a more reasonable value to be 4.33.

The most common way of obtaining biochemical deoxygenation data is the standard five day laboratory BOD (Biochemical Oxygen Demand) test at 20^oC (APHA, 1981). Numerous methods exist for obtaining carbonaceous deoxygenation data through the inhibition of nitrification, as discussed by Young (1973). Nitrogenous deoxygenation data have been obtained indirectly from BOD tests taken to ultimate values where nitrification was not suppressed. More commonly nitrogenous deoxygenation data are based on the concentration of ammonia converted to an oxygen demand using the 4.57 or 4.33 factor.

Because of the time lag (greater than 5 days) leading to the nitrogenous stage, the 5 day BOD test was once considered to be nearly equivalent to the carbonaceous oxygen demand only. Sawyer and Bradney (1946), however, provided overwhelming evidence that demonstrates the error in making this assumption by demonstrating that the nitrogenous stage may occur without a time lag, depending on the waste sample's initial concentration of nitrifying bacteria.

Zison et al. (1978) listed many factors known to affect the rate of biochemical deoxygenation. Although temperature is the main factor affecting the carbonaceous deoxygenation rate under laboratory conditions, a number of factors affect the nitrogenous rate. Among those affecting the nitrogenous rate were pH, temperature, mixing, suspended particle concentrations, dissolved oxygen, and initial nitrifier concentration.

Results from the standard BOD tests have commonly been referred to as "laboratory" tests. First-order reactions for both carbonaceous and nitrogenous deoxygenation have been proposed for describing the observed laboratory reaction (Streeter, 1935a; 1935b). Zero and second order reactions have been proposed for use, but first-order reactions are usually assumed for modeling simplicity (IDEQ, 1975a).

Bansal (1975) pointed out that differences often exist between "laboratory" reaction rates and reaction rates observed in the river. He attributed those differences to factors which were not present in the laboratory. Studies of "river" reaction rates have been performed by Thomas (1948), Streeter (1958), and McKee and Wolf (1963). Typically, "river" reaction rates have been determined using the following relationship:

$$K_r = 1/t \log (L_u/L_d)$$

where

K_r = River deoxygenation rate constant (either carbonaceous and/or nitrogenous), base 10 (1/day),

T = Time of travel between sampling points (days),

L_u = Constituent value at the upstream location (typically, ultimate carbonaceous or nitrogenous BOD data in mg/1), and

L_d = Constituent value at the downstream location (mg/1).

McKee and Wolf (1963) distinguished the differences between "laboratory" (K_1) and "river" (K_r) reaction rates as

$$K_3 = K_r - K_1$$

where

K_3 is the reaction rate of the differences, base 10 (1/day).

McKee and Wolf (1963) noted that the factors making K_r different from K_1 fell into two groups, making K_3 either positive or negative. Factors making K_3 positive included sedimentation, volatilization, flocculation, adsorption, and biological activities. Factors making K_3 negative included contributions from sludge deposits, channel scour, longitudinal mixing, and short-circuiting.

McKee and Wolf (1963) also noted that seasonal variations in K_3 could be expected. Bosko (1966) developed an equation relating K_r and K_1, incorporating stream depth and velocity as shown below:

$$K_r = K_1 + n(v/d)$$

where

K_r and K_1 are as previously defined,

v = stream velocity (fps),

d = stream depth (feet), and

n = coefficient of bed activity.

The value of "n" was related to stream slope with the following values shown in Table 4.

Table 4. Values of bed activities versus stream slopes[a]

n	Stream Slope (ft/mi)
0.10	2.5
0.15	5.0
0.25	10.0
0.40	25.0
0.60	50.0

[a]Source: Zison *et al.* (1978).

Biostimulation and algal uptake

Biostimulation of stream reaches below wastewater treatment plants has been observed by many researchers, including O'Connell and Thomas (1965) and Dougal (1969). Proof of the stimulation is typically in the physical sighting of excessive plant growth or the observance of *widely varying* diurnal dissolved oxygen patterns downstream from the wastewater treatment plant effluent. The above researchers attributed the stimulation to be a result of nutrient addition or biostimulation. Burkholder-Crecco and Bachmann (1979) provided evidence to suggest that suspended algal populations in Central Iowa streams may also be light limited, since chlorophyll-a concentrations increased in river samples incubated at higher light levels.

Closely associated with biostimulation is the concept of nutrient limitation. Gibson (1971) noted that a great deal of confusion existed over the term "nutrient limitation," and that this was probably due to the definition of "nutrient limitation" itself. Gibson (1971) suggested that, "a factor is not limiting if, when it is increased, no effect on growth is observed."

Coinciding with biostimulation of aquatic plant life, many researchers have found that uptake of nutrients, such as phosphorous and ammonia by algae, can have a dramatic effect on water quality parameters (Zison et al., 1978). As a result, many researchers recognized that if NH_3 was taken up by algae, it would not be able to enter the nitrification steps, thereby causing less of an oxygen demand on receiving streams (Dougal, 1969 and JRB Associates (JRB), 1983a).

Predictions of the amount of NH_3 uptake have been performed by others, including Dougal (1969) and Shindala et al. (as cited by JRB, 1983a). Dougal estimated that less than 50% of the ammonia was nitrified from a mass balance approach indicating that the remainder may be used directly by algae. JRB Associates presented an equation to predict the amount of NH_3 uptake by algae, which was obtained from the work of Shindala et al. (as cited by JRB, 1983a).

Crumpton (Department of Botany, Iowa State University, personal communication, 1984) expressed doubt about the ability of algae to maintain the uptake of NH_3 for any extended period and questioned the validity of trying to model such an event. Crumpton suggested that algae would help even out NH_3 concentration peaks and valleys in the river, but over an extended period NH_3 inflow would equal NH_3 outflow.

Ammonia toxicity

Molecular ammonia (NH_3) in addition to being a nutrient and an oxygen demanding material, has been found to be acutely toxic to fish and aquatic life, according to McKee and Wolf (1963). The toxicity problem occurs as increasing ammonia (NH_3) concentrations inhibit the ability of fish hemoglobin to combine with oxygen.

Ammonia exists in an equilibrium state in water as shown in a simplistic manner below.

$$NH_3 + H^+ \rightleftharpoons NH_4^+, \quad (K = 5.0 \times 10^{-10} \text{ at } 20^oC)$$

The molecular form of ammonia has been reported to be lethal in the range of 0.2 to over 2.0 mg/l (Alabaster and Lloyd, 1980). As indicated by the equilibrium equation, even a slight increase in pH may cause a great increase in the concentration of molecular ammonia, and hence, its toxicity. Other factors which have been shown to increase ammonia toxicity at a given pH include greater dissolved oxygen and carbon dioxide concentrations, higher temperatures, and bicarbonate alkalinity as summarized by the Environmental Protection Agency (EPA, 1976).

Current criterion limiting unionized ammonia concentrations in streams has been set at 0.02 mg/l (EPA, 1976). Table 5 shows the concentrations of total ammonia (NH_3 plus NH_4^+) which contain 0.02 mg/l of unionized ammonia for various temperature and pH ranges.

JRB Associates (JRB, 1983c) suggested that a draft equation developed by the EPA may allow greater concentrations of ammonia to be allowed in Iowa rivers than are at present. Summer and winter time

Table 5. Concentrations of total ammonia ($NH_3 + NH_4^+$) which contain an unionized ammonia concentration of 0.020 mg/l (NH_3)[a]

Temperature (^oC)	pH Value								
	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0
5	160.	51.	16.	5.1	1.6	0.53	0.18	0.071	0.036
10	110.	34.	11.	3.4	1.1	0.36	0.13	0.054	0.031
15	73.	23.	7.3	2.3	0.75	0.25	0.093	0.043	0.027
20	50.	16.	5.1	1.6	0.52	0.18	0.070	0.036	0.025
25	35.	11.	3.5	1.1	0.37	0.13	0.055	0.031	0.024
30	25.	7.9	2.5	0.81	0.27	0.099	0.045	0.028	0.022

[a]Source: EPA (1976).

limitations of 3.91 mg/l and 12.8 mg/l respectively, were suggested for implementation at a pH of 7.5 (JRB, 1983c).

Historical Water Quality Modeling

Streeter and Phelps (1925) are generally recognized as the first to model a stream's oxygen resources by combining the two opposing reactions of carbonaceous organic waste deoxygenation and atmospheric reaeration. Integration of the combined equation resulted in the following:

$$D = \frac{K_1 L_o}{K_2 - K_1} \left(e^{-K_1 t} - e^{-K_2 t}\right) + D_o \, e^{-K_2 t}$$

where,

D = dissolved oxygen deficit below saturation, mg/l,

D_o = initial dissolved oxygen saturation deficit at the initial point of reference (t=0), mg/l,

L_o = initial ultimate carbonaceous oxygen demand, mg/l,

K_1 = carbonaceous rate constant, per day (base e),

K_2 = reaeration rate constant, per day (base e).

Typical results for the Streeter-Phelps equation produce an "oxygen sag curve," as shown in Figure 3. However, Dougal (1969) found that an "oxygen bulge curve" actually existed downstream from the Ames treatment plant (during the daylight) in response to stimulated photosynthetic activity.

Numerous changes to the original Streeter-Phelps equation developed during the years following 1925. The first major change allowed for easier calculations of maximum initial loadings to avoid anaerobic or

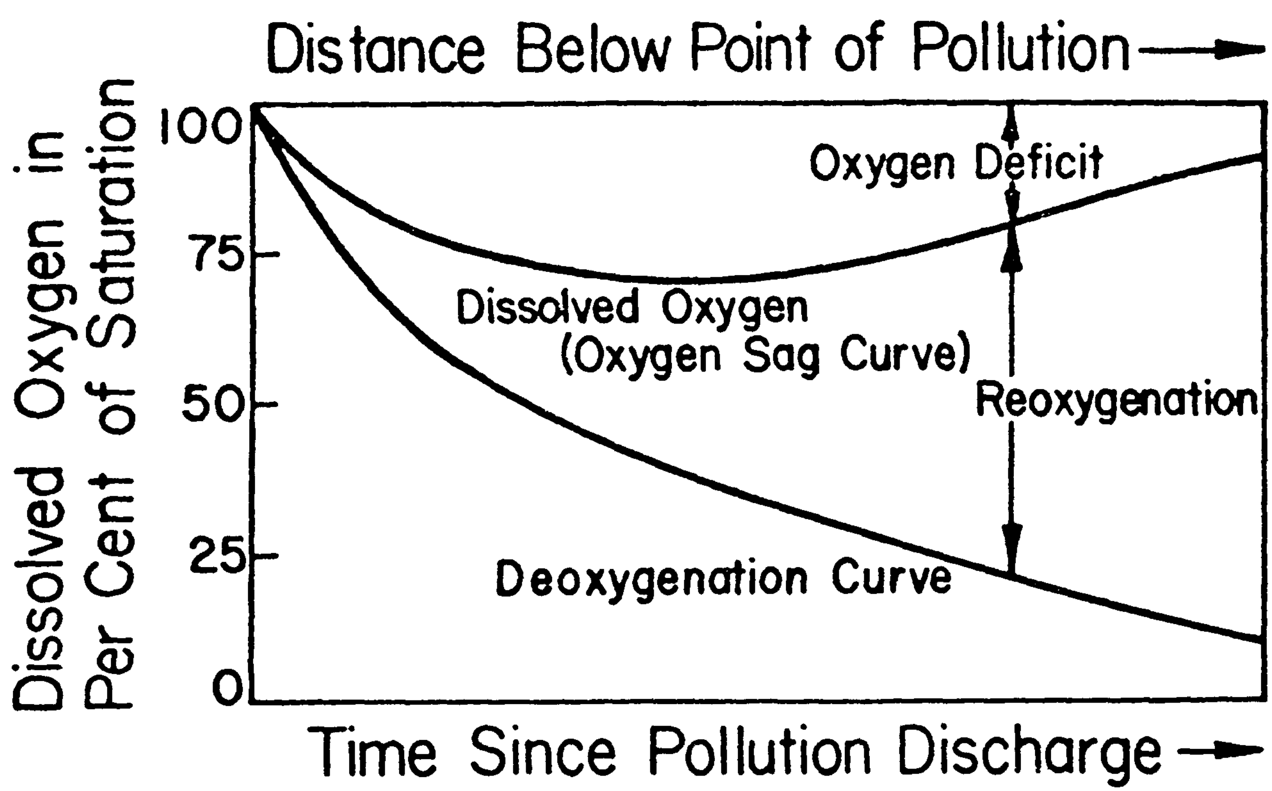

Figure 3. Characteristics of the oxygen sag curve (Dougal, 1969)

septic conditions. This change resulted in the introduction of the term labeled as the coefficient of "self-purification" for streams (Fair and Geyer, 1954). This coefficient was simply the ratio of the reaeration rate divided by the carbonaceous deoxygenation rate or,

$$f = K_2/K_1 \quad .$$

The coefficient "f" was used in ascertaining maximum loadings and critical time periods to the point of minimum dissolved oxygen levels.

Other developments to the original Streeter-Phelps equation involved the effect that distributed load contributions of pollution or dilution had on stream conditions (Streeter and Phelps, 1925).

Closely related to distributed loadings is the effect of sludge loading contributions as developed by Streeter (1935a) and Velz (1970). However, Dougal (1969) pointed out that unlike distributed loadings, sludge loadings represent a continuous, steady-state demand to be exerted in terms of mg/l of oxygen per unit time.

Dougal (1969) summarized other major developments including the additional effects of algae, nitrogenous matter, and the introduction of "river" deoxygenation rate constants. These developments have been previously discussed.

Dougal (1969) found that <u>no one</u> model, to that date, had combined all the possible <u>major</u> interactions into one equation. Consequently, Dougal (1969) developed an equation which included the effects of initial D.O. deficits, carbonaceous oxygen demands, nitrogenous oxygen demands, distributed loading contributions, <u>uniform</u> sludge loading demands, atmospheric reaeration, and <u>net</u> photosynthesis. For brevity, the equation is not presented here, as many of the components will be described in more detail later in this thesis.

Local Water Quality Modeling and Studies of Interest

Important water quality studies in the State of Iowa have been performed on and near the Skunk river and are highlighted below.

By far, the most complete water quality study was accomplished by Dougal in 1969 (Dougal, 1969). Dougal's study examined in detail the technical, economic, and institutional factors associated with the establishment of stream water quality standards. Dougal used the Skunk River near Ames as a case study in the paper, which led to the development of a mathematical computer model for use in simulating, verifying, and forecasting stream water quality. Interesting features of the dissertation included a dye tracer study and a comprehensive sampling program. The sampling program involved extensive periods of low flow sampling, including a time when only settled raw sewage (primary effluent) was discharged to the river.

Another local study of interest involved the work of Speiran (1977) on the Des Moines River. Speiran looked at the impacts of algae and point source pollution effects on water quality in that river.

Historically, water quality modeling has occurred on large rivers, such that low flow stream situations have largely been ignored. However, a study by Shelton et al. (1978) pointed out that as treatment facilities are upgraded on these smaller low flow streams, water quality modeling must incorporate the wider ranges of environmental factors that are often disregarded in the larger studies. Shelton et al. (1978) calibrated and applied a mathematical modeling approach taking into account changes in oxygen deficit, due to carbonaceous and nitrogenous deoxygenation, stream reaeration, benthic (or sludge) loadings, net photosynthesis, and locally produced toxic metal effects which reduced the deoxygenation rate constants.

CHARACTERISTICS OF THE SKUNK RIVER BASIN FOR THE STUDY AREA

General

The entire Skunk River basin lies within the boundaries of the State of Iowa as shown in Figure 4. The Skunk River basin is essentially rectangularly shaped having an overall length of 180 miles and an average width of 24 miles (IDEQ, 1976). The basic flow pattern is towards the Southeast, beginning in the central portion of the State and flowing to the southeast corner of Iowa.

ADD Precipitation for the basin ranges from about 29" in Hamilton County to about 34" at its mouth (IDEQ, 1975b). Temperature ranges are quite wide throughout the basin, but range from a mean maximum July temperature of near 90°F to a mean minimum of only 9°F in winter (IDEQ, 1975b).

Geological

Glaciers and surface water erosion have largely established the present day "physiographic conformation of the Skunk River basin" (IDEQ, 1976). Additionally, the effects of faulting on the location and flow direction of the upper portions of the Skunk River have been shown by Willie (1984).

The four major glaciers affecting the Skunk River basin were the Nebraskan, Kansan, Illinoian, and the Wisconsin. The Wisconsin glacier covered only the upper portion of the Skunk River basin and has only

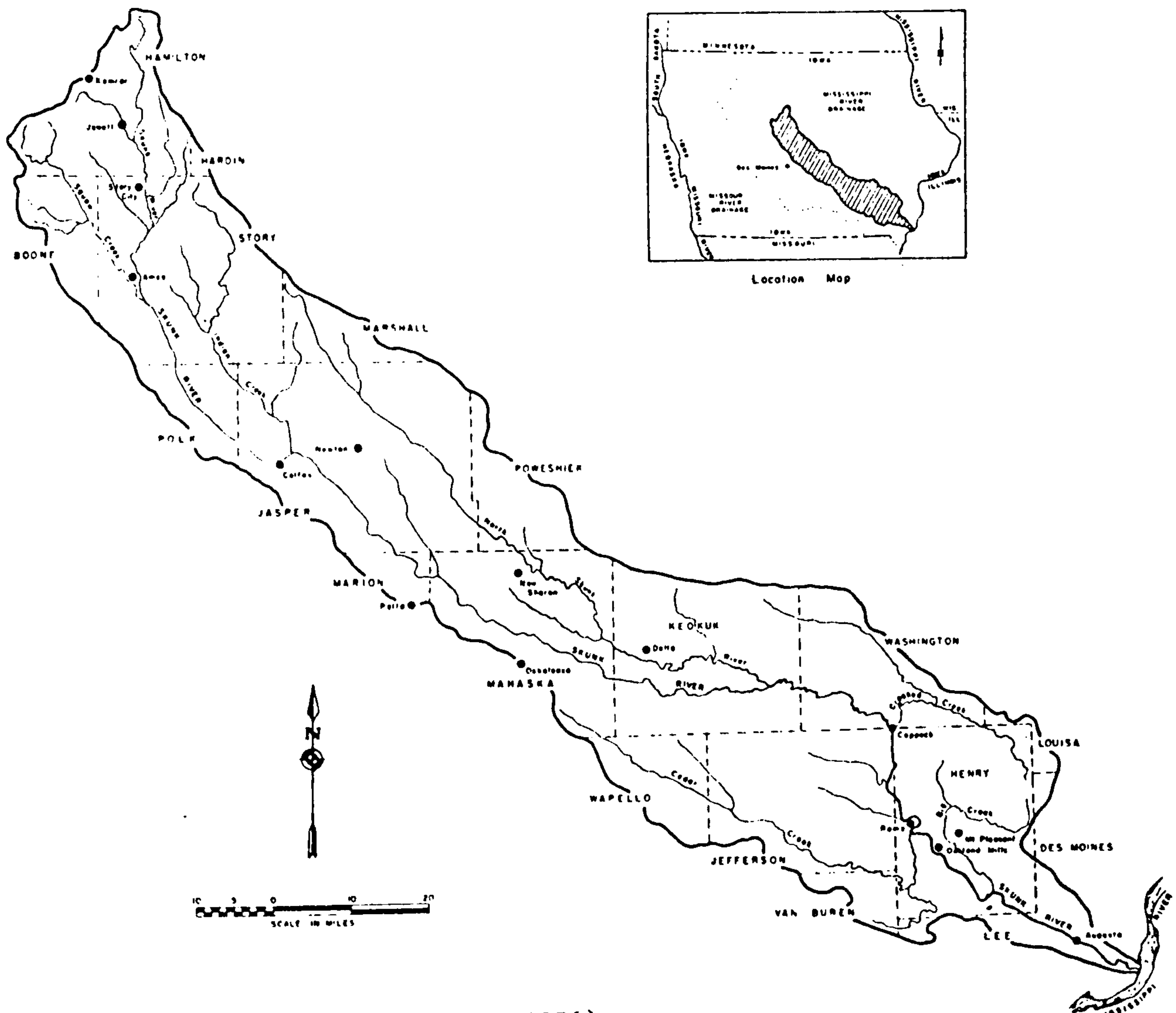

Figure 4. Entire Skunk River basin (IDEQ, 1976)

recently (geologically) retreated. As a result of the recent retreat, the upper portion of the basin is characterized as having "youthful" or poorly drained land areas (IDEQ, 1976). The existence of marshes or swamps, indicative of the Ames area prior to drainage, is consistent with this youthful topography.

Several geologic investigations have been conducted in the Skunk River basin, especially near Ames. Willie (1984) recently summarized the major studies near the Ames area in a geologic investigation south and east of Ames.

The Skunk River drainage and flow characteristics are strongly influenced by buried preglacial channels which come in direct contact with both the Squaw Creek and the Skunk River at Ames. These buried channels unite just south of Ames and continue southward along the present day Skunk River channel.

Many physical features of the Skunk River basin can be explained geologically, but will be included in the next section for simplicity.

Physical

Physical characteristics of interest in the Skunk River basin study area include river slopes, lengths, widths, depths, and substrate material.

Beginning at its origin in Hamilton County and continuing in part of Story County, the Skunk River meanders in a relatively narrow valley with depths that are relatively shallow to moderate. Bottom substrates consist of rock and mud. This is the steepest portion of the Skunk River and slopes average 7.8 feet per mile from Kamrar to Story City, falling to 5.0 feet per mile from Story City to Ames (IDEQ, 1976).

Outcroppings of sandstone, shale, and limestone restrict the width of the river north of Ames, but the river widens rapidly into a broad flood plain immediately above Ames (Larimer, 1957), as the river enters the previously mentioned preglacial channel. The widened preglacial channel substrate primarily consists of shifting sands (Jones, 1972). Shifting sands may also be found for the substrate material for the remainder of the study area, from Ames to Colfax.

The major physical feature of the Skunk River from Ames to Colfax is a direct result of channel straightening by dredging, which occurred during the years 1893 to 1923. Some meandering has been reported in the straightened portion by Wells (1956), however, the effects of the dredging have been fairly permanent.

Average slopes from Ames to Cambridge are near 3.5 feet per mile and taper off to about 2.6 feet per mile from Cambridge to Colfax (Larimer, 1957).

Widths and depths can generally be characterized as wide and shallow, respectively, but directly depend on discharge. River widths greater than 100 feet are typical in widened channel, with depths often less than one-half foot for extended river widths.

River mileage between sampling points of interest were examined below Ames in preparation of contemplated sampling trips. This mileage was scaled off the 7 1/2 minute USGS quadrangle maps and was found to differ from Dougal's (1969) values which were obtained from aerial photography. A comparison of river mileage for 20 points of interest may be found in Table 6.

Table 6. River mileage comparison from Ames to Colfax at 20 selected sites

Site Description	Dougal's mileage	Adjusted mileage
South Sixteenth Street Bridge (SK1). First bridge north of U.S. Route 30	.00	.00
Centerline U.S. Route 30 Bridges	0.19	0.18
Ames WPCP effluent discharge point	0.37	0.37
First bridge south of the Ames WPCP, on an unimproved road, at the end of Ken Maril Road. (SK2)	1.80	2.01
First bridge upstream of I-35, designated as BR876 on Huxley Quadrangle map (HQM). (SK3)	2.93	3.25
First bridge downstream of I-35. No designation on HQM. (SK4)	5.34	5.58
"Askew" bridge, designated as BR 865 on HQM.	6.49	6.74
Bridge northeast of Cambridge on State Route 211.	8.94	9.18
Bridge southeast of Cambridgee designated as BR 853 on HQM.	9.82	10.05
Bridge on Iowa Route 210, Southeast of Cambridge.	10.97	11.23
Bridge on NE 158th Avenue, designated as BR 842 on Elkhart quadrangle map.	12.97	13.21
Bridge on NE 150th Avenue, designated as BR 837 on Loring quadrangle map (LQM).	14.16	14.38
Bridge on Yoder Drive, connecting NE 126th Avenue with NE 134th Avenue. No designation on LQM	17.57	17.69

Table 6. Continued

Bridge on NE 118th Avenue. No designation on LQM.	19.58	19.61
Bridge on U.S. Route 65. Designated as BR 819 on Altoona quadrangle map.	22.81	22.77
Bridge on NE 112th Street, designated as BR 808 on Mitchellville quadrangle map (MQM).	24.73	24.62
Bridge on local road just upstream of I-80 bridges. No designation of MQM.	28.95	29.04
Centerline of I-80 bridges	29.20	29.43
Bridge just north of Colfax on State Route 117.	31.87	31.89
Bridge east of Colfax on State Route 90.	34.56	35.55

Hydrological

The hydrological basin characteristics of a river are largely portrayed by statistical analysis of high and low flows. Average flows and other flow frequency ratios may also be used for comparisons. Flow data for the above analysis were obtained through a network of flow gaging stations along the stream or river.

Three gaging stations near Ames have historically been used to describe the hydrological basin characteristics for the portion of the Skunk River in the study area. These gaging stations were presented earlier in Figure 1, and are described in more detail in Appendix B. Two of the gaging stations (Iowa Geologic Survey "IGS" identification numbers 05-4700.00 and 05-4705.00) are still being used today, but the third station (IGS #05-4710.00) was discontinued in 1979.

Low and high flow analyses for all three of the gaging stations near Ames have been performed. The discontinued gaging station data, however, are of greatest interest to the modeling effort, since the station is located immediately upstream of the Ames WPCP effluent discharge. Only the low and high flow analyses for that station will be presented here.

Low flow frequency data for the discontinued gaging station south of Ames at the South Sixteenth Street bridge are shown in Table 7. In Iowa, the 7 day average low flow condition, which occurs once every 10 years (7Q10), is used in the WLA procedure. For the flow south of Ames, this is 0 cfs (cubic feet per second). Dougal (1969) identified the Skunk River basin as having poor low flow characteristics, indicating that minimal sustaining groundwater contributions occur during dry weather.

High flow or flood frequency distributions for the discontinued

Table 7. Magnitude and frequency of annual low flow for the South Sixteenth Street gaging station (IGS # 05-4710.00)[a]

Recurrence Interval (Years)	Lowest Average Flow, in Ft^3/Sec, for Indicated Period in Consecutive Days 3	7	14	30	60	120	183
1.5	7.7	9.0	9.2	15.0	28	59	80
2	2.7	2.6	2.7	4.3	10.0	22	33
5	0	0	0	0.02	0.75	2.4	4.6
10	0	0	0	0	0.11	0.61	1.5
20	0	0	0	0	0	0.18	0.52

[a] Source: (Lara, 1979).

gaging station are shown in Table 8. This information was obtained using a computer program from the U.S. Army Corps of Engineers entitled, "Flood Flow Frequency Analysis, Water Resources Version." The version date listed on the computer printout was January, 1980. A generalized skew of -0.4 and an adopted skew of -0.5 were used in the computations.

The problem of obtaining a discharge at this gaging station and four alternative methods of solving this problem will be presented in more detail in a separate section of this thesis entitled, "Evaluation of the Discharge Measurement South of Ames."

Biological

Several limnological surveys have recently been conducted on the Skunk River basin. Many of these specifically addressed the Skunk River near Ames and included work by Coon (1971), Kilkus (1972), and Jones (1972). A subcommittee report entitled "Water Use Plan for Ames," (Water Use Subcommittee, 1982) addressed several of these limnological surveys and discussed them in regards to the planned wastewater treatment facility near Ames. To gain an appreciation of the salient points of these studies, a brief review of each one will be presented.

In 1970, Coon (1971) conducted a rigorous fish sampling program on the Skunk River from Story City to Ames. Over 8,000 fish were collected during the study. Diversity of fish species decreased dramatically as bottom substrates changed near Ames. Subsequent analysis by Jones *et al.* (1974) suggested that the substrate change was probably the more important parameter causing the decreased fish diversity, although they did not rule out the effluent from the Ames WPCP. Carp comprised the

Table 8. Magnitude and frequency of the computed and expected probability flows for the South Sixteenth Street gaging station (IGS # 05-4710.00)

Recurrence Interval (Years)	Peak Flow, in Ft^3/Sec Computed Flow	Expected Probability
1.25	4120	4060
2	5990	5990
5	8290	8380
10	9640	9820
25	11,200	11,500
50	12,200	12,700
100	13,100	13,900
500	15,100	16,300

greatest percentage of fish species caught ranging from 69 to 84% of the total.

Kilkus (1972) examined the effect of nutrient concentrations on several Iowa streams including the Skunk River. Results from this study suggested that high concentrations of nitrogen and phosphorous existed in Iowa's streams and primarily occurred from sources other than municipal sewage plants. Kilkus (1972) also suggested that algal limitation in Iowa's rivers was probably controlled by factors other than nitrogen or phosphorus.

Jones (1972) examined water quality above and below Ames on the Skunk River. Conclusions drawn indicated that the water quality was significantly affected downstream from the Ames WPCP.

A subcommittee on water use in Ames (Water Use Subcommittee, 1982), however, reached a conclusion based on the studies mentioned above indicating that water quality degradation below the Ames WPCP has had little (if any) noticeable effect on fish populations in the Skunk. As a result, the committee's final conclusion was that substantial water quality improvement below Ames would not appreciably increase fish diversity in that area.

Historical Water Quality

Water quality sampling performed on a regular basis can provide a great wealth of knowledge concerning seasonal, yearly, or flow related water quality patterns. This type of information is typically not collected, as it requires a continuous sampling program with subsequent funding.

Fortunately, the Ames WPCP has obtained and analyzed samples from the Skunk River for at least two locations since the early sixties. The ongoing sampling program typically consists of collecting grab samples at each location (one above the effluent discharge point and one or more below the discharge) on a weekly basis and analyzing them for nine important water quality parameters. The nine parameters have included temperature (Temp), dissolved oxygen (D.O.), biochemical oxygen demand (BOD), total suspended solids (TSS), volatile suspended solids (VSS), ammonia (NH_3 as nitrogen), nitrate (NO_3 as nitrogen), phosphorous (PO_4 as phosphorus), and pH.

Sampling sites used in the program include the bridge immediately below the confluence with Squaw Creek, as the upstream site, and either the first or second bridge below the Ames WPCP effluent discharge location. Occasionally, a third site is used in the sampling program, as an additional downstream site, and is located slightly over 5 miles below the Ames WPCP discharge. These four locations correspond to the sampling sites listed in the river mileage comparison as SK1, SK2, SK3, and SK4 as presented in Table 6 and shown in Figure 1.

The sampling program has been subject to changes since 1960 and some months lack data collection entirely. However, the consistency of sampling has improved recently such that only <u>two</u> months have not included at least one sampling event since 1977, with those occurring in 1978.

Sampling procedures typically involve obtaining grab samples from the bridge itself, unless low flow conditions permit (or require) wading into the river. D.O. samples are obtained by filling the BOD bottle directly from the grab sample bucket, while attempting to minimize aeration upon filling. All chemical analysis are performed back at the Ames WPCP laboratory immediately upon completing the sampling trip. No preservation steps are taken due to the small amount of time (0.5 to 1.0 hour) required to complete the trip. Samples are normally collected in the early part of the day, beginning around 8:00 a.m.

Chemical procedures followed for analysis are those as defined in Standard Methods (APHA, 1981) and are performed by the WPCP personnel.

A complete and thorough analysis of all of the data available was not viewed as a productive exercise for this thesis topic. However, an analysis of the "clean stream" water quality parameters (approximated at the bridge immediately below the confluence with Squaw Creek) for various discharge rates was considered important for the modeling procedure.

Figures 5, 6, 7 and 8 represent BOD, NH_3, PO_4 and NO_3 fluctuations compared with discharge rates for January 1978 through November 1983, respectively. Values shown are monthly average concentrations obtained for the number of samples collected during that calendar month. Missing data are appropriately shown on the figures and were placed in accordance with the assumed trend lines.

Sources of Pollution

Pollution sources can be classified as "point" or "nonpoint" sources, depending on whether or not a specific entry location of the pollution can readily be established. Typical "point" sources include municipal, industrial, and public - semi-public discharges, which are characterized by effluent discharge pipes at one location. "Nonpoint"

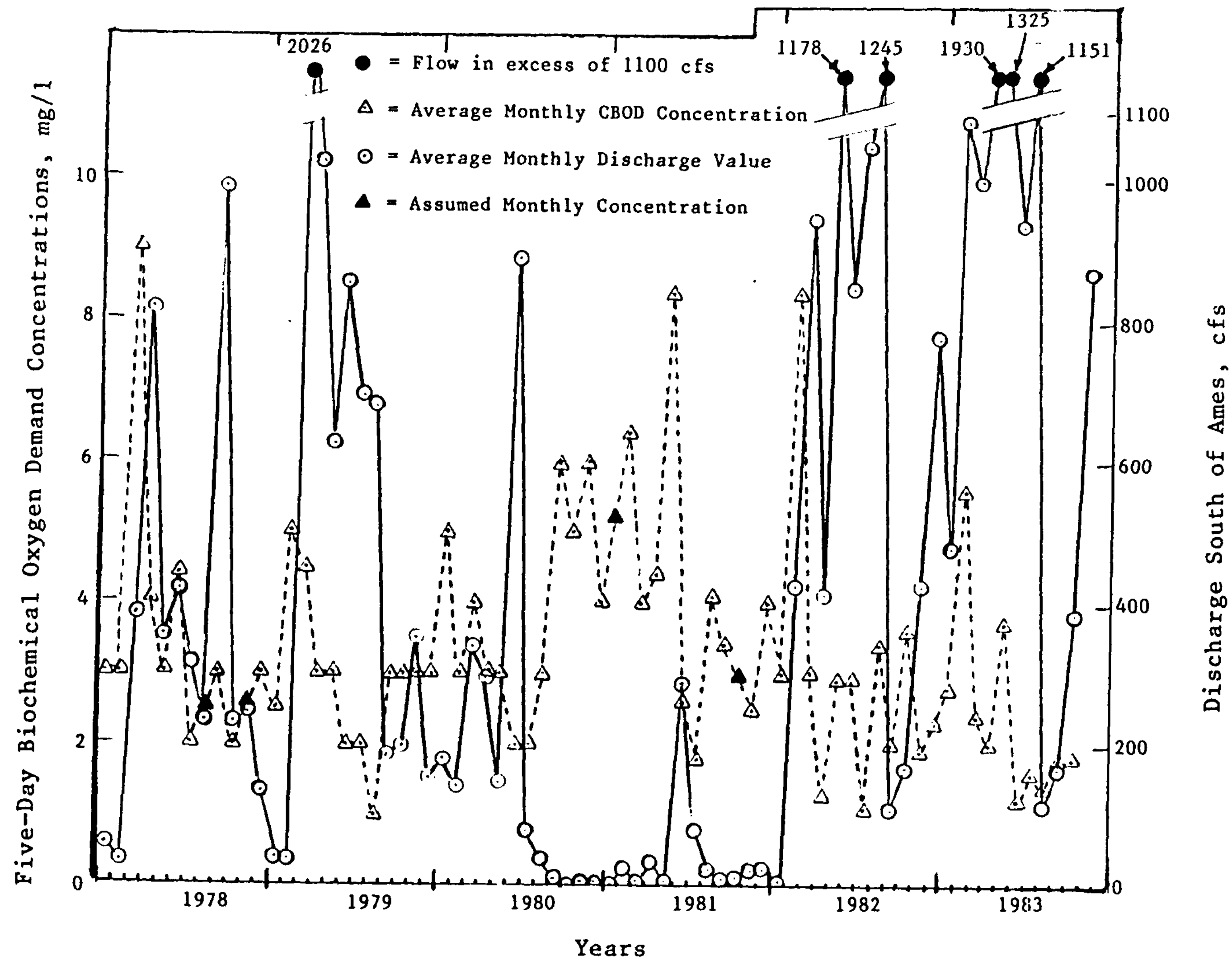

Figure 5. Monthly five-day BOD concentrations above the Ames WPCP

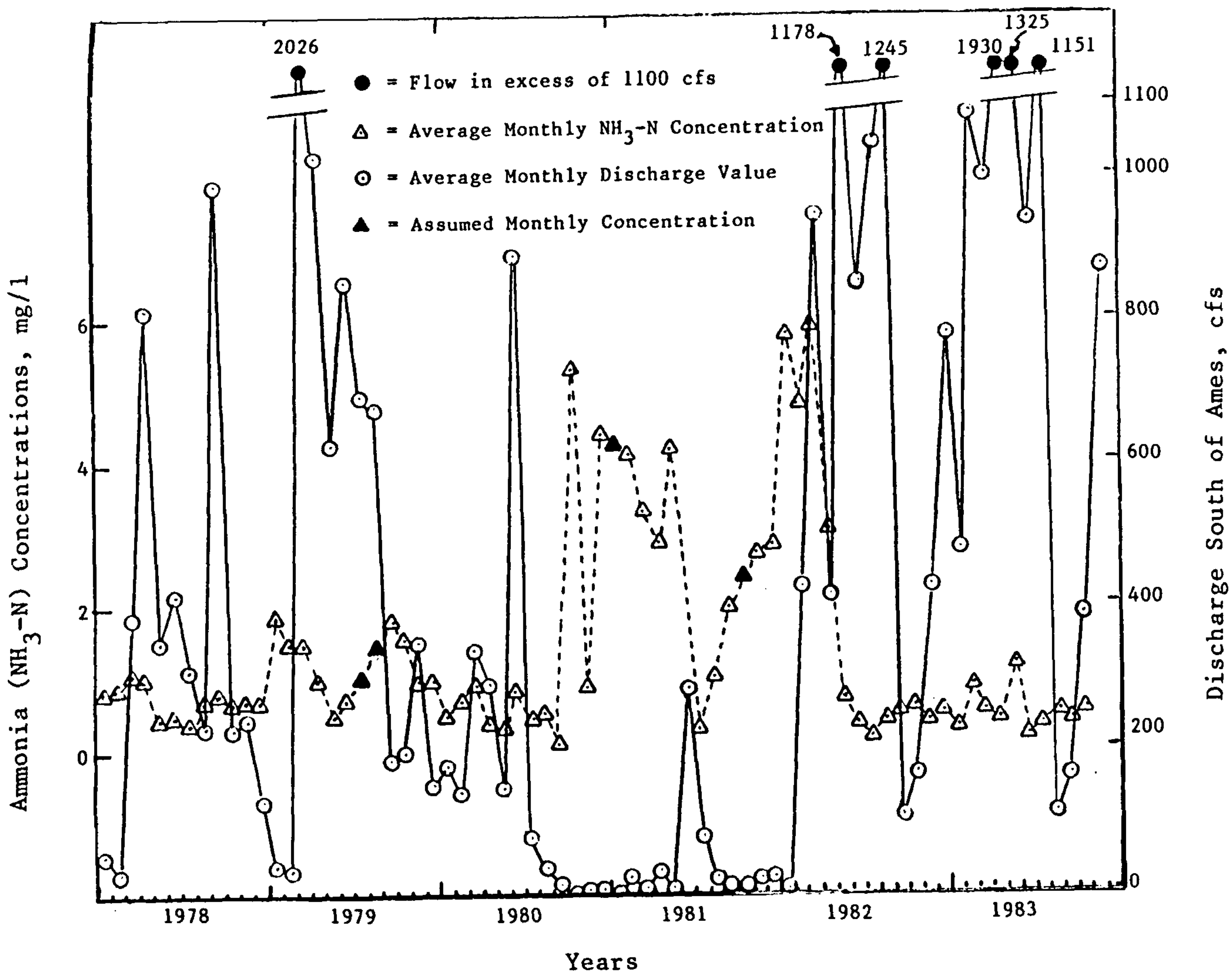

Figure 6. Monthly ammonia concentrations above the Ames WPCP

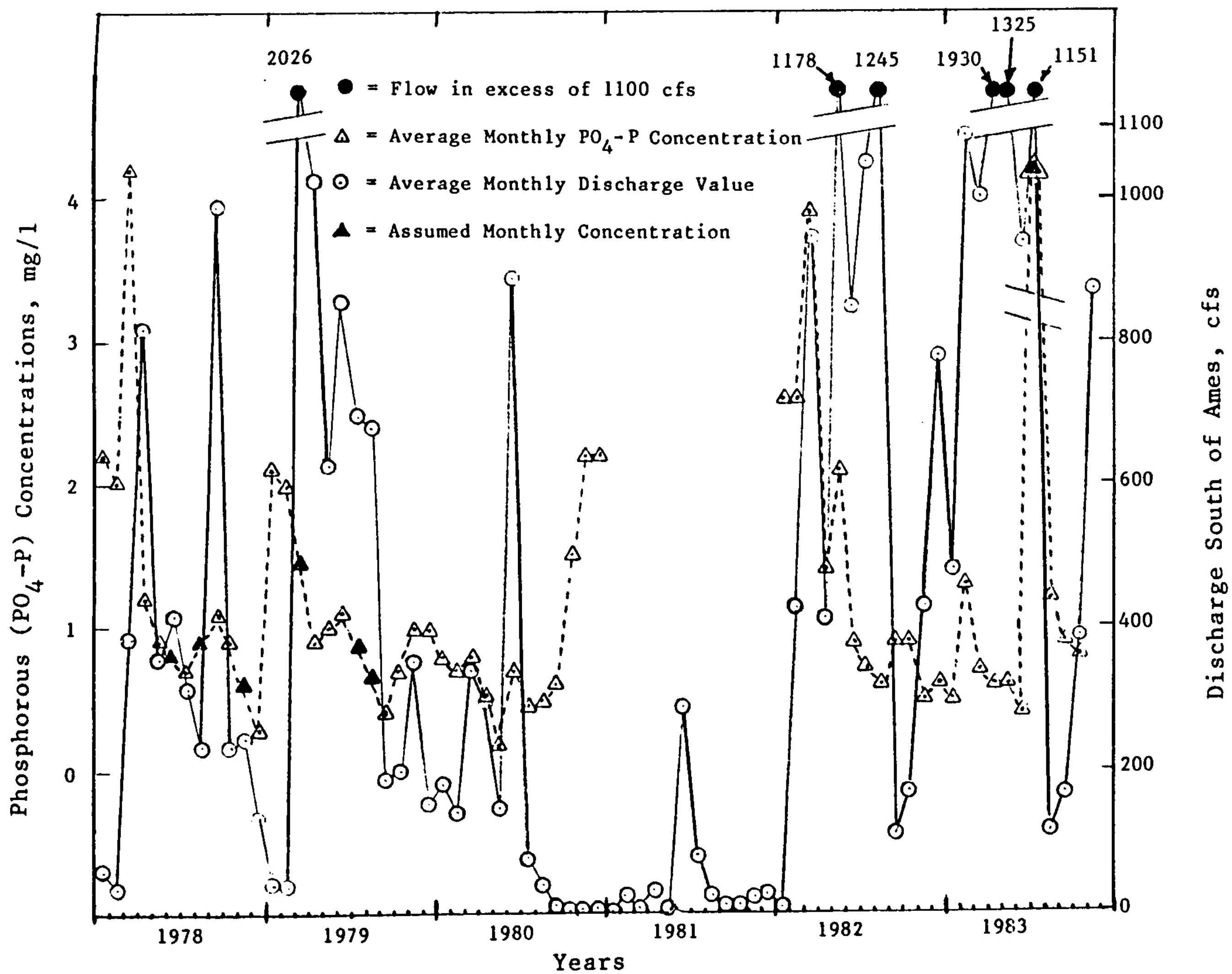

Figure 7. Monthly phosphorous concentrations above the Ames WPCP

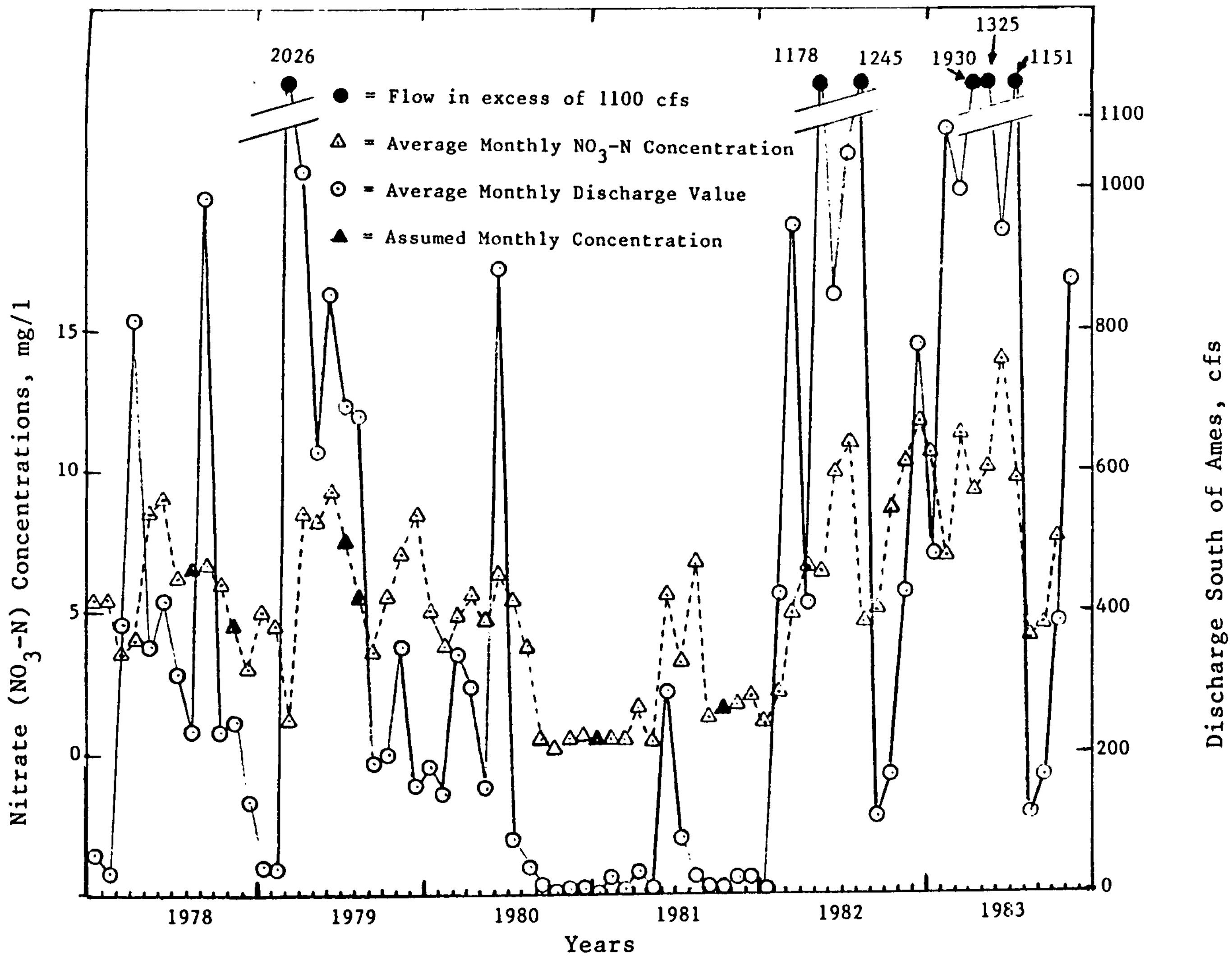

Figure 8. Monthly nitrate concentrations above the Ames WPCP

sources primarily originate from agricultural activities, but may include urban sources, as well. Typical "nonpoint" sources include feedlot runoff, fertilizer and pesticide application, sediment contributions from erosion, and other farmsteading operations.

Dougal (1969) investigated pollution effects of both "point" and "nonpoint" sources and concluded that although "nonpoint" sources contribute immensely to water quality pollution, their effect is minimal during low flow conditions. Thus, only the municipal and other point source dischargers were considered of major importance in modeling during low flow situations.

The State of Iowa (IDEQ, 1976) has summarized point source dischargers for the entire Skunk River basin. The 1976 state report (IDEQ, 1976) showed that 8 municipal, 6 industrial, and 17 public - semi-public dischargers exist upstream of the Ames WPCP. Only the eight municipal dischargers are of interest in modeling, as the other discharges are small or consist primarily of cooling water discharges or other large volume, low pollutant discharge. Seven of the 8 dischargers are currently using lagoon systems and so these too are of little significance in modeling the Skunk. The only discharge of interest is that of Story City which treats wastes using an Imhoff tank and trickling filter built in 1963 (IDEQ, 1976). (The City of Gilbert was given a 2000 year peak wet weather flow of 0.2 mgd for establishing the Ames WLA in 1982, as indicated in Appendix C. The basis for this flow allocation is presently unknown since the City of Gilbert uses a logoon system.)

One main "point" source of pollution exists below Ames, the Ames Water Pollution Control Plant. The plant consists of complete secondary treatment, using trickling filters for biological treatment. The plant was built in 1949-1950 and reached design population capacity in 1955, due to the tremendous growth at Ames and I.S.U. according to Dougal (1969).

As a result of decreasing pollution removal efficiencies since 1955, the treatment facility has been a major contributor of water quality degradaton to the stream. Average effluent from the plant consists of 5-25 mg/l of NH_3-N, 15-25 mg/l of BOD, 12-18 mg/l of PO_4, 4-9 mg/l of NO_3-N, and 15-20 mg/l of TSS with an average flow of 5-7 mgd (million gallons per day), based on last half of 1983 data.

EVALUATION OF THE DISCHARGE SOUTH OF AMES

Introduction

As indicated earlier, the discharge measurement is an important physical parameter used in water quality modeling. Of particular importance in modeling the Skunk River below Ames is the discharge measurement made at the now discontinued gaging station located below the confluence with Squaw Creek. (Iowa Geological Station, "IGS", identification number 05-4710.00) This gaging station may be found in Figure 1 and is located immediately south of the confluence with Squaw Creek near the South Sixteenth Street bridge at Ames. A more detailed description of the location for this gaging station can be found in Appendix B, as well as other pertinent discharge information. This station will be referred to as the "South Sixteenth Street" gaging station for the remainder of this thesis.

The South Sixteenth Street gaging station was discontinued on September 30, 1979, due primarily to budgetary reductions at IGS. The poor condition of the concrete overflow control weir, in addition to the close proximity of the two upstream gaging stations, strongly influenced the choice of discontinuing this station. Nonetheless, an easy and reliable method of obtaining a discharge measurement at this location was investigated for a number of reasons.

First of all, the gaging station is located only 0.37 mile upstream from the Ames Water Pollution Control Plant (Ames WPCP) effluent discharge pipe. Thus, it is possible to obtain water quantity and quality measurements in the stream reach immediately above and below the effluent discharge pipe, with only a minimum of additional sampling required. This can be accomplished through the use of an equation, presented by Babbitt and Baumann (1958), as shown below:

$$Cm = \frac{Ce\ Qe + Cr\ Qr}{Qe + Qr}$$

where,

Cm = the amount or concentration of the substance in the combined mixture,

Ce = the concentration of the substance in the effluent,

Cr = the concentration of the substance in the receiving water initially,

Qe = the quantity or rate of flow of the effluent, and

Qr = the quantity or rate of flow of the receiving water initially.

Thus, the left-hand side of the equation can be found by ascertaining the components on the right-hand side. These algebraic computations represent a reliable and practical alternative to the difficult task of sampling below the effluent discharge pipe. This is because complete and ideal mixing in short distances below the outfall would be rare.

This short reach also makes it possible to assume that water quantity and quality measurements made at the gaging station are identical to those that would be obtained at the effluent discharge pipe, since the time of travel between the two points is short and the amount of additional drainage into the river is relatively insignificant.

Secondly, Dougal (1969) used the discharge measurement from the now discontinued gaging station to assess the river's average velocity from his extensive dye tracer studies. The determination of velocity is another important physical parameter which directly or indirectly affects every component used in water quality modeling. A relationship obtained by Dougal (1969), portraying average velocity and discharge, is shown in Figure 9. Tracer studies probably represent the most accurate way to obtain these velocity measurements since independent determinations of volume and discharge or current-meter readings of velocity are not required. Hence, the benefits of using measurements at this location should be emphasized.

Four Alternative Methods of Obtaining the Discharge

To continue using Dougal's velocity-discharge relationship, four methods of obtaining the reference discharge were investigated.

The first two methods investigated were arrived at by considering how the discharge measurement was taken when the gaging station was operational. Usually, the discharge measurement was obtained by a water-stage recorder which continuously monitored fluctuations in gage height. In addition, a wire weight gage placed on the south side of the South Sixteenth Street bridge could be used to physically obtain the gage height. Upon discontinuation, the station house instrumentation was removed, but the wire weight gage was left on the bridge. These facts suggested two obvious methods for determining the discharge measurement at the discontinued gaging station and are discussed below.

The <u>first</u> method involved the use of the existing wire weight gage to obtain a gage height, which could be used to obtain a discharge, given an up-to-date stage-discharge curve.

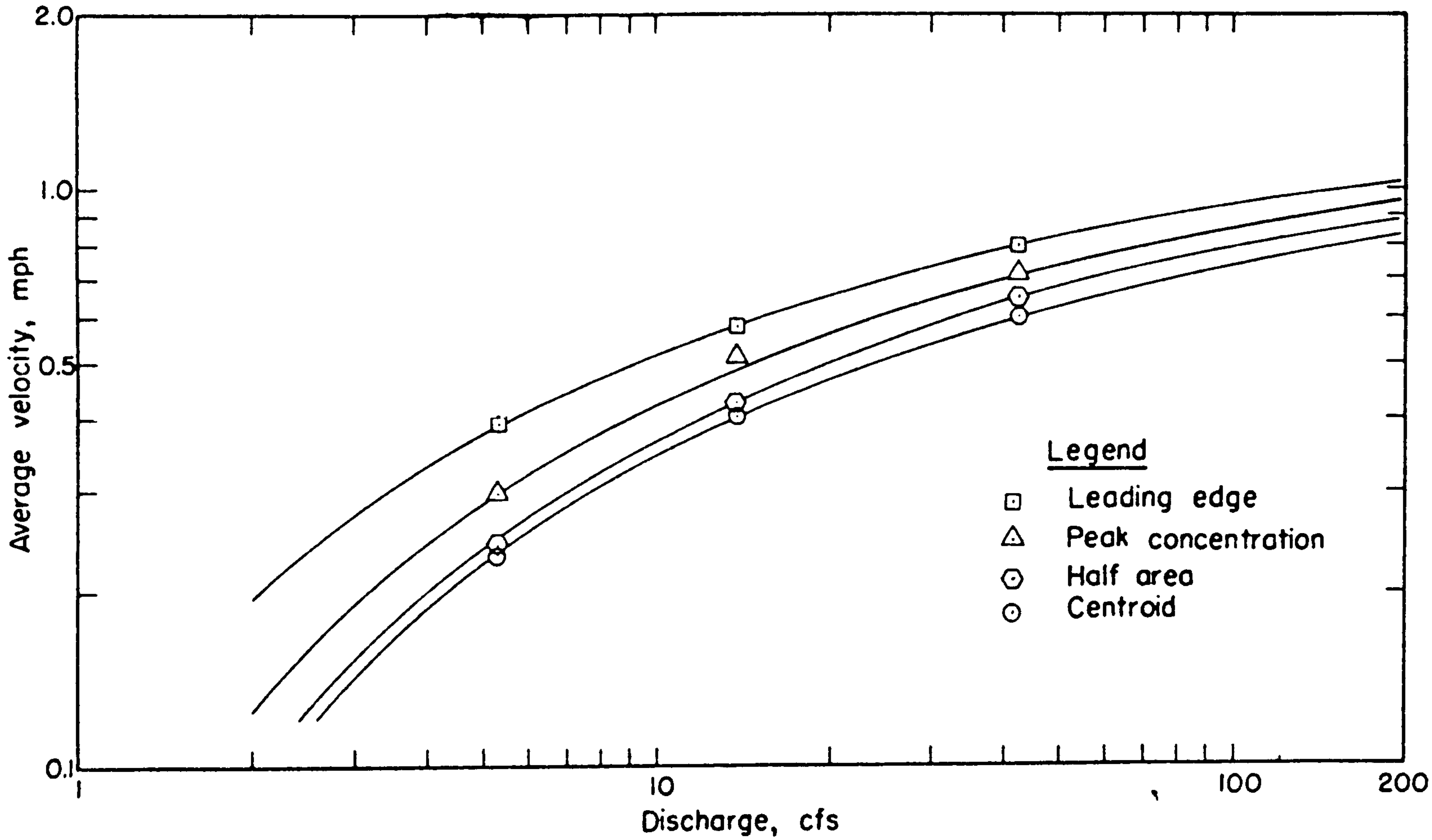

Figure 9. Relationships between average stream velocity and discharge based on the combined discharge below Squaw Creek and the Ames WPCP (Dougal, 1969)

The second method involved the replacement of the missing equipment at the gaging station house to return the station back to its original mode of operation.

While the use of the wire weight gage has a decidedly superior economic advantage over the latter method, it was found to have serious shortcomings, especially at low flow conditions. The drawbacks are primarily due to the erosion of the concrete overflow control weir, which has subsequently lowered the local streambed, in addition to relocating the main channel. Consequently, the wire weight gage is, at present, too short to measure flows under about 100 cfs. Even if the wire were lengthened, it would ultimately be resting on the exposed portion of the streambed, where continued gaging of water levels would be impossible.

Costs of moving the wire weight gage or replacing the missing station house equipment were not extensively investigated due to the overriding problem posed by the eroding concrete overflow control weir. This erosion would repeatedly render the existing stage-discharge curve inadequate, thereby requiring a new one to be developed periodically. Optimistically, it would be desirable to rebuild and reinforce the concrete overflow control weir if it were decided to resume using this gaging station. The costs and details of accomplishing this task are beyond the scope of this project, and so other methods of arriving at the discharge were sought.

The third method investigated was that of making direct velocity measurements by use of a current-meter. Two types of current-meters are available for use through the Civil Engineering Department at ISU. Both current-meters were manufactured by "W. and L. E. Gurley" and each consists of six conical cups which rotate about a vertical axis in the flowing water. One of the current-meters (Model #622) is substantially larger than the other current-meter (Model #625) and is therefore restricted to water depths of 1.5 feet or more because of its physical size (Based on measurements at six-tenths water depth). Measurements of water velocity, with the larger model, are made by lowering the current-meter with an attached weight into the water from a bridge (or other structure) by a cable. Measurements with the smaller current-meter are made quite differently, as the conical cups are simply mounted on a stick. Hence, velocity measurements must be made by wading in the river itself. This smaller current-meter, commonly referred to as a "Pygmy stick", can measure velocities (based on six-tenths water depth) in as little as 0.3 feet of water, thus making it very suitable for low flows.

The larger current-meter was used by the author and C.S. Oulman to make discharge measurements at four different sites on the Skunk River on July 28, 1983. The results of the exercise can be found in Appendix A. While primarily intended to measure differences in discharges while moving downstream, the exercise also made it evident that this type of discharge measurement was very time consuming, besides having questionable accuracy. In addition, this type of measurement poses extreme difficulties in data collection during poor weather and ice conditions. These difficulties apply equally to the use of the "Pygmy stick" current-meter, as well as the larger current-meter. Therefore, while it may be entirely feasible to utilize this direct measurement approach, it does not lend itself well to repeated measurements which may be required for a research endeavor. Consequently, another method for measuring the discharge was sought.

The <u>fourth</u> method investigated involved approximating the discharge measurement at the South Sixteenth Street gaging station by obtaining the discharge immediately upstream at the two operational gaging stations. The two operational gaging stations include one located on the Skunk River, just north of Ames near Hallett's Quarry and the other located on Squaw Creek, just east of the ISU campus at Ames. (The IGS identification numbers corresponding to these gaging stations are 05-4700.00 and 05-4705.00, respectively.) Appendix B gives a more detailed location of each gaging station, as well as other pertinent discharge information. These stations will be referred to as the gaging station "near Hallett's Quarry" and the gaging station "on Squaw Creek," respectively, for the remainder of this thesis. This fourth method is presented in the results section of this thesis.

EVALUATION OF IOWA'S WATER QUALITY MODELS AND WLAS

Introduction

Responding to the Federal Water Pollution Control Act of 1972, the State of Iowa assigned the responsibility of protecting and maintaining Iowa's surface and ground water quality to the Department of Environmental Quality (IDEQ), now the Department of Water, Air, and Waste Management (DWAWM). A major element of the Federal act was to establish "basin planning" as a means to obtain "water quality suitable for the protection and propagation of fish and wildlife, as well as, for recreational activities in all surface waters" (IDEQ, 1976).

The main objective of Iowa's water quality program is to provide acceptable water quality conditions for "designated" water uses, which in turn limits the amount and quality of effluent which can be discharged to Iowa's streams and rivers.

Four major water "use classifications" have developed from Iowa's main objective and are listed below:

1. Class A - Primary Contact Recreation,
2. Class B - Wildlife, Secondary Contact, Recreation, and Aquatic Life (with subclasses for cold and warm water),
3. Class C - Potable Water Supply, and
4. General Water Quality Criteria.

Each "use classification" grouping has its own set of water quality standards which establishes limits for dissolved oxygen, pH, turbidity, fecal coliform, temperature, chemical constituents, and radioactive substances. All of the Skunk River system near Ames is classified as

Class B (warm water), and includes the Squaw Creek from near Gilbert to its mouth and the South Skunk River from Story City to near Oskaloosa (IDEQ, 1976).

To maintain acceptable water quality conditions, the State of Iowa monitors waste discharges through a coordinated effort with the National Pollutant Discharge Elimination System Discharge Permit Program (NPDES). In addition to setting water quality limitations on the effluent to be discharged, the program prescribes compliance schedules for bringing about correctons, and requires the permit holder to monitor the effluent's water quality characteristics. (Currently only "point" sources of pollution are being analyzed, with "nonpoint" sources to receive greater consideration in the future.) Limiting effluent discharge concentrations are arrived at by a "waste load allocation" process, where the "assimilative" nature of the water body and the effluent's characteristics are taken into account to maintain appropriate constituent levels.

Iowa uses various water quality models to simulate the response of Iowa's streams (or rivers) to pollutant loads discharged into them. In general, the constituents which most often violate the water "use classification" limitation levels are dissolved oxygen (D.O.) and ammonia (NH_3) (IDEQ, 1976). Consequently, these two parameters are the ones primarily modeled by the State in establishing effluent discharge limitations. Other water quality parameters found in violation are modeled on a case by case basis only (IDEQ, 1976).

Prior to 1983, the mathematical model used to simulate D.O. and NH_3 levels in a stream was one developed by Stanley Consultants, Inc.

in 1975 (IDEQ, 1975a). In June of 1983, JRB Associates modified the "Stanley" model to account for inadequacies in the older model (JRB, 1983a). In addition to modifying the "Stanley" model, JRB introduced a more sophisticated mathematical model for the state of Iowa to use in its WLA process. This model was originally developed for the Environmental Protection Agency (EPA) and is called Qual II. The version of Qual II which JRB introduced was modified by the State of Vermont and shall be referred to as the "Vermont" version of Qual II hereafter (JRB, 1983a).

Currently, the State employs the modified version of the original "Stanley" model and the Vermont version of Qual II, in addition to hand calculations, in determining WLAs. The use of the original "Stanley" model has been discontinued, but will be discussed for comparison with the other models. The actual model chosen depends on the "degree of sophistication" required in the WLA process. A detailed explanation of the sequencing procedure used in the model selection can be found in the DWAWM draft of its WLA procedure (DWAWM, 1984). Briefly, however, hand calculations are used when the assimilative capacity of the stream will not be exceeded with minimum treatment levels or standard secondary effluents. If water quality levels are exceeded, the modified version of the original model is then used. Qual II is used if the modified model suggests that advanced treatment will be required. JRB Associates recommended that the revised model be used only to screen those stream reaches where it appeared that advanced wastewater treatment facilities would need to be constructed, thereby justifying the use of the more sophisticated Qual II in establishing more precise WLAs (JRB, 1983a).

Hand Calculations

Hand calculations are used to determine whether the stream has the capacity to assimilate wastes when the minimum required level of treatment is used. If the capacity is not exceeded, the final WLA will be the appropriate standard BOD and ammonia limitations imposed by the level of treatment.

Available stream capacities for BOD and ammonia are calculated using the following equations.

For carbonaceous BOD:

$$BOD_L = (Qu + Qd)\ 20.0\ \text{lbs/cfs-day}$$

where,

BOD_L = Carbonaceous five-day BOD stream capacity (lbs/day),

Qu = 7Q10 low flow (cfs), and

Qd = Future dry weather wastewater discharge (cfs).

For ammonia nitrogen

$$NH_3\text{-}N_L\ (\text{summer}) = (Qu + Qd)\ 11.0\ \text{lbs/cfs-day and}$$

$$NH_3\text{-}N_L\ (\text{winter}) = (Qu + Qd)\ 24.0\ \text{lbs/cfs-day}$$

where,

$NH_3\text{-}N_L$ = Ammonia nitrogen stream capacity (lbs/day).

The "Stanley" Water Quality Model

The model developed by Stanley Consultants in 1975, monitored the levels of D.O., NH_3, and carbonaceous biochemical oxygen demand (CBOD); assuming completely mixed and steady-state flow conditions (IDEQ, 1975b). Completely mixed conditions assume the river to be homogenous horizontally across the width of the river, as well as, vertically in depth. Steady-state conditions assume no change in the rate, velocity, or depth of flow, with respect to time. These assumptions are rather common in water quality modeling and also apply to the Qual II model. Because the river is nonuniform along its length, the river system is broken up into many sections or reaches, where the physical constraints of steady-state can be reasonably applied. The conditions of steady-state also apply to temperature and biological conditions throughout the stream reach as well. New reaches can be expected at each tributary, wastewater discharge location, change in river characteristic (geological, biological, etc), or at a dam.

The predictive equations used in the original "Stanley" model are shown in Table 9. Equation 1 models the dissolved oxygen deficits as a function of time downstream from a discharge point using the familiar modified Streeter-Phelps equation. D.O. deficits are deducted from D.O. saturation values, which are temperature dependent. (See Table 12 for the D.O. saturation equation used.)

The modified Streeter-Phelps equation models the changes in D.O. that result from the biochemical breakdown of carbonaceous and nitrogenous (during nitrification of NH_3) matter, in addition to the physical input of oxygen into the stream, called reaeration. The effects

Table 9. Predictive equations used in the original "Stanley" model

$$D(t) = \frac{K_1 L_o}{K_2 - K_1}\left(e^{-K_1 t} - e^{-K_2 t}\right) + \frac{K_N N_o}{K_2 - K_N}\left(e^{-K_N t} - e^{-K_2 t}\right) + D_o e^{-K_2 t} \qquad (1)$$

Where:

$D(t)$ = DO deficit at time t (mg/l).

K_1 = Carbonaceous deoxygenation rate constant at temperature $T(day^{-1})$.

L_o = Initial ultimate carbonaceous BOD concentration (mg/l).

K_2 = Reaeration rate constant at temperature $T(day^{-1})$.

t = Time of travel through reach (day).

K_N = Nitrogenous deoxygenation rate constant at temperature $T(day^{-1})$.

N_o = Initial nitrogenous BOD concentration (mg/l).

D_o = Initial DO deficit at temperature T(mg/l).

T = Temperature (^oC).

$$L(t) = L_o e^{-K_1 t} \qquad (2)$$

Where:

$L(t)$ = Ultimate carbonaceous BOD at time t(mg/l). L_o, K_1, and t as previously defined above.

$$N(t) = N_o e^{-K_N t} \qquad (3)$$

Where:

$N(t)$ = Ultimate nitrogenous BOD at time t(mg/l). N_o, K_N, and t as previously defined above.

of algal photosynthesis, respiration, and assimilation are disregarded, as well as, benthal deposition, rescouring, and pollutant volatilization in the "Stanley" model.

While it appears that ammonia is not being modeled, it is indirectly, as the ammonia concentration (in mg/l as N) is converted to an approximate nitrogenous biochemical oxygen demanding material (NBOD) by the factor 4.5. Ultimate CBOD is assumed to be 1.5 times the value of the 5 day uninhibited value of the laboratory BOD test at $20^{o}C$. A great deal of controversy surrounds the use of the uninhibited BOD test as it appears that a double counting of "oxygen demanding" material is occurring. The uninhibited BOD test will be specifically addressed in the discussion section of this paper.

Ultimate CBOD is modeled using equation 2 of Table 9, with the nitrogenous portion (NBOD) modeled using equation 3. Both equations use first-order reaction kinetics to estimate the decay rate to preserve the model simplicity.

Input data required for the predictive equations are either entered as constants for the given reach, or calculated from equations within the program using other input data. Two calculated input variables include time of travel (t) and the reaeration coefficient (K_2). The equations used to arrive at these values can be found in Table 10. (Note that two methods exist for determination of the velocity term depending on the input data available to the modeler.) A list of all the input requirements can be found in Table 11. Table 11 also lists the value typically assigned by the State for each input parameter, the principal source used in finding the values, and those specific values used for the

Table 10. Calculated input variables for the original "Stanley" model

$$t = \frac{D}{86,400V} \quad (1)$$

Where:

t = Time of travel through reach (day).

D = Distance along the travelled reach (feet).

V = Mean velocity through the reach (feet/second).

$$V = Q/Wd \quad (2a) \qquad \text{or} \qquad V = aQ^b \quad (2b)$$

Where:

Q = River discharge (cfs).

W = Water surface width (feet).

d = Mean water depth (feet).

a,b = Empirical constants from historical stream data (dimensionless).

$$d = \left[\frac{Qn}{1.5WS^{1/2}}\right]^{3/5} \quad \text{(Used with 2a only).} \quad (3)$$

Where:

n = Mannings roughness coefficient (dimensionless).

S = Channel slope (dimensionless).

$$K_2 = \frac{c\Delta h}{t} \quad (4)$$

Where:

K_2 = Reaeration rate constant at 20°C in base e (day^{-1}).

C = Tsivoglou gas escape coefficient ($feet^{-1}$).

Δh = Change in water surface elevation (feet).

t as previously defined above.

Table 11. Input parameters required for use with original "Stanley" model with specific values for Ames WLA

Parameter	Typical Values	Source of Information	Ames WLA Value
Initial D.O. deficit, D_o	varies	Previously modeled or assumed values (PMOAV)[a]	6.39 mg/1 (Winter) 6.60 mg/1 (Summer)
Initial ultimate (CBOD) L_o	varies	PMOAV	26.9 mg/1 (Winter 17.2 mg/1 (Summer)
Initial NH_3-N concentrations, N_o	varies	PMOAV	3.6 mg/1 (Winter) 0.65 mg/1 (Summer)
Water Temperature, T	1-2^{o}C Winter	-	2^{o}C Winter[b] 26^{o}C Summer
Initial Flow Rate	7Q10 plus upstream discharges	Bulletin No. 13 (Lara, 1979)	1.14 cfs[c]
Stream Width[d], W	varies	USGS data or field estimates	8 feet above Ames discharge 28 feet below Ames discharge
Channel Slope, S	varies	USGS data or field estimates	0.000676 ft/ft
Mannings roughness[d]	0.035	Hydrology text and field survey	0.05
Leopold - Maddock[d] empirical constants, a and b	varies	Regression analysis of historical flow data	a = 0.187 b = 0.3432 V in mph

Background stream flow	varies	Analysis of discharges and 7Q10's	0.06 cfs/mile loss
Tsivoglou gas escape coefficient "C"	0.054 for 15 to 3000 CFS 0.115 for 0 to 15 CFS	-	0.115 (foot^{-1})
% Ice Cover (Ice)	varies	Climatological data or field estimates	90% (Winter) 0% (Summer)
Reaeration Rate Constant, (K_2)	formula	Tsivoglou equation	3.306/day at 20°C
Carbonaceous Deoxygenation	0.2/day at 20°C	Assumed value	0.2/day at 20°C
Nitrogenous Deoxygenation Rate Constant, (K_N)	0.3 day at 20°C	Assumed value	0.3/day at 20°C
Discharge Rate	varies	Wet weather flow at year 2000[e]	9.9 mgd
Reach Length, D	varies	USGS Topographic maps	-

[a]See Table 18 for assumed values.

[b]Allowance made for winter temperature rise to 4°C in first 1.8 miles below Ames discharge.

[c]Wasteload allocation value assigned in 1982 for design basis flow calculations in year 2000.

[d]Use with equation either 2a or 2b required for velocity determination as required.

[e]Wet weather flows changed to dry weather flows in 1984.

Ames WLA process. (Appendix C also lists Ames WLA values.) While the input data required in Table 11 can be obtained from published literature values, it was recognized that future stream investigatons would verify the particular constants and assumptions used (IDEQ, 1975b).

Table 12 lists the equations used to adjust the deoxygenation and reaeration rate constants, as well as, the D.O. saturation values. Equations 1 through 3 are used to adjust for changes due to instream temperature conditions and are of the form

$$K(T^{o}C) = K(20^{o}C) \times \Theta^{(T-20)^{o}C}$$

where, T is the temperature to which it is being adjusted.

The reaeration rate constant, K_2, is also adjusted by equation 4, of Table 12, which is used to reduce K_2 due to "ice cover." Currently, the reduction in K_2 is in direct proportion to the percent of "ice cover." This is a slight change from the original draft of the Supporting Document for the "Stanley" model where the reaeration rates were reduced in proportion to the percent of "ice cover minus 5 percent" (IDEQ, 1975b). This accounted for some reaeration even at 100% ice cover. Thus, 100% ice cover would result in 95% reduction in K_2 rates, 95% ice cover would result in 90% reduction in K_2 rates, and so on. Now, to avoid having zero reaeration with 100% ice cover, the State has simply put an upper limit of 95% on the amount of possible ice cover for winter WLAs. As a result, 95% ice cover results in 95% reduction in K_2 rates, 90% ice cover results in 90% reduction in K_2 rates, and so on. Equation 5 is used to predict D.O. saturation values for temperature.

Table 12. Equations used to adjust the deoxygenation and reaeration rate constants, and the dissolved oxygen saturation value for the original "Stanley" model

$K_{1(T)} = K_1(20) \times 1.047^{T-20}$.		(1)
$K_{2(T)} = K_2(20) \times 1.024^{T-20}$.		(2)
$K_{N(T)} = K_N(20) \times [(0.058\ T) - 0.16]$	: $T > 3^oC$.	(3a)
$K_N(T) = 0$	: $T \leq 3^oC$.	(3b)

Where:

T = Water temperature (^oC).

$K_{2(ice)} = K_{2(T)}(ice)$. (4a)

ICE = (1 - Percent ice cover). (4b)

Where:

$K_{2(ice)}$ = Adjusted reaeration rate for ice cover in base e (day^{-1}).

ICE = Factor reflecting the effect of ice cover on reaeration rate (dimensionless).

$C_s = 24.89 - 0.426\underline{T} + 0.0037\underline{T}^2 - 0.00001335\underline{T}^3$. (5)

Where:

C_s = Saturation value for oxygen at temperature $\underline{T}$, L_o, K_1, and t as previously defined above.

$\underline{T}$ = Water temperature (^oF).

Modifications to the "Stanley" Model

A calibration and verification study of the original model occurred in 1978 by TenEch Environmental Consultants (TenEch, 1978a). Several deficiencies were noted in the models ability to predict NH_3 concentrations, in both winter and summer, and D.O. concentrations in the summer. As a result, JRB Associates were contracted by the EPA to evaluate the "Stanley" model and subsequently to make modifications to the model to improve its predictive capabilities. The following changes were taken from the "User's Manual for the Modified Iowa DEQ Model" as published by JRB Associates in June of 1983 (JRB, 1983a).

The revised model made three substantial changes to the original "Stanley" model as indicated below, while still preserving its simplistic structure:

1. Addition of a "photosynthesis minus respiration" (P-R) term to improve D.O. simulation in the summer,
2. Allowance for algal uptake of NH_3 by phytoplankton to improve NH_3 simulation in the summer, and
3. Replacement of the K_N temperature adjustment equation to improve NH_3 simulation in the winter.

The equations used in the modified JRB model will be presented in an identical format to that used in presenting the "Stanley" model. To highlight changes, an asterisk (*) will appear behind the applicable equation number in each table. The initial assumptions of completely mixed and steady-state conditions still apply.

Table 13 lists the predictive equations used for D.O. deficits and the degradation of ultimate CBOD and NBOD. While the D.O. deficit

Table 13. Predictive equations used in the modified JRB model

$$D(t) = \frac{K_1 L_o}{K_2 - K_1} \left(e^{-K_1 t} - e^{-K_2 t}\right) + \frac{K_N N_o}{K_2 - K_N} \left(e^{-K_N t} - e^{-K_2 t}\right)$$

$$+ D_o e^{-K_2 t} + \frac{(R-P)}{K_2} \left(1 - e^{-K_2 t}\right) \qquad (1)^*$$

Where:

$D(t)$ = D.O. deficit at time t (mg/l).

K_1 = Carbonaceous deoxygenation rate constant at temperature T (day^{-1}).

L_o = Initial ultimate carbonaceous BOD concentration (mg/l).

K_2 = Reaeration rate constant at temperature T (day^{-1}).

t = Time of travel through reach (day).

K_N = Nitrogenous deoxygenation rate constant at temperature T (day^{-1}).

N_o = Initial nitrogenous BOD concentration (mg/l).

D_o = Initial DO deficit at temperature T (mg/l).

R = Algal respiration oxygen utilization (mg/l/day).

T = Temperature (^oC).

$$L(t) = L_o e^{-K_1 t} \qquad (2)$$

Where:

$L(t)$ = Ultimate carbonaceous BOD at time t (mg/l). L_o, K_1, and t as previously defined above.

Table 13. continued

$$N(t) = N_o e^{-K_1 t} \qquad (3)*$$

Where:

$N(t)$ = Nitrogenous BOD concentration at time t (mg/l). N_o, K_N, and t as previously defined above.

equation appears to be the only equation that has undergone a change from the "Stanley" model, an asterisk appears behind the NBOD equation as a result of a change in the relationship between NH_3-N and NBOD. The modified JRB model uses a factor of 4.33 to covert the NH_3-N concentration to NBOD. This is in comparison to the "Stanley" model which used 4.5. The slight reduction comes from the synthesis-oxidation equations presented earlier in the literature review.

The first major change, however, is in the D.O. deficit equation where a term has been added to account for oxygen production due to algal (phytoplanktonic) photosynthesis. The last term accurately shows the photosynthesis minus respiration (R-P) component as (R-P), since D.O. deficits are being predicted.

Table 14 lists the equations necessary to arrive at values for P and R. The equations presented were taken from a fresh water stream model that JRB refers to as "MS-ECOL" (JRB, 1983a). Adequate documentation on this model was unavailable, however the equations shown in Table 14 are similar to those found elsewhere in the literature (Zison et al., 1978). Typical values for these constants and other variables can be found in Table 15, along with their expected ranges. It should be pointed out that the growth rate (GP) must be calculated outside the model for each stream reach.

The second major change in the "modified JRB" model lies in the uptake of NH_3 by phytoplanktonic algae. The amount of NH_3-N which can be assimilated by algae is expressed by the equation presented in Table 16. This equation was also adopted from the MS-ECOL model (JRB, 1983a). Table 15 also shows the typical values used in the WLA process

Table 14. Algal photosynthetic and respiration terms for modified JRB model

---

$$P = \frac{(OP)(GP-DP)(CHLA)}{AP} \tag{1}$$

where

- P = Photosynthetic oxygen production (mg/l/day)
- OP = mg oxygen produced by algae/mg algae
- AP = ug chlorophylla/mg algae
- GP = Algal growth rate (day^{-1})
- DP = Algal death rate (day^{-1})
- CHLA = Chlorophyll a concentration (ug/l)

$$R = 0.025\ CHLA \tag{2}$$

where

- R = Algal respiration oxygen utilization (mg/l/day) and CHLA as previously defined

$$GP = \bar{u}\ \frac{N}{(N + K_{MN})}\ \frac{PO}{(PO + K_{MP})}\ \frac{LI}{(LI + K_{LI})} \tag{3}$$

where

- GP = Local algal growth rate at 20° (day^{-1})
- $\bar{u}$ = Maximum specific algal growth rate at 23°C (day^{-1})
- N = Sum of observed instream concentrations of NH_3-N and NO_3-N (mg/l)
- K_{MN} = Michaelis-Menton half-saturation constant for total inorganic N (mg/l)
- PO = Observed instream concentration of inorganic phosphorous (mg/l)
- K_{MP} = Michaelis-Menton half saturation constant for inorganic PO_4-P (mg/l)
- LI = Average incident light intensity ($Kcal/m^2$-sec)
- K_{LI} = Michaelis-Menton half saturation constant for light intensity ($Kcal/m^2$-sec)

NOTE: All equations are entirely new to the original "Stanley" model

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Table 15. Input parameters required for use with the modified JRB model

Parameter	Typical values	Recommended value
Initial Conditions D_0, L_0, N_0, T flow rate	Previously modeled or assumed values	(See Table 18)
Stream width, W	varies	Field data
Channel slope, S	varies	Field data
Mannings roughness coefficient, n	0.02 to 0.20	0.035
Leopold-Maddock empirical constants, a, b	varies	Historical data
mg Oxygen produced/ mg algae, OP	1.4 to 1.8	1.63
mg Chlorophyll-a/mg algae, AP	10 - 100	Calibrate
Algal growth rate, GP	varies, (day^{-1})	Requires calculation outside the model
Sum of instream NH_3-N and NO_3-N concentrations, N	varies, (mg/1)	Sampling data
Instream PO_4-P concentration	varies (mg/1)	Sampling data
Average incident light intensity, LI	varies ($Kcal/m^2$-sec)	National Weather Service data

Table 15. *Continued*

Michaelis-Menton half saturation constant for Nitrogen K_{MN}	0.01 to 0.20 (μg/l)	Calibrate
Michaelis-Menton half saturation constant for Phosphorus K_{MP}	0.01 to 0.05 (μg/l)	Calibrate
Michaelis-Menton half saturation constant for light K_{LI}	0.002 to 0.006 (Kcal/m^2-sec)	0.0035
Algal death rate, DP	0.024 or 0.24 day^{-1})	Use higher value if nutrient are scarce or chlorophyll-a concentration exceeds 50 u g/l, otherwise use lower value
Chlorophyll-a concentration, CHLA	(μg/l)	Sampling data
Maximum algal growth rate, 2 (day^{-1})	1-3 (day^{-1})	
Mg Nitrogen/μg Chlorophyll-a, ANP	0.0007 to 0.009	Calibrate
Preferential NH_3 uptake factor, NF	0 to 0.9	Calibrate
Carbonaceous deoxygenation rate constant, K_1	0.02 to 3.4 (day^{-1})	Calibrate
Reaeration rate constant, K_1	varies	Tsivoglou equation
Nitrogenous deoxygenation rare constant, K_N	0.3 to 3.0 (day^{-1})	Calibrate
Tsivoglou gas escape coefficient, C	0.054 and 0.115 (ft-1)	Use lower value with Q from 15 to 3000 cfs and higher value if under 15 cfs.

Table 16. Amount of NH_3-N assimilated by algae in modified JRB model

$$UP = \frac{(GP)(ANP)(NF)(CHLA)(e^{(GP-DP)(t)} - e^{-(K_N)(t)})}{(GP - DP + K_N)} \qquad (1)$$

where

UP = Amount of NH_3-N removed in a reach by phytoplankton (mg/l)

ANP = (mg N)/(ug chlorophyll-a)

NF = Fraction of NH_3 preferred for algal uptake (dimensionless)

t = Time of travel through reach (day)

K_N = Nitrogenous deoxygenation rate constant a temperature T (day^{-1})

GP = Local algal growth rate at 20^o (day^{-1})

DP = Algal death rate (day^{-1})

CHLA = Chlorophylla concentration (ug/l)

NOTE: This equation is entirely new to the original "Stanley" model.

for these constants.

Table 17 shows the equations used to adjust the deoxygenation and reaeration rate constants, as well as the D.O. saturation values. Equations 1, 5, and 8 are unchanged from the original "Stanley" model. Equation 2 has been modified slightly with a value of θ taken from Vermont's Qual II model. Equation 4 is required in the modified model, due to the installation of the (P-R) term and is of the form currently found in the literature (Zison et al., 1978). Equations 3, 6, and 7 comprise the third major change to the original "Stanley" model and attempts to improve NH_3 simulation in the winter time. Equation 3 changes the value of θ from the original "Stanley" model, which was based on rate changes in biological treatment facilities, to a value more commonly used in stream modeling (IDEQ, 1975a and JRB, 1983a). D.O. concentrations also affect the rate of nitrification and hence, equations 6 and 7 are included to reduce K_N during low D.O. levels and was adopted by JRB Associates from Wisconsin's Qual III model (JRB, 1983a).

The Fortran source code, as published by JRB Associates, for the Leopold-Maddock version of the "modified JRB" model can be found in Appendix D (JRB, 1983a). The Mannings "n" version is similar. The next section will specifically analyze the modeling procedure used by the DWAWM for the "modified JRB" model. The original "Stanley" model follows the same general procedure with minor exception being made to the omission of the added terms in the "JRB" model.

Table 17. Equations used to adjust the deoxygenation and reaeration rate constants, and the dissolved oxygen saturation value for the modified JRB model

---

$$K_{1(T)} = K_{1(20)} \times 1.047^{T-20} \quad (1)$$

$$K_{2(T)} = K_{2(20)} \times 1.0159^{T-20} \quad (2)*$$

$$K_{N(T)} = K_{N(20)} \times 1.080^{T-20} \quad (3)*$$

$$GP_{(T)} = GP_{(20)} \times 1.047^{T-20} \quad (4)*$$

where:

T = Water temperature (^{o}C)
K_1, K_2, K_N, and GP as previously defined

$$C_s = 24.89 - 0.426T + 0.0037T^2 - 0.00001335T^3 \quad (5)$$

where:

C_s = Saturation value for oxygen at temperature T and standard pressure (mg/l)
T = Water temperature (^{o}F)

$$CK_N = K_N \times PN \quad (6)*$$

$$PN = 1-e^{-(.52)(DO)} \quad (7)*$$

where:

CK_N = Adjusted nitrification rate at temperature T (day^{-1})
PN = Nitrification reduction factor (dimensionless)
DO = Dissolved oxygen concentration (mg/l)
K_N as previously defined

Table 17. Continued

$$K_2 = \frac{(ICE)(C)(\Delta h)}{t} \qquad (8a)$$

$$ICE = (1 - \%\ \text{ice cover})(\text{Dimensionless}) \qquad (8b)$$

where:

ICE = Factor reflecting effect of ice cover on reaeration rate

C = Tsivogloy gas escape coefficient (ft^{-1})

Δh = Difference in water surface elevation between upstream and downstream ends of reach (ft)

t = Time of travel through reach (day)

Modeling Procedures and the WLA Process

Four steps are involved in the modeling procedure used by the DWAWM in reaching WLA's after model selection as outlined below:

1. Stream Description and Data Collection,
2. Model Calibration,
3. Model Verification, and
4. Establishment of WLA.

The first step in the procedure involves the physical description of the stream itself, including collection and analysis of the data available for the stream in queston. As noted earlier, each stream is divided into reaches where the assumptions of steady-state can reasonably be applied. Each reach is also divided into sections which allow for calculation of CBOD, NH_3-N, and D.O. concentrations throughout the reach length.

The IDEQ "Supporting Document" of 1975, describes procedures and available data sources that a modeler may use in describing a stream system (IDEQ, 1975b). The following headings were addressed in the document and are briefly presented below:

River mileage - Establishment of "reach" lengths are required after the locations of all tributaries, wastewater discharges, or changes in river's characteristics are known. Several sources of base maps are available to measure these lengths accurately to a tenth of a mile (or less). USGS (United States Geological Survey) topographic contour section or quadrangle maps seem to provide adequate information in establishing these lengths. Other sources such as state and county road maps or the Corp of Engineer's "established river mileage" may augment the "best" base map available.

River Channel Slopes - Typically, river channel slopes have been estimated from the best available topographic map. These channel slopes can be assumed to be equal to the water surface slopes when calculating velocities using Manning's equation. Other sources include the use of existing or new surveys and published data on average slopes.

Field Reconnaissance - Actual field excursions to the river itself are invaluable to the modeler, especially if ample photographs can be taken for future reference while in the office. Information that can be obtained in the field include:

1. Precise location of wastewater discharges,
2. Location, physical description and condition of any dam or other structure which would pond water,
3. River width determination,
4. Shape of channel cross sections,
5. Basic channel characteristics to aid in determination of channel roughness coefficients, if the Mannings-n approach is to be used in estimating stream velocities, and
6. Checking of river channel slopes.

Discharge Information - The quantification of the amount of flow in a river is an inherent requirement for all water quality models. The river discharge data directly or indirectly affects nearly every calculation made in any of the Iowa models. Consequently, all inputs to the river system must be known to the modeler. These inputs include all wastewater discharge flows, established stream (low) flows, and all groundwater inflows or outflows.

Quantities of wastewater effluent discharge flows were formerly based on future wet weather discharge conditions as determined from the current NPDES records or from design data. A 1984 proposal by DWAWM changed the wet weather discharge condition to dry weather conditions (DWAWM, 1984). Currently, Iowa predicts discharges to the year 2000, thus allowing for future community growth which otherwise is unaccounted for in the WLA process.

Stream flows used in the WLA process are established at a prescribed statistical frequency of occurrence and adjusted for waste dischargers and groundwater contributions upstream. The State of Iowa uses the average 7 day low flow condition that occurs once in every 10 years (7Q10) for a basis in stream modeling. To this base flow, the State adds all future wastewater discharges upstream, and corrects this for the present groundwater inflow or outflow contributions. Groundwater inflows or outflows are uniformly distributed along the main channel of the river if differences exist (usually this is the case) between the summation of tributary inflows and waste discharges versus the gauged flow. Essentially, this procedure increases the statistical 7Q10 low flow by an amount equivalent to the incremental increase in future flow conditions.

7Q10 low flows have been determined for Iowa streams and are available for each gaging station (Lara, 1979). The flows at these gaging stations can be proportioned to other parts of the river or its tributaries in proportion to the corresponding drainage areas. The drainage areas may be determined from contour maps or other published material such as the "Iowa Highway Research Bulletin No. 7" (Larimer, 1957).

Velocity Determinations - Input requirements vary for the determination of stream velocity depending on which method is employed for the calculation. Two methods are available and consist of the Leopold-Maddock and Manning's equations. The Leopold-Maddock equation requires the input of two regression constants arrived at from historical stream data, which relates velocity and discharge over a range of flows. The use of the Manning's equation requires knowledge of the river's width and roughness coefficient. Widths can be obtained from field observation or from periodic USGS calibrations of each gaging station for the low flow discharge being modeled. Roughness coefficients can be arrived using tables and techniques in hydraulic textbooks, or from back calculations, using appropriate discharge-velocity measurements.

Rate Constants - Typical values (or formulations) for rate constants are arrived at using the data presented earlier in Tables 11 and 15.

Dams and Impoundments - Treatment of dams and impoundments may be accomplished by treating the impoundment as a slower moving section of the river with a flat slope (corresponding to its hydroscopic gradient) and treating the dam as a very short reach (0.001 mile long) with a steep slope (corresponding to the height divided by length).

Winter Ice Cover - Little information is available regarding the percentage of ice cover normally on a river. Complete ice cover was not assumed to be coincident with winter low flows; hence, general climatic conditions and/or field observations must be relied upon (IDEQ, 1979).

Water Quality Assumptions - Water quality inflow to a river system is either taken from a previously modeled stream segment or assumed from the information presented in Table 18. A change proposed in 1984, would eliminate the usual modeling of the entire basin length (DWAWM, 1984) and would restrict modeling to the assimilative reach of the stream only. Hence, fewer stream segments will now be modeled ahead of waste dischargers. Table 18 also lists other water quality assumptions for effluent discharges.

The second step of the modeling procedure, after the physical description, involves the calibration of the model input parameters. Some of the input parameters are assumed from the outset, while others are established only through model calibration.

The following steps are suggested by JRB Associates to expeditiously facilitate the calibration step (JRB, 1983a).

1. Back calculate the value of the rate constant K_1 to successfully simulate observed ultimate BOD concentrations. This is convenient to calibrate first since the concentration of ultimate BOD is entirely dependent on the value of the rate constant K_1,
2. Assume a reasonable value of K_N and Tsivoglou's gas escape coefficient C. If uptake of ammonia by algae is anticipated, a large value of K_N may result in the undersimulation of NH_3-N. This, of course, should be avoided,
3. Establish photosynthesis and respiration terms by: a) calculating the local algal growth rate (GP) outside the program and b) entering values for GP, OP, AP, DP and CHLA to the program. Because the range of algal death rates is very small, the maximum algal growth rate ($\bar{u}$) will have the

Table 18. Assumed water quality levels for model input[a]

Inflow Source	Summer				Winter			
	BOD[b] (mg/l)	NH -N (mg/l)	D.O. (mg/l)	Temperature (°C)	BOD[b] (mg/l)	NH -N (mg/l)	D.O. (mg/l	Temperature (°C)
Initial Conditions	6.0	0.0	(Sat.)[c]	26	6.0	0.5	(Sat.)[c]	1-2
Tributaries	6.0	0.0	(Sat.)[c]	26	6.0	0.5	(Sat.)[c]	1-2
Ground water	6.0	0.0	2.0	51	6.0	0.5	2.0	1-2
Secondary Treatment	-	10-15	4.0	20	-	10-15	4.0	9
Advanced Treatment	-	-	5.0	20	-	-	6.0	9
Aerated Effluents	-	-	6-8	20	-	-	6.8	9
Industrial Plants	Each Discharge Handled Individually							

[a]Data from IDEQ (1979).

[b]Ultimate carbonaceous BOD.

[c]Saturation.

largest impact upon D.O. simulation. A high maximum growth rate will increase simulated photosynthesis, which will in turn increase simulated D.O. levels,

4. Adjust D.O. calibration by varying Tsivoglou's C and adjusting the terms above. Since the range of applicable C values is small, the D.O. simulation may be relatively insensitive to changes in C,
5. Adjust the NH_3-N simulation, by establishing a preference factor if preferential algal uptake is expected, and
6. The calibration is complete if the modeler has successfully simulated BOD, NH_3-N and D.O. levels. If, however, the modeling is unsuccessful, the calibration must be repeated in a manner as suggested below: a) If NH_3-N has been oversimulated, the modeler must increase the value of K_N or increase the preferential algal uptake factor (NP) and vice versa for undersimulation. b) If D.O. has been undersimulated, the value of K_N should be reduced or the factors from steps 3 and 4 adjusted and vice versa for oversimulation.

The third step of the modeling procedure involves a <u>check</u> on the calibration performed above, using data from a different sampling event. The verification event must involve either different flow rates, temperature, and/or wastewater load conditions to assess whether or not the calibration step was adequately performed. JRB Associates point out that some of the previously modeled parameters may change, especially if the sampling events occurred in different seasons (JRB, 1983a). While these modeled parameters primarily include those physical and biological characteristics, such as percent ice cover, temperature, and chlorophyll-a concentrations, that would be abstracted from the sampling data, it could include factors such as NP, AP, or DP. JRB Associates goes on to state that under no circumstance should the values of K_1, K_2 or K_N be adjusted (JRB, 1983a).

While the JRB report does not specifically state what would happen if the verification step proved the previous calibration in error, one could assume that a compromise would be reached regarding the coefficients arrived at during the calibration and verification steps. Turkle (Department of Water, Air, and Waste Management, Des Moines, Iowa, personal communication, 1984) however, stated that the State typically looks at the WLA developed from each step to determine if any differences exist in the final allocation. If no changes exist (or if they are minor) the State assumes the WLA to be valid.

The fourth step involves the establishment of the WLA itself. In this step, the dry weather discharge from the year 2000 is impacted upon the stream, with a selected effluent quality. (Prior to 1984 this was a wet weather discharge.) This waste effluent quality is varied until minimal water quality conditions for the designated stream are maintained. The modeling is done for both summer and winter low flow conditions using the coefficients developed from the calibration and verification steps above. Temperature and percent ice cover conditions are assumed in accordance with the procedures set forth above. The most stringent effluent concentrations from either the summer or winter season, establish a <u>single</u> year effluent discharge limitation.

Proposed changes to the WLA procedure in 1984 have also included provisions for flow-variable ammonia limitations and detailed mixing zone calculations (DWAWM, 1984). The flow-variable ammonia limitations would allow for greater ammonia discharges during periods of flow in excess of the 7Q10. A discussion of the mixing zone calculations will not be attempted in this thesis.

EVALUATION OF THE QUAL-II WATER QUALITY MODEL

Introduction

The use of Qual II for modeling Iowa's streams and rivers was first proposed by JRB Associates in 1982, after their review of the State's modeling procedure (JRB, 1982). The intent of introducing Qual II was to allow more accurate stream simulations to occur, hence, more appropriate WLAs. JRB Associates introduced a version of Qual II that was adapted from the State of Vermont. This Vermont version of Qual II will briefly be summarized in the following paragraphs. Because of the numerous and complex routines and subroutines available in Qual II, a complete review of this material will not be possible.

The Qual models were originally developed by F.D. Masch and Associates, and the Texas Water Development Board in 1971 (Roesner et al., 1981). As noted earlier, several revisions to that early model have been made throughout the years to incorporate additional parameters and parameter interactions.

The parameters capable of being simulated by the Vermont version of Qual II include the following:

1. Dissolved Oxygen (D.O.)
2. Biochemical Oxygen Demand (BOD)
3. Temperature
4. Algae as Chlorophyll-a
5. Organic Nitrogen
6. Ammonia
7. Nitrite
8. Nitrate
9. Dissolved Phosphorous
10. Organic Phosphorous
11. Coliforms, and
12. Conservative Substances.

Most Qual II programs have these capabilities with the exception of organic nitrogen and organic phosphorous. Other modifications in the Vermont version include the following features abstracted from JRB Associates Qual II User Manual (JRB, 1983b):

1. Provision for algal uptake of ammonia,
2. Steady-state calculation of D.O. diurnal variation with dynamic simulation deleted,
3. Inclusion of dam reaeration,
4. Alternate methods available for reaeration rate constant calculation, and
5. Deletion of radionuclide simulation.

The Vermont version of Qual II allows dendritic stream systems to be modeled with the following limiting assumptions:

1. Stream is well-mixed and the major transport mechanisms of advection and dispersion are important only in the longitudinal direction,
2. Input loads and inflows are constant over time, but may originate from multiple point or distributed sources (steady-state), and
3. Stream may be divided into many segments where all processes are conceptualized as a series of completely mixed reactors.

General Model Relationships

The Qual II program is structured around a main program which allows different subroutines to be called upon as required. This essentially allows a modeler to add new parameters to the model without major modifications (Roesner et al., 1981).

The Vermont version of Qual II simulates the major interactions of nutrient cycles, algal production, benthic activity, oxygen demanding material, reaeration, and the effects of these on dissolved oxygen concentrations with time. Some important features of the subroutines will now be briefly examined, as adapted from Roesner et al. (1981).

Stream velocity ,depth, and width are calculated in Qual II using empirical formulae equivalent to the previously seen Leopold-Maddock equation for velocity. Input options exist for the empirical coefficients and range from complete entry to internal calculation with raw data input.

Algal kinetics employ the familiar Michaelis-Menton growth limiting equations. Limitation occurs with light intensity and the minimum value for phosphorous or combined ammonia and nitrate.

Ammonia concentrations change in the stream due to nitrogen cycle effects. The Vermont version of Qual II allows ammonia increases, as a result of organic nitrogen hydrolysis and benthic sources, and ammonia decreases, as a result of nitrification and algal uptake.

Carbonaceous BOD deoxygenation assumes first-order decay rates with inclusion of bed activity and instream settling.

Reaeration rates can be computed using any of 7 options available. The Vemont version of Qual II does not use the Tsivoglou expression suggested for use by JRB Associates (JRB, 1983b). Consequently, it was recommended that it be computed outside the model and input directly as a constant value. Ice cover must also be applied in this manner as the Vermont version of Qual II does not have this capability (JRB, 1983b).

The original EPA version had a quasi-dynamic simulation approach for D.O. simulation (Roesner _et al_., 1981). This capability was deleted in the Vermont version of Qual II and replaced with a diurnal curve analysis, which can predict daily minimum or maximum D.O. values (JRB, 1983b).

This brief overview was intended to introduce Qual II only as a water quality tool. It is generally considered to be the "state-of-the-art" in water quality modeling (DWAWM, 1984). Due to its sophisticated and complex equations, a sensitivity analysis was not attempted.

RESULTS

Sensitivity Analysis

General

The primary goal of a sensitivity analysis is to demonstrate how changes in model input may affect model output. Analysis of these input-output relationships can lead to numerous secondary goals which include the following:

1. Indication of the relative significance of all input parameters,

2. Establishment of a data acquisition program which can concentrate sampling efforts on those parameters that have the greatest impact on the model output,

3. Establishment of standard sampling practices and allowable error measurements for those parameters which are obtained from the field,

4. Indication of model weaknesses and equation limitations, which otherwise wouldn't be apparent, and

5. Indication of the complex interrelationships which can occur, even in simple mathematical models.

A sensitivity analysis must assume that the mathematical model itself is able to simulate water quality conditions within acceptable limits, as a sensitivity analysis can not provide a direct means for assessing the model's reliability. A calibration-verification procedure could help in predicting reliability. Also, it should assume that the input data values are subject to statistical variations since they are a part of a larger population.

TenEch (1978a) performed a sensitivity analysis on the original "Stanley" model and evaluated the parameters listed below.

1. Velocity,
    a) Channel slope,
    b) Water surface width,
    c) Manning's roughness coefficient,
    d) Stream discharge,
    e) Leopold-Maddock coefficients,
2. Stream temperature,
3. Reaeration rate constant and equation,
4. Carbonaceous deoxygenation rate constant, and
5. Nitrogenous deoxygenation rate constant.

This thesis will summarize the important findings of the TenEch study using their graphs when required. A further analysis on the original "Stanley" model and the modified JRB model will be presented to help demonstrate other important input-output relationshps, which were not covered in the TenEch report.

TenEch analysis

TenEch (1978a) recognized that stream velocity was particularly important and as a result, they analyzed the major input parameters to both the Manning and Leopold-Maddock velocity formulae. Use of the Manning equation requires the initial input of the Manning's roughness coefficient, stream discharge, water surface width, and channel slope. The Leopold-Maddock equation requires the initial input of two empirically derived coefficients.

Analysis of the Manning approach determined that the most critical input parameter to be the roughness coefficient. Figure 10, shows the effect that the roughness coefficient has on the computed velocity value with a constant channel slope, water surface width, and stream discharge.

This figure clearly shows the following:

1) The value of the roughness coefficient becomes more significant to velocity values as discharge increases and

2) The value of the roughness coefficient becomes more significant to velocity values as the roughness coefficient decreases in value.

Figure 11 shows the effect that the water surface width has on the computed velocity and indicates the following:

1) Widths become less significant as widths increase beyond that which produces a peak velocity,

2) Widths less than that which produce a peak velocity have a major impact on velocity values, and

3) Width has a greater impact on velocity as discharge increases.

Figure 12 shows the effect that the channel slope has on computed velocity value indicating that:

1) Channel slope becomes more significant to velocity values as discharge increases and

2) Channel slope becomes less significant as channel slope increases.

Analysis of the Leopold-Maddock approach determined that the values of the empirically derived coefficients become more significant to velocity as the coefficients themselves become larger. Hence, it becomes

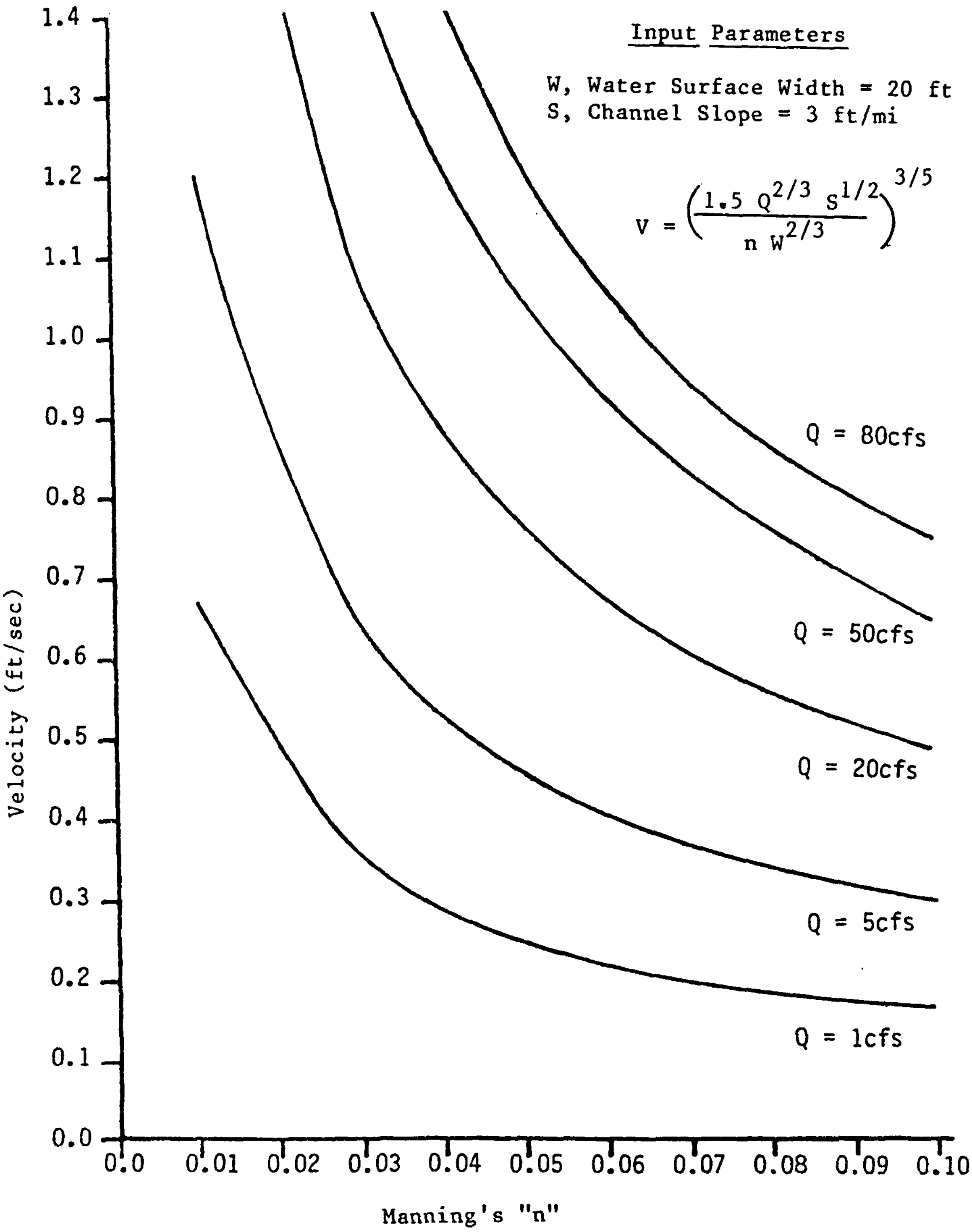

Figure 10. Velocity relationships for various discharge and Manning's "n" values (TenEch, 1978a)

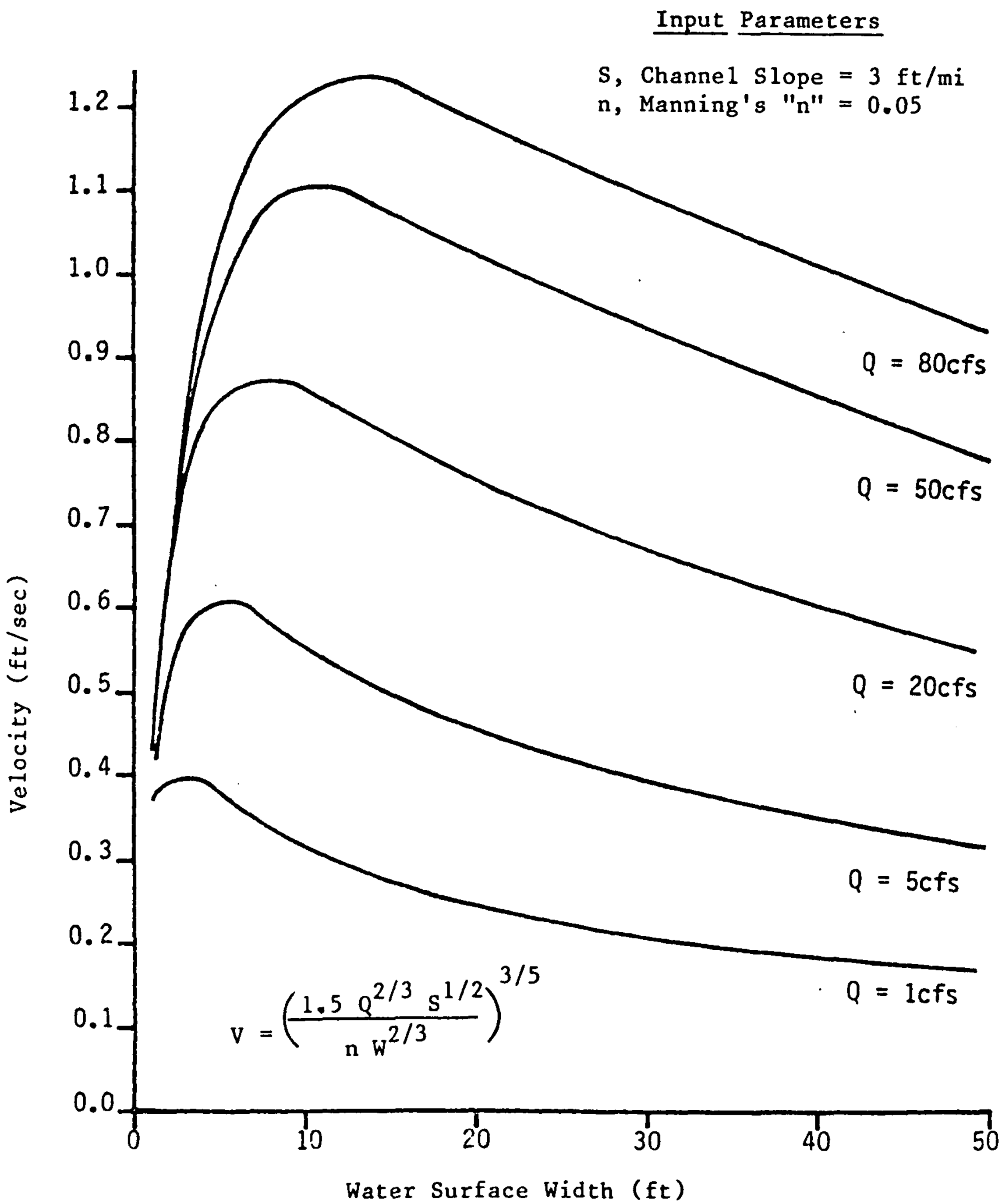

Figure 11. Velocity relationships for various discharge and water surface widths (TenEch 1978a)

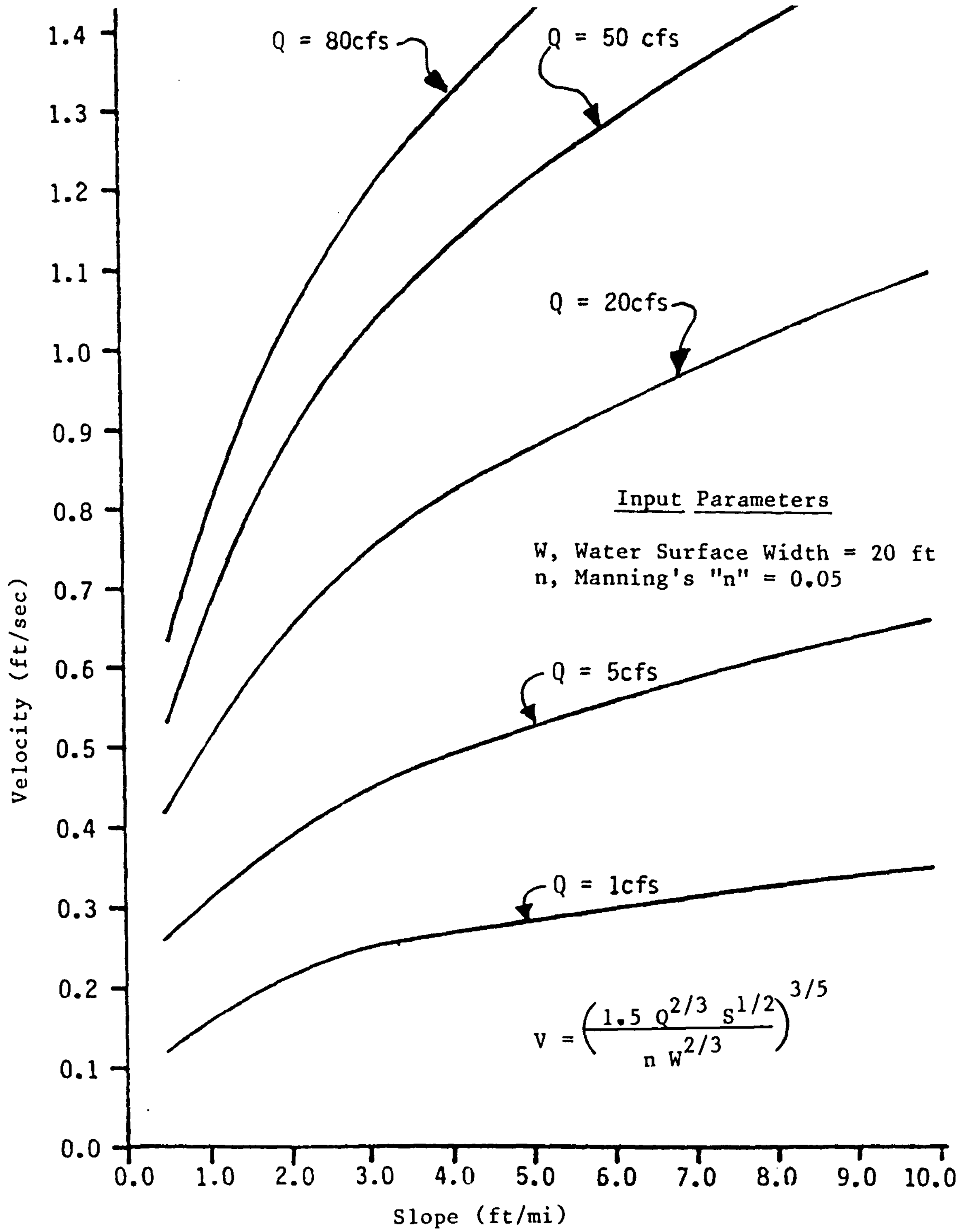

Figure 12. Velocity relationships for various discharge and slope values (TenEch 1978a)

more important to accurately obtain these coefficients when they are large, as opposed to small.

The effects of stream temperature have an impact on all of the reaction rate constants used in the original "Stanley" model. While essentially constant temperatures exist during the Waste Load Allocation (WLA) process, the sensitivity of temperature analysis has a significant importance in model calibration and verification.

TenEch's (1978a) study found the following temperature relationships:

1. Temperature becomes more significant as each reaction rate increases (K_1, K_2, and K_N),

2. Increasing temperatures become more significant for reaction rates as the value of θ increases. Hence, in the original "Stanley" model this applies only to K_1 and K_2, and

3. Temperature produces a linear trend in the K_N reaction rate above 3°C. (This changes in modified JRB analysis.)

The reaeration rate is determined using the Tsivoglou formula as described earlier and shown below.

$$K_2 = C \Delta h/t$$

where all terms have been previously defined. Manipulation of the equation results in the following:

$$K_2 = C\ S\ V$$

where

S = Channel slope (ft/ft) and

V = Velocity (ft/sec).

As shown, the reaeration rate (K_2) is directly proportional to

C, S, and V. However, the interrelationships of input parameters become apparent when considering the determination of V by the Manning equation, which also uses channel slope (S).

Figure 13 shows the typical effect that slope (S) and the Tsivoglou gas escape coefficient (C), have on K_2. The values of velocity were determined with constant values for stream discharge, water surface width, and Manning's roughness coefficient.

Figure 13 indicates the following:

1) Changes in channel slope become more significant as channel slope increases and

2) Changes in channel slope become more significant as the Tsivoglou gas escape coefficient increases.

Further analysis by TenEch at larger values of stream discharge showed that changes in channel slope become more significant at higher values of stream discharge.

Further analysis

To demonstrate the effects of varying the rate constants K_1, K_2, and K_N, TenEch set up an example discharge situation and plotted minimum D.O. values versus a varying K_1 (or K_n) with a constant K_n (or K_1). An example of this type of plot can be found in Figure 14. Plotting of minimum D.O. values is a concise way to graphically portray a great deal of information, but it does not give the modeler a feel for what is happening to the entire D.O. sag curve. This knowledge is useful in D.O. "curve fitting", which <u>must</u> occur in calibration and verification steps. Consequently, another approach was

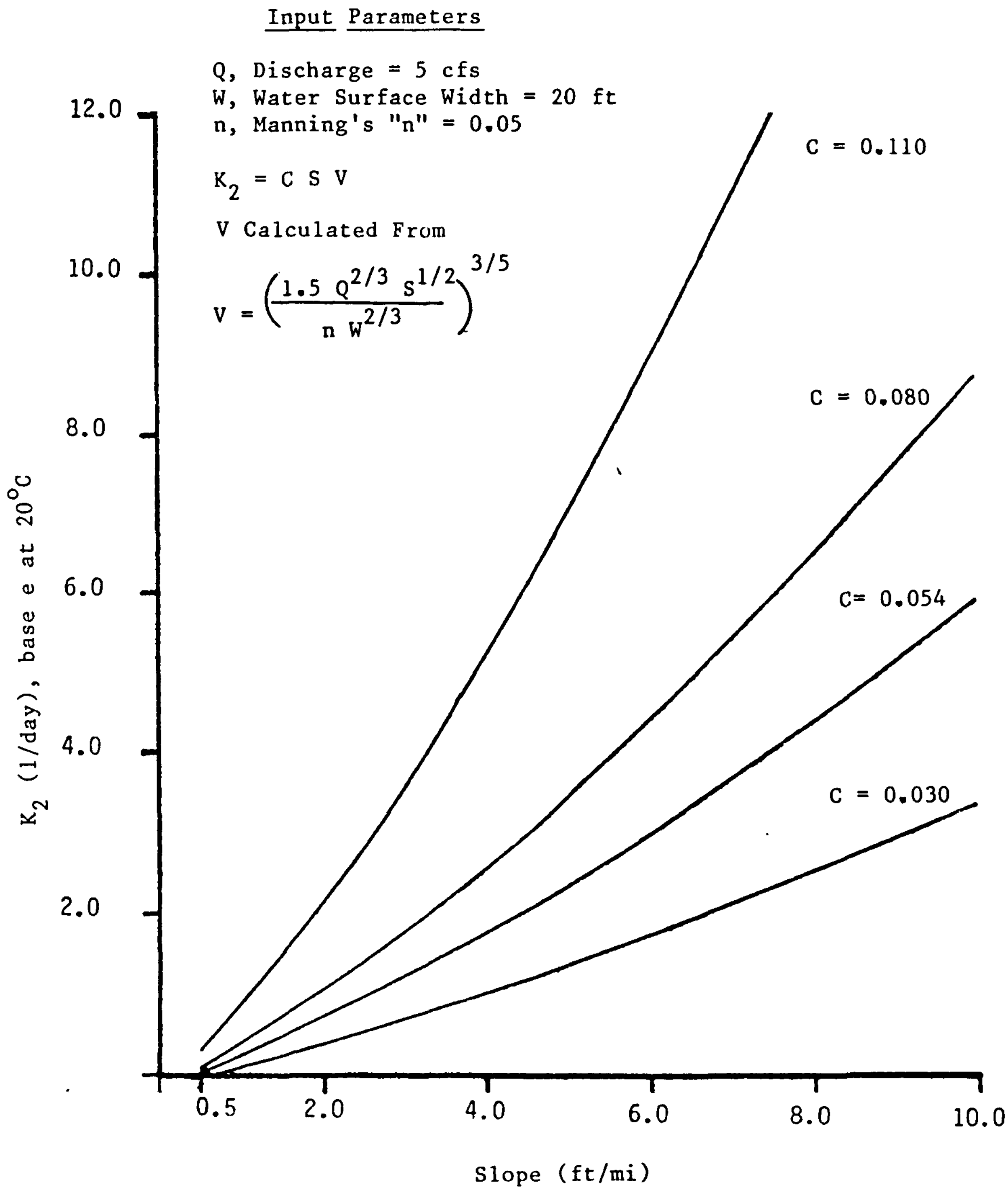

Figure 13. Reaeration rate constant relationships for various slope and Tsivoglou's "C" values (TenEch 1978a)

Input Parameters

Q, Discharge = 2 cfs
Ultimate CBOD = 45 mg/l
Ammonia as N = 15 mg/l
Initial D.O. Concentrations = 5 mg/l

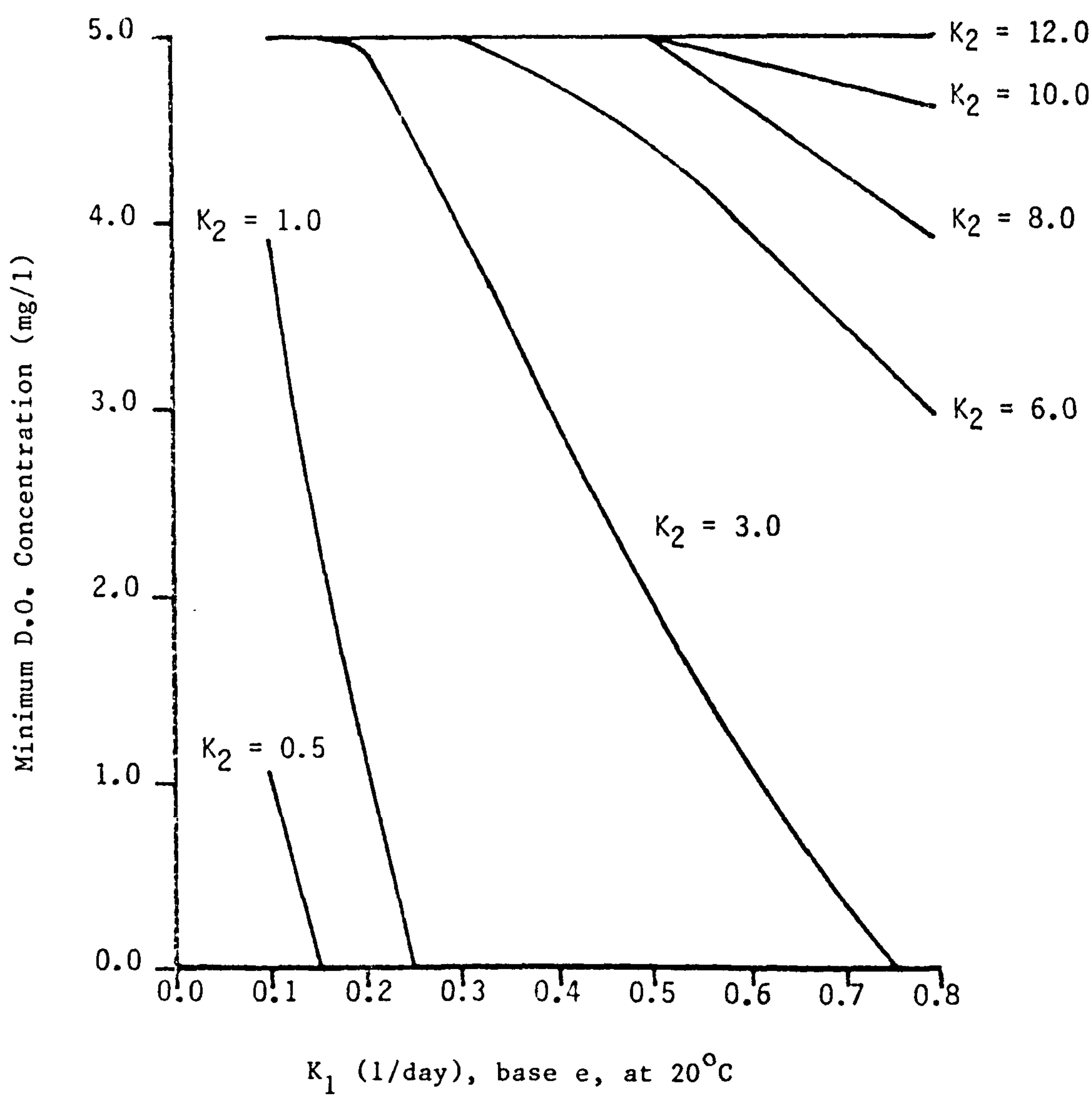

Figure 14. Minimum D.O. relationships for various reaeration and carbonaceous deoxygenation rate constant values (TenEch 1978a)

taken which could benefit future "curve fitting" exercises.

To further analyze the original "Stanley" model with "curve fitting" in mind, a computer program was written to aid in the computations. The program's source code may be found in Appendix E. The program itself was written in BASIC and allowed changes in default input parameter values, through an interactive mode of operaton. Continuous looping, with prompting, allowed the modeler to stay within the program after any number of successive changes, thus permitting numerous runs to be made without leaving the program. The program format also allowed the modeler to immediately see results after making input changes.

The following six graphs show the effect on the D.O. deficit with each of the following conditions:

1) Varying waste loads,

2) Varying each rate constant independently (i.e., K_1, K_2, and K_N), and

3) Varying the initial D.O. deficit.

Plotting of D.O. deficits has some advantage over plotting D.O. concentrations since they are temperature independent.

Figure 15 shows the effect that waste load variations can have on D.O. deficits. The results of this first analysis are <u>not</u> very surprising, but provide an ideal starting point. Figure 15 clearly shows the following:

1. Critical D.O. levels drop with higher waste loadings,

2. Critical deficits occur at the same location downstream, regardless of the waste loading,

3. A decline in L_o or N_o results in a proportionate decline

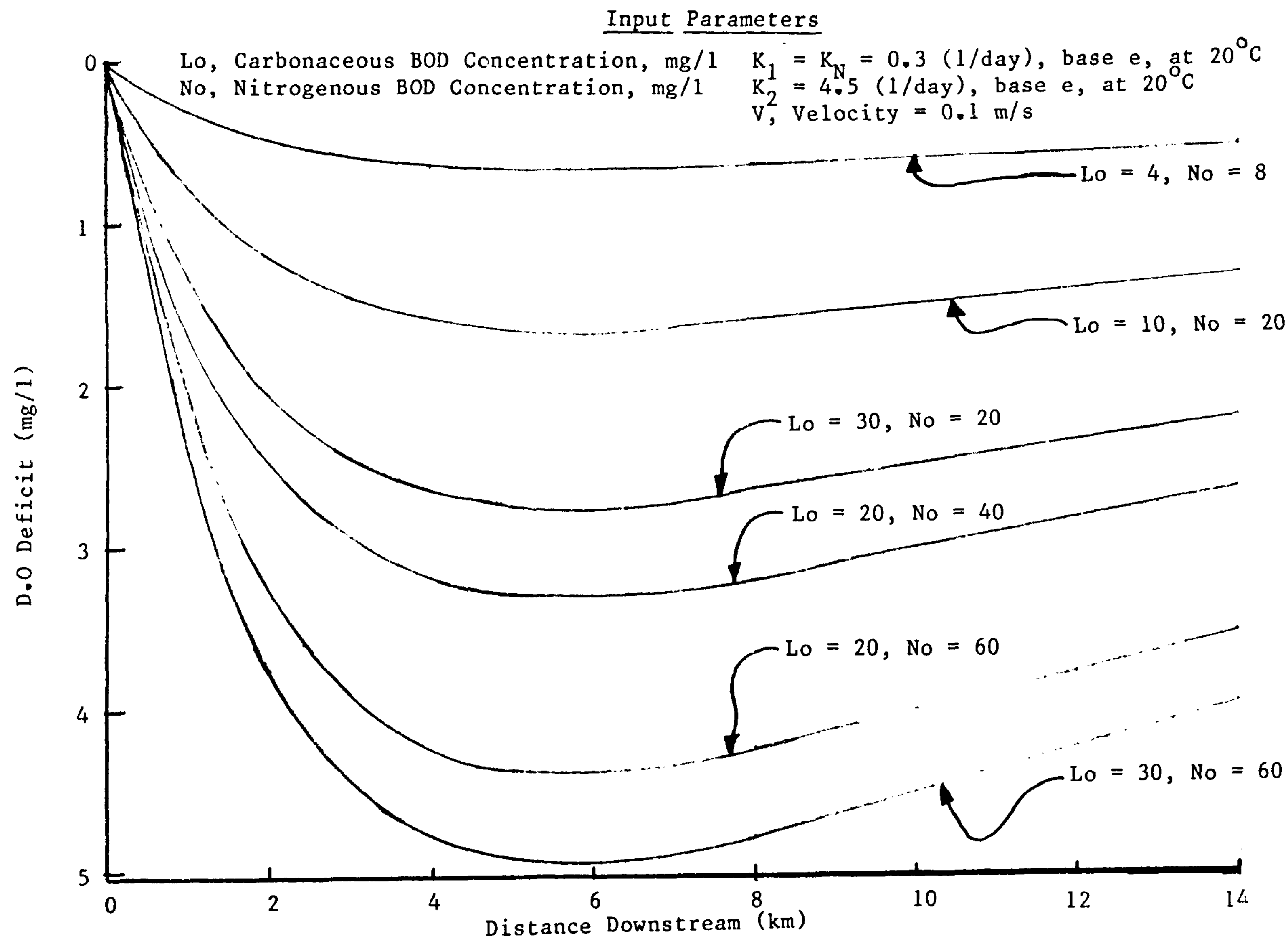

Figure 15. Effect on D.O. with varying waste loads

in D.O. levels. For the data shown a 10 mg/l change in L_o or N_o resulted in about a 0.55 mg/l D.O. change, as either L_o or N_o produced the same D.O. change since, for this computer run K_1 = K_N, and

4. The upswing line, which characterizes reaeration after the minimum D.O. level occurrence, becomes progressively steeper with higher waste loadings.

Figure 16 shows the effects of varying the carbonaceous deoxygenation rate (K_1) on D.O. deficits. The figure indicates the following:

1. Critical D.O. levels drop with higher values of K_1, but at a substantially declining rate, indicating that the impact of K_1 on the minimum D.O. level decreases as K_1 increases,

2. Critical deficit location occurs further upstream (closer to waste load source) as K_1 increases, and

3. The upswing line becomes steeper as K_1 increases and asymptotically approaches a line, in the downstream reach, characterized by $K_1 = 0$.

The effects of varying the nitrogenous deoxygenation rate (K_N) on D.O. deficits is shown in Figure 17. The figure is nearly identical to Figure 16, resulting in identical conclusions as well. Figure 17 shows the following:

1. Critical D.O. levels drop with higher values of K_N, but at a substantially declining rate, indicating that the impact of K_N on the minimum D.O. level decreases as K_N increases,

2. Critical deficit location occurs further upstream as K_N

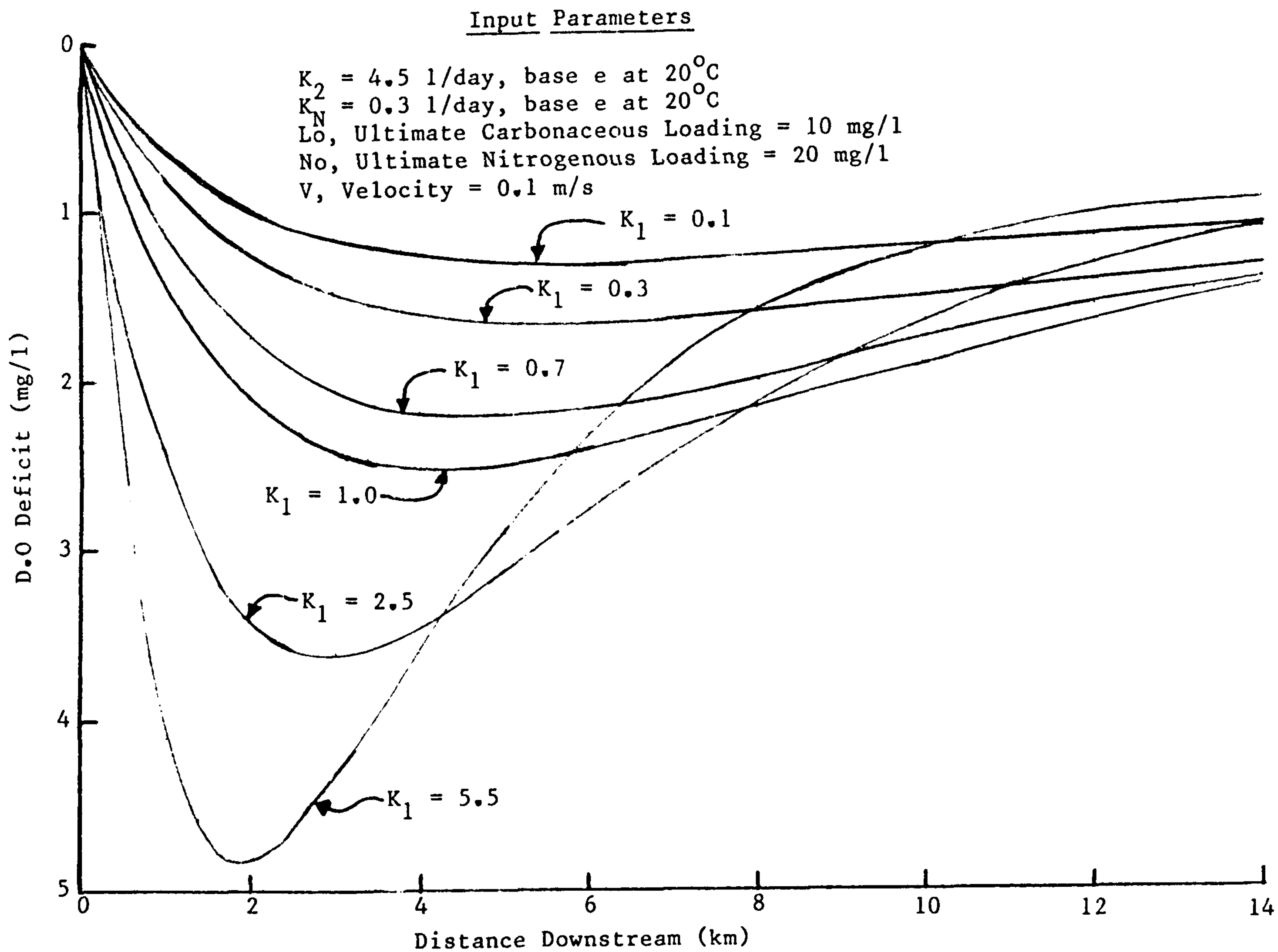

Figure 16. Effect on D.O. with varying carbonaceous deoxygenation rate constants

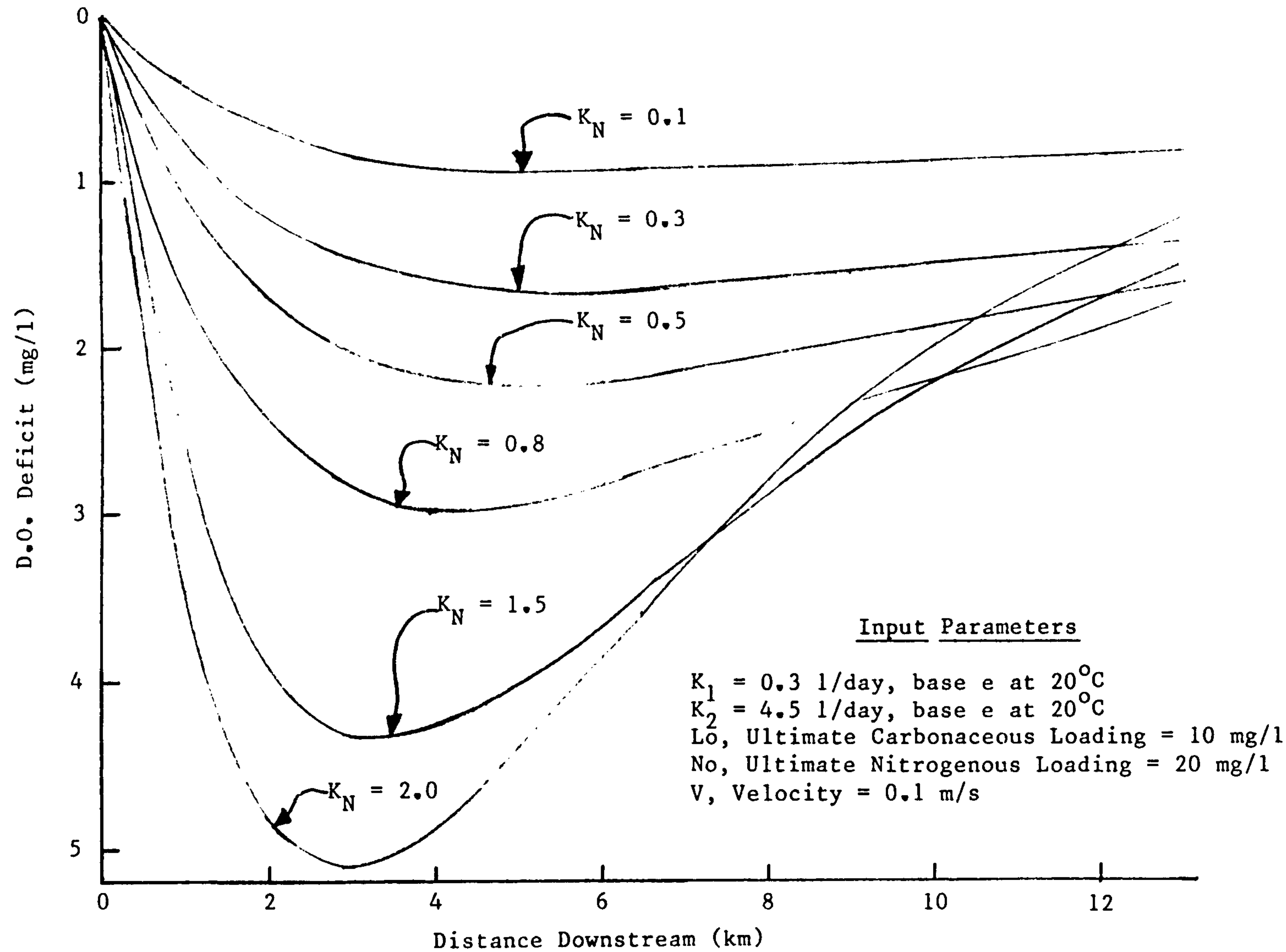

Figure 17. Effect on D.O. with varying nitrogenous deoxygenation rate constants

increases, and

3. The upswing line becomes steeper and asymptotically approaches a line characterized by $K_N = 0$.

The effect of varying the reaeration rate (K_2) on D.O. deficits is shown in Figure 18. Results of the analysis indicate that:

1. Critical D.O. levels drop with lower K_2 values,

2. Critical deficit location moves upstream with higher K_2 values,

3. The impact of K_2 on the minimum D.O. level decreases as the value of K_2 increases, and

4. Upswing lines are not substantially affected by the reaeration rate and actually become slightly flatter at higher values of K_2.

Figures 19 and 20 show the effects of varying the initial D.O. deficit value, from the assumed saturated value at 0 mg/l deficit. Figure 19 presents a situation with a small initial oxygen sag, whereas Figure 20 portrays a much larger sag condition. Results of the analysis indicate that:

1. Critical deficit location moves rapidly upstream with lower initial D.O. deficits and

2. Deoxygenating effect of L_o and N_o material is not additive to the initial D.O. deficit, but is actually reduced as the initial D.O. deficit increases, until a point where the initial D.O. deficit is the critical D.O. location and reaeration alone governs.

The analysis by TenEch (1978a) allowed the following two conclusions regarding reaeration to be drawn from their analysis. These include the following:

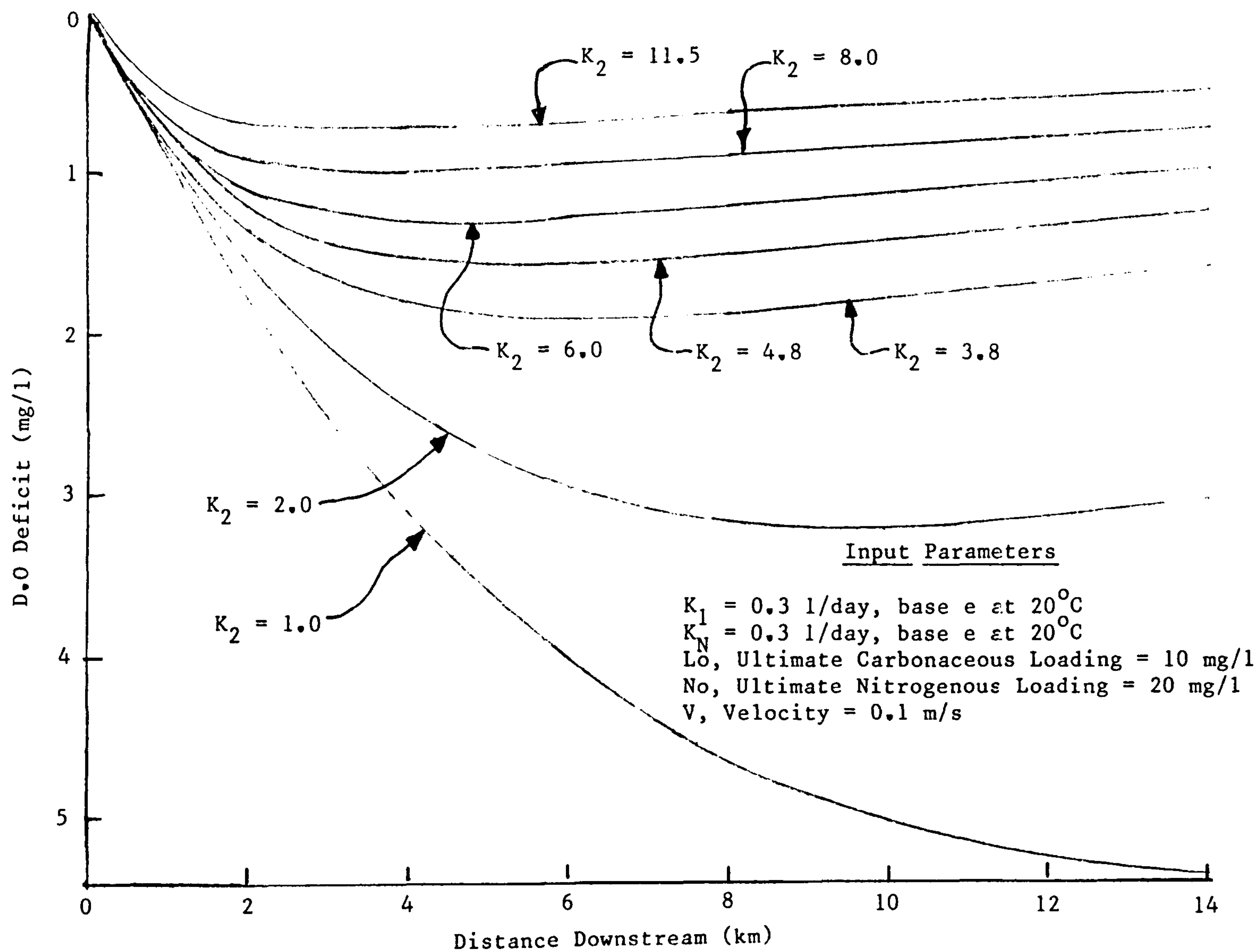

Figure 18. Effect on D.O. with varying reaeration rate constants

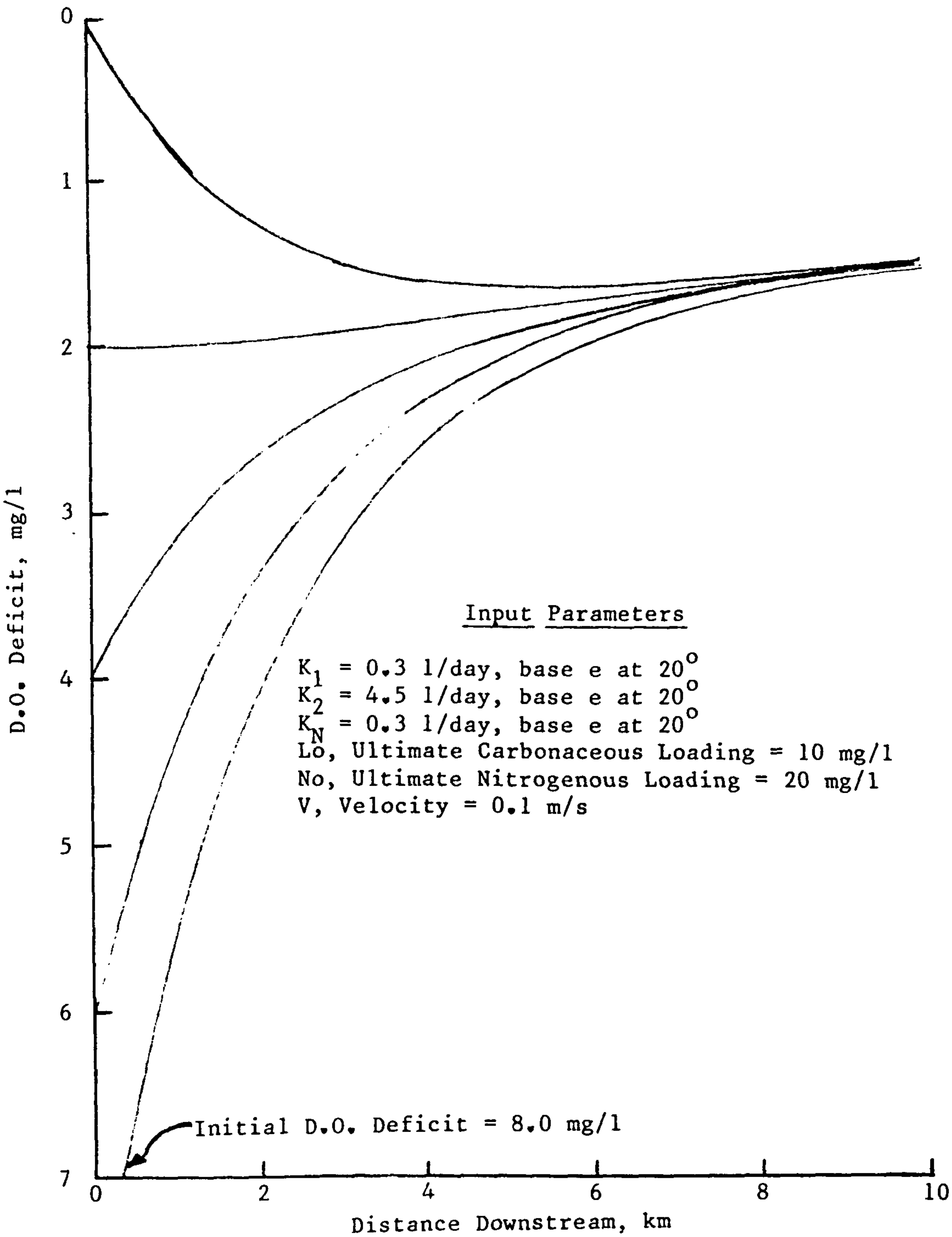

Figure 19. Effect on D.O. with varying initial D.O. deficits and low initial deoxygenation

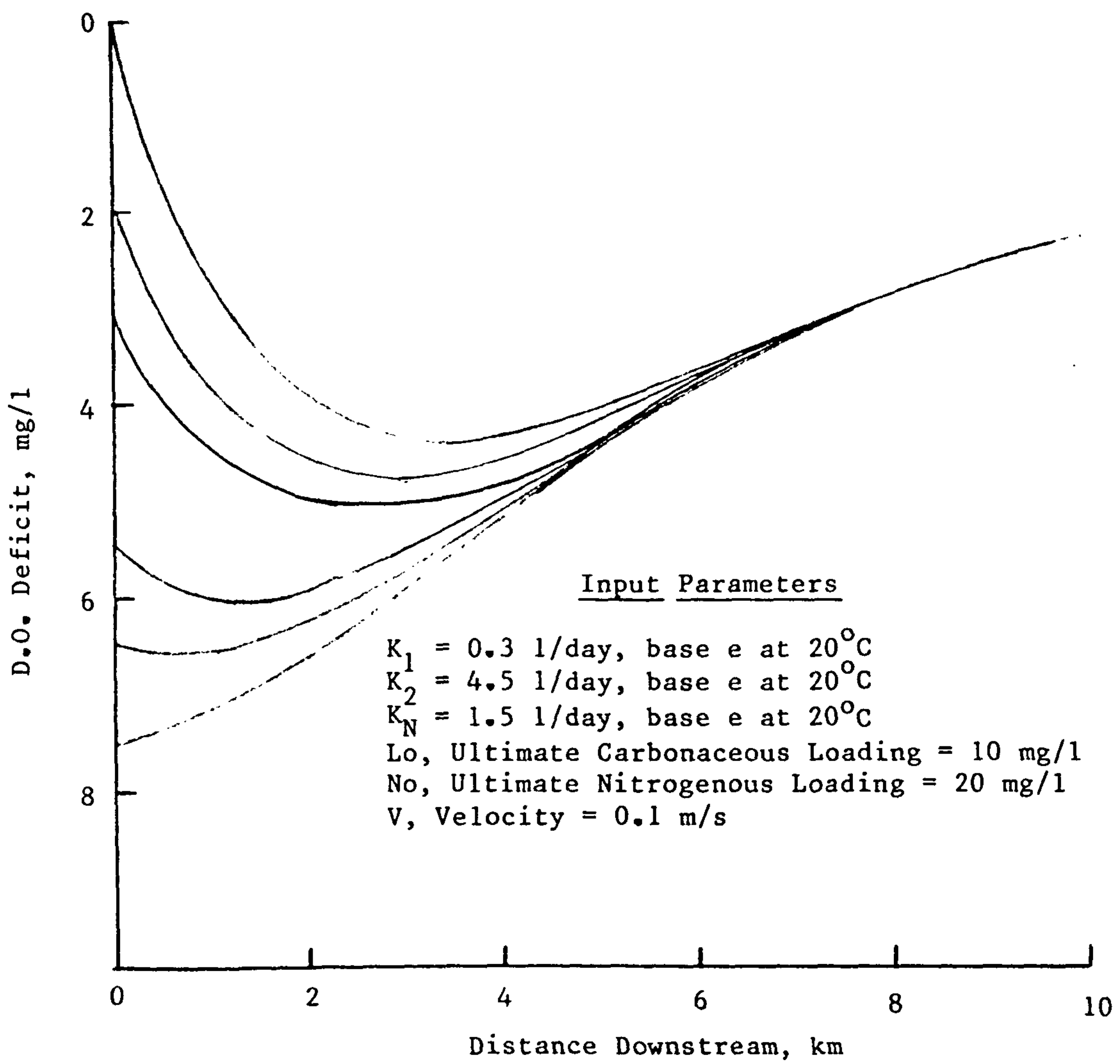

Figure 20. Effect on D.O. with varying initial D.O. deficits and high initial deoxygenation

1. As K_2 increases the impact of K_1 on the dissolved oxygen level decreases and

2. As K_2 increases, the impact of K_N on the dissolved oxygen level decreases.

The reaeration constant (K_2) is reduced proportionally by the percentage of ice cover as described earlier and applies to both the original "Stanley" and modified JRB model. While the effects of reducing the reaeration on the D.O. level have already been shown, the sensitivity of this reduction in comparison to the actual percentage of ice cover has not. Figure 21 compares the ratio of K_2 (without ice) divided by K_2 (with ice) versus the percentage of ice cover. Expressed as an equation, the ratio can be simply described as follows:

$$\text{Ice Factor Ratio} = \frac{K_2(\text{without ice})}{K_2(\text{with ice})} = \frac{1}{(1\text{-\% ice cover})}$$

This appears as the solid line in Figure 21. The dashed line represents the same ratio versus the percentage of ice cover, but using a slightly different formula for expressing the K_2 reduction. This slightly different formula was discussed earlier and came from the initial draft of the "Stanley" model (IDEQ, 1979). The dashed line can be expressed by the following equation:

$$\text{Ice Factor Ratio} = \frac{K_2(\text{without ice})}{K_2(\text{with ice})} = \frac{1}{1\text{-(\% ice cover-5\%)}}$$

Figure 21 clearly shows the great impact that ice cover has on the reaeration rate when the percentage exceeds about 80%. Also, the impact of the slight formula change is readily apparent above the 80% mark.

In performing a sensitivity analysis on the modified version of the modified JRB model, a more simplistic approach was taken to avoid duplicating the analysis completed on the identical terms in the original "Stanley" model. Consequently, only the new or changed terms will be commented on. The three major changes to the original "Stanley" model included the following:

1. Improvements to the adjustment of K_N for temperature and low dissolved oxygen and K_2 for temperature only,

2. Introduction of a series of equations to allow for the uptake of NH_3-N by phytoplanktonic algae, and

3. Addition of a photosynthesis minus respiration term in determining D.O. deficits.

The results obtained earlier in the TenEch, (1978a) study apply to the temperature correction equations used in adjusting K_N and K_2. This occurs as all temperature adjustment equations are of the form shown below.

$$K_{T^oC} = K_{20^oC} \times \theta^{(T-20)^oC}$$

Since all θ's proposed for use in the JRB model are greater than one, increasing temperatures become more significant as the value of θ increases. The other conclusion drawn from the TenEch study showed that temperature becomes increasingly more significant as the reaction rates increase.

K_N is also adjusted for low dissolved oxygen levels in the

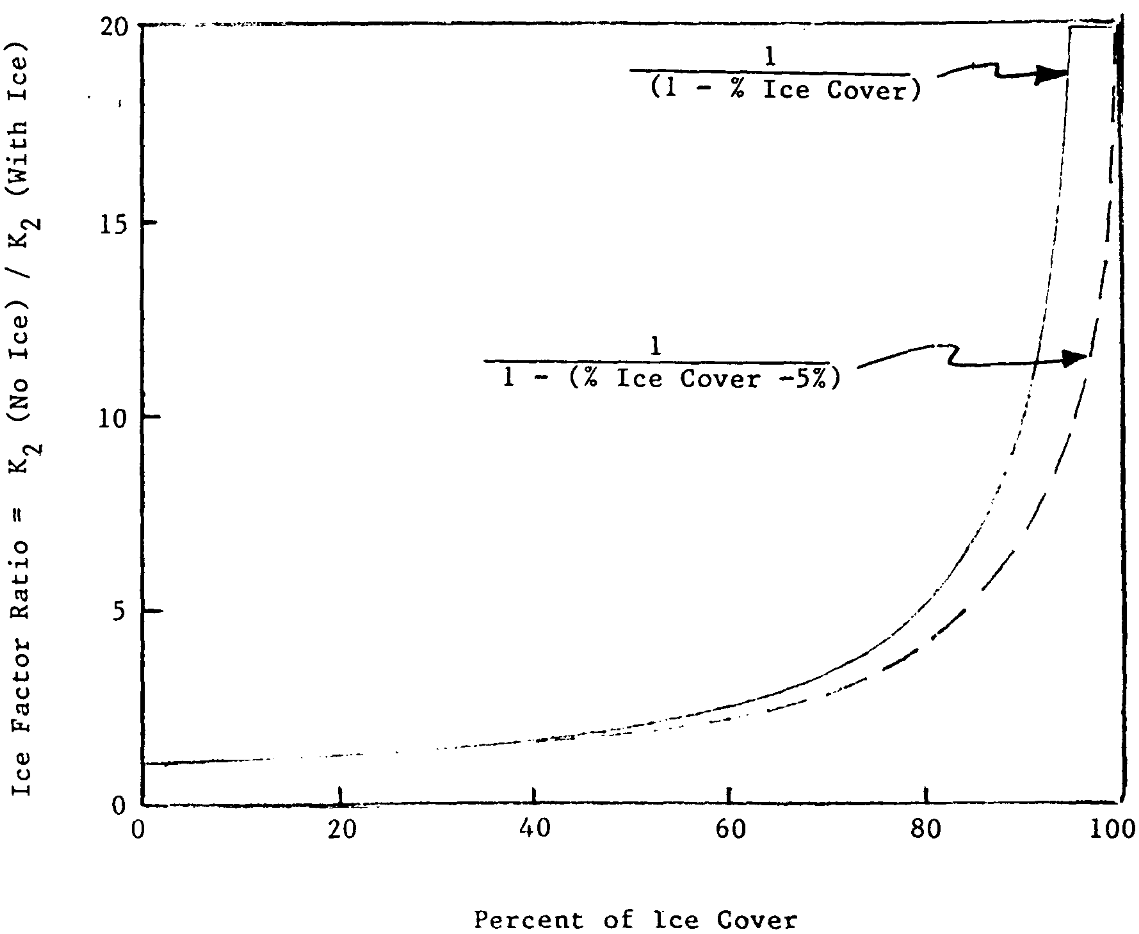

Figure 21. Effect of ice cover on the reaeration rate constant

modified JRB model. Figure 22 shows the reduction factor used to reduce K_N for a given D.O. level. The reduced K_N is found by simply multiplying the unreduced K_N rate constant by the reduction factor. The equation used in reducing K_N rates is very sensitive to D.O. changes under 3 mg/l, as it is intended.

Three equations were added to the modified JRB model in an attempt to model NH_3-N uptake by algae. The equations were presented in detail in an earlier section and will not be repeated here. The equations, however, include a calculation of local algal growth rates, which utilize Michaelis-Menton growth reduction terms, a temperature correction term for the local algal growth rate and finally, an equation to express the reduction of NH_3-N through algal uptake.

The equation for local algal growth rate (GP) consists of a maximum growth rate ($\bar{u}$) at 20^{o}C multiplied by three growth limiting terms of the general form $(S/(S+K_S))$ for combined nitrogen, phosphorous, and light intensity. The local growth rate term can be shown to have a major impact on the value for GP since it is directly proportional to GP. The impact of the growth reduction terms on the value of GP depends on the relative values for S and K_S. If K_S is small compared to S, the Michaelis-Menton terms approach one and have no influence on the value of GP. If, on the other hand, K_S is large compared to S, the Michaelis-Menton terms approach (S/K_S) and result in a drastic reduction in the value for GP.

GP is also adjusted for temperature changes. The adjustment equation used is of the form used for K_1, K_2, and K_N with θ greater than 1.0. Therefore, the same conclusions reached in that

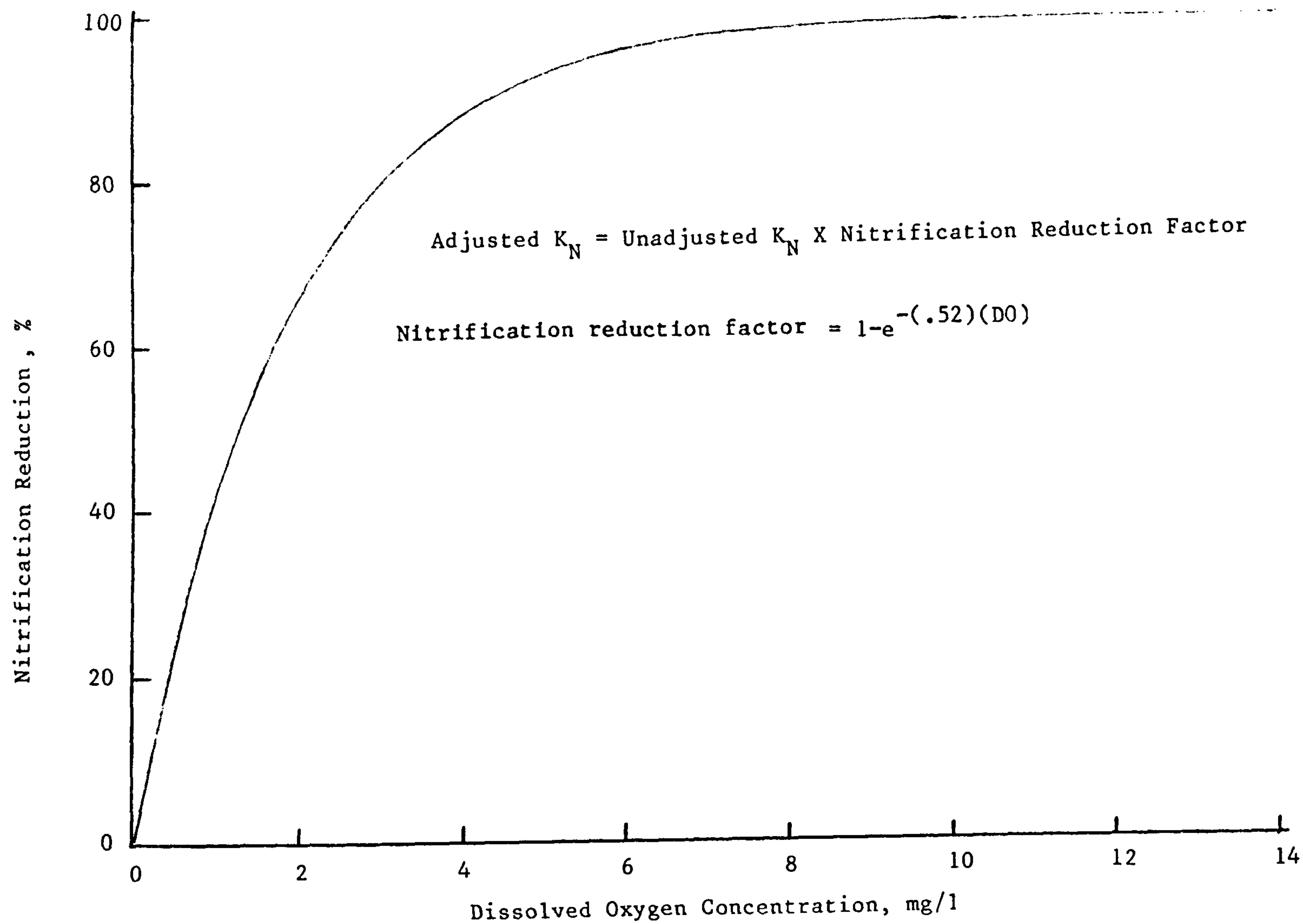

Figure 22. Nitrification reduction factor used in reducing the nitrogenous deoxygenation rate constant

analysis apply equally well here.

The algal uptake equation used in the modified JRB model was taken from another model referenced as MS-ECOL by Shindala et al. (as cited by JRB, 1983a). While discussed in more detail previously, the equation is shown below to facilitate discussion:

$$UP = \frac{(GP)(ANP)(NF)(CHLA)[e^{(GP-DP)} - e^{-K_N}]}{GP - DP + K_N}$$

where all terms have been previously defined.

The equation shows the terms ANP, NF, and CHLA to be directly proportional to the value UP. The terms GP, DP, K_N and t occur more than once in the equation and hence, their impact on the value of UP is more difficult to ascertain. The impact of the algal death rate, DP, on the calculated value of UP depends on the relative value of the local algal growth rate, GP. For small values of GP, DP will have a great impact on the calculated UP value. When DP is small compared to GP, its influence is negligible.

The impact of varying K_N on the value of UP is shown in Figure 23. For assumed and constant values of GP, ANP, NF, DP, and t; smaller K_N values have a greater impact on the value of UP.

The effects of increasing the values for (GP) and (t) have a tremendous impact on the value of UP due to the inclusion of the term in the numerator where e is raised to the (GP-DP)(t) power. Figure 24 shows the effects of increasing GP on the value of UP with all other values being constant. A similar result is shown in Figure 25 when (t) is varied.

$$UP = \frac{(GP)(ANP)(NF)(CHLA)[e^{(GP-DP)t} - e^{-K_N t}]}{(GP-DP+K_N)}$$

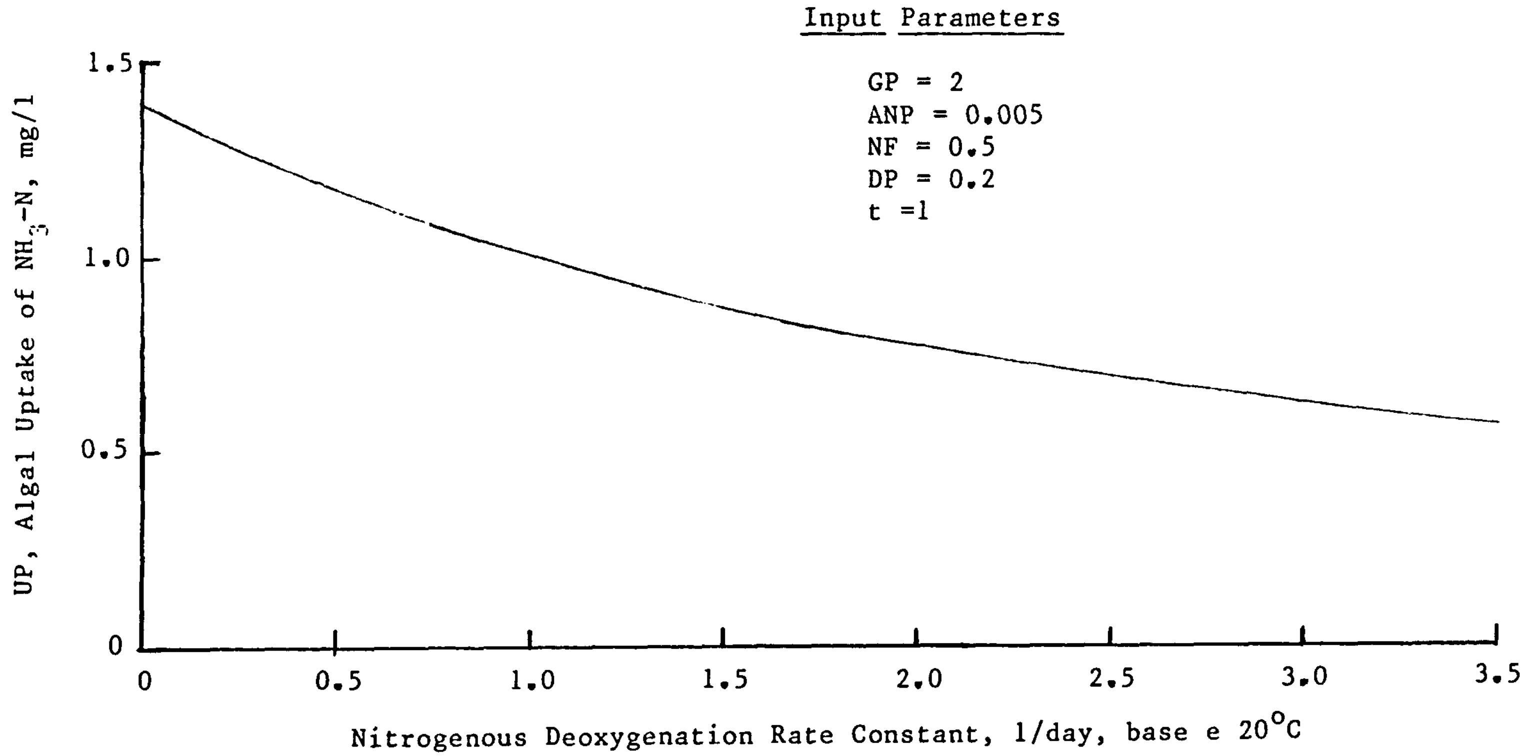

Figure 23. Relationship between algal uptake of ammonia and the nitrogenous deoxygenation rate constant

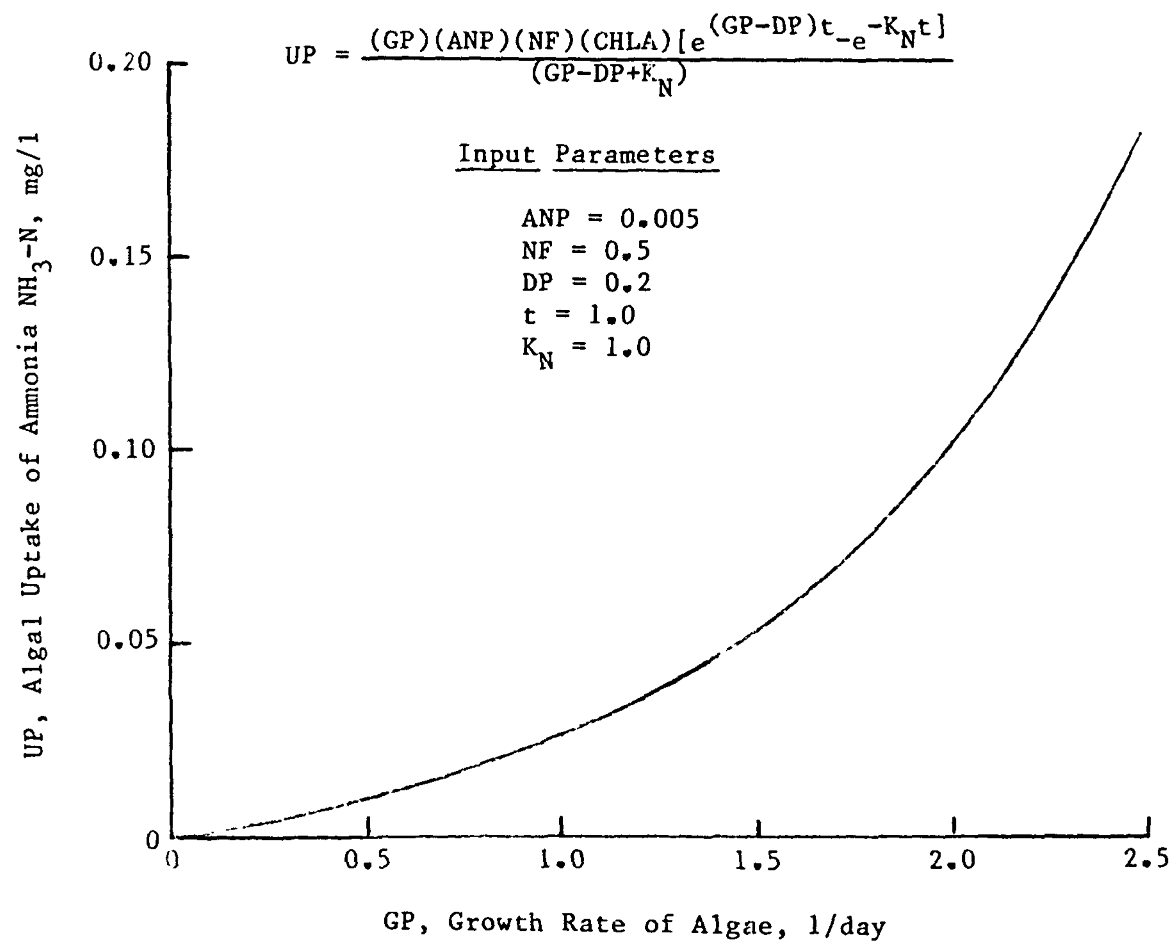

Figure 24. Relationship between algal uptake of ammonia and varying algal growth rates

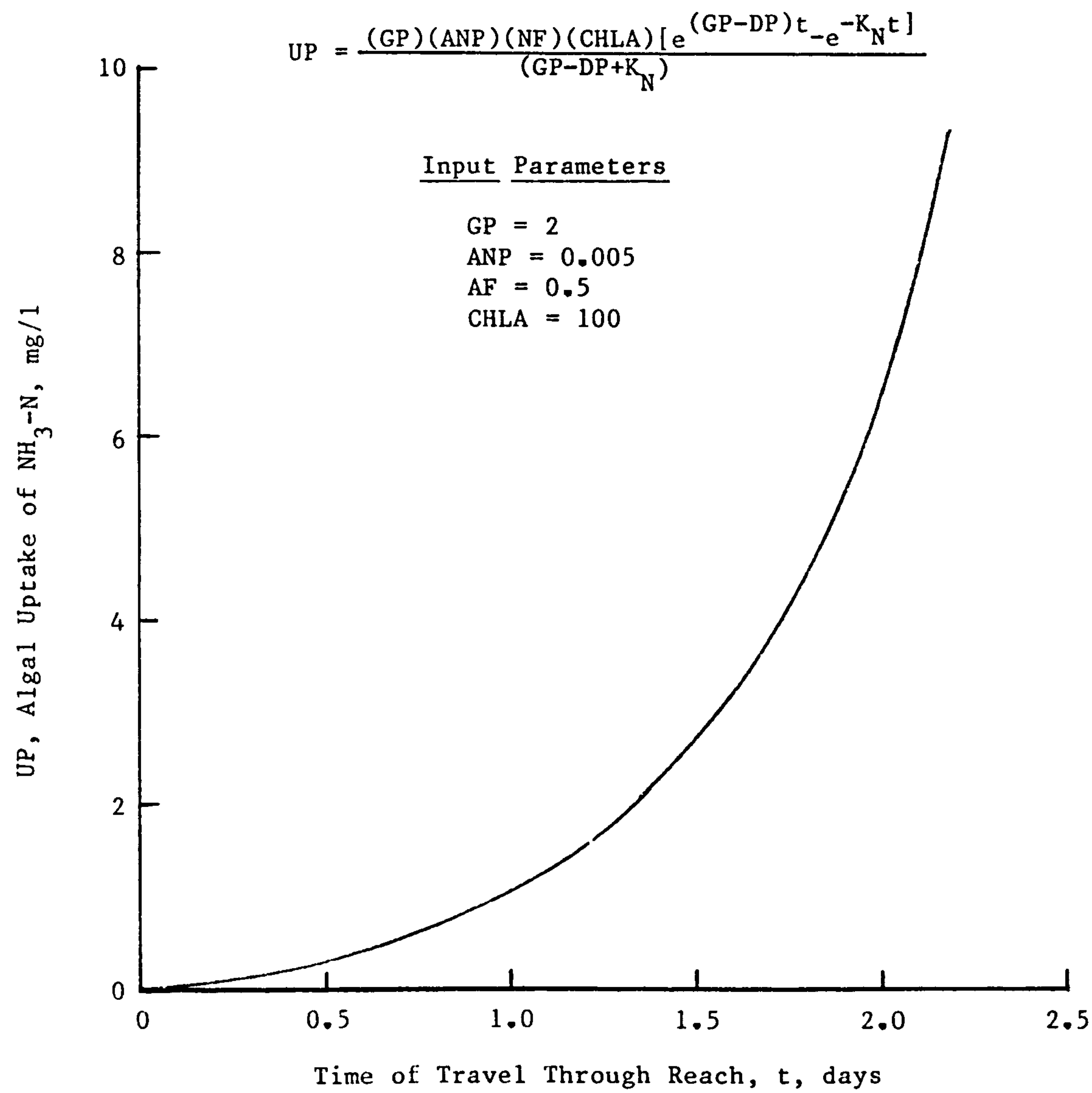

Figure 25. Relationship between algal uptake of ammonia and varying reach travel times

Three additional equations were added to the modified JRB model to predict D.O. changes associated with photosynthesis and respiration.

The simple equation used to predict respiration (R), is affected directly by the concentration of chlorophyll-a (CHLA) found in the stream. The photosynthesis (P) equation is only slightly more complicated. The value of P is directly proportional to the values assigned for OP, (GP-DP), and CHLA. P is also seen to be inversely proportional to the value of AP.

The influence of changing GP on the value of P is affected by the value assigned to DP. This occurs as smaller values of GP will be affected by DP much more than large values for GP.

The impact of a unit increase in the value for AP will have a greater effect on P when AP is smaller, then when it is larger. (This conversely applies to those directly proportional terms.)

The P and R equations are combined in the D.O. deficit formula taking on the form shown below:

$$\frac{(R-P)(1-e^{-K_2 t})}{K_2}$$

where all terms have been previously defined.

The R and P terms have been described above leaving only the (K_2) and (t) terms left. Figure 26 shows how the D.O. deficit can change with a constant (R-P) value and varying K_2 rate. As shown, the impact on D.O. deficits are large when K_2 is small.

The impact of changing (t) was not investigated by TenEch, since for first-order kinetics, a plot on logarithmic paper yields a straight line.

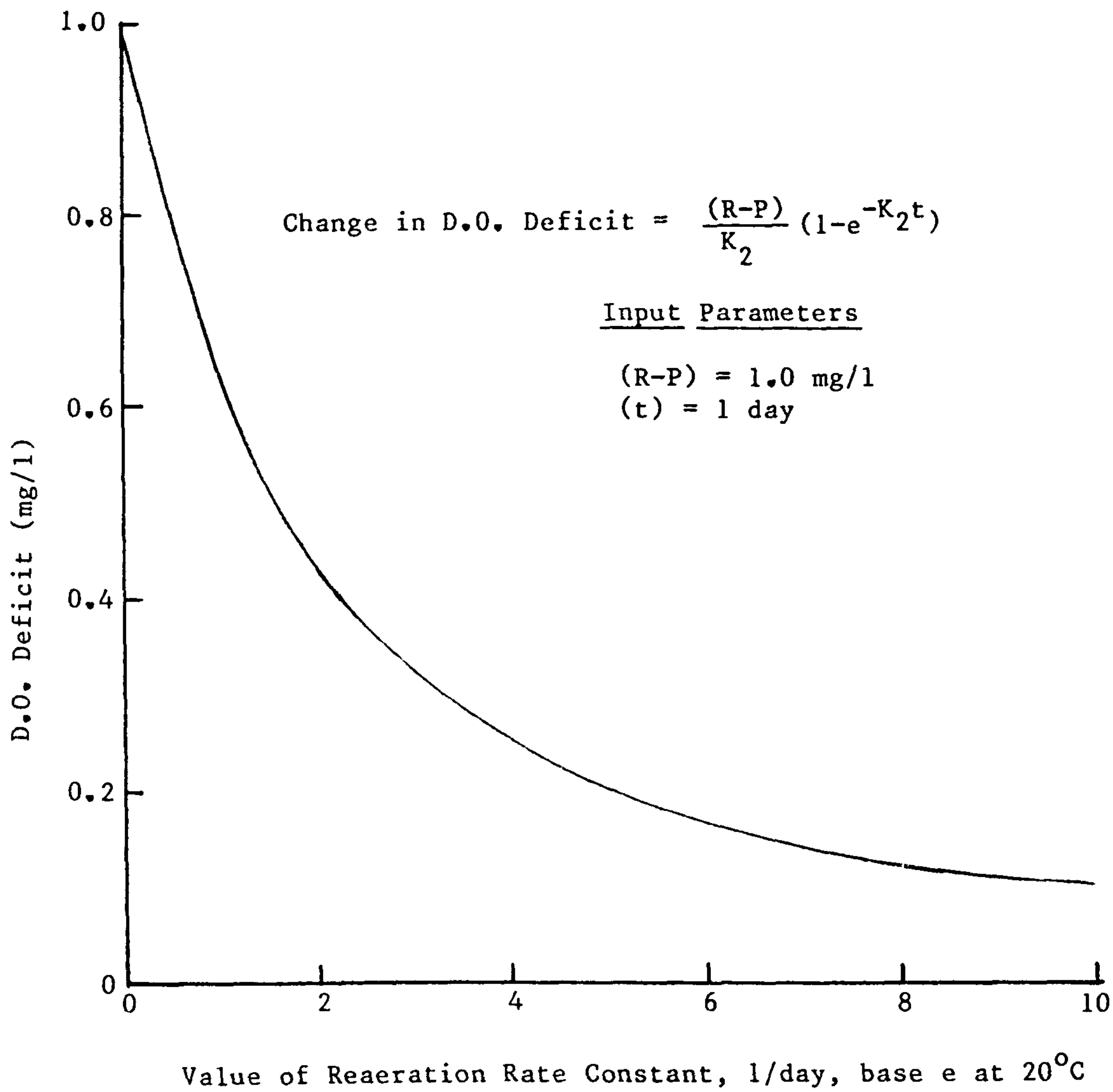

Figure 26. Effect of varying reaeration rate constant on D.O. deficit in the photosynthesis minus respiration equation

However, the basis for the sensitivity analysis itself was to demonstrate how model input affects model output. Throughout TenEch's analysis, all of the results for the sensitivity were based on arithmetic plot comparisons. To continue with the arithmetic comparisons, Figure 27 shows the impact that (t) has on all values of (e^{-t}). As shown, a unit increase in (t) will have a major affect on other variables when (t) is small. As (t) is directly affected by the velocity (V) term, the above comparison applies to changes in V as well. The TenEch (1978a) analysis seems to have neglected this issue.

Discharge Measurement South of Ames

A close approximation for the discharge at the South Sixteenth Street gaging station can be made by simply adding the two upstream station discharges, since little additional drainage occurs between the three stations. An even closer approximation could be made by proportionately increasing the combined discharge, which would accompany the additional increase in drainage area. Appendix B shows that the combined drainage areas of the two upstream gaging stations totals 519 square miles. This compares to 556 square miles for the drainage area at the South Sixteenth Street gaging station. Expressed as a ratio of the South Sixteenth Street gaging station drainage area divided by the sum of the two upstream gaging station drainage areas, the expected increase, in a corresponding discharge, would be 1.071 (556 sq. mi./519 sq. mi. = 1.071). This is 7.1% greater than the summation of the two upstream drainage areas or ultimately their discharges. However, a comparison of over 14 years of combined monthly discharges (when all 3 gaging stations

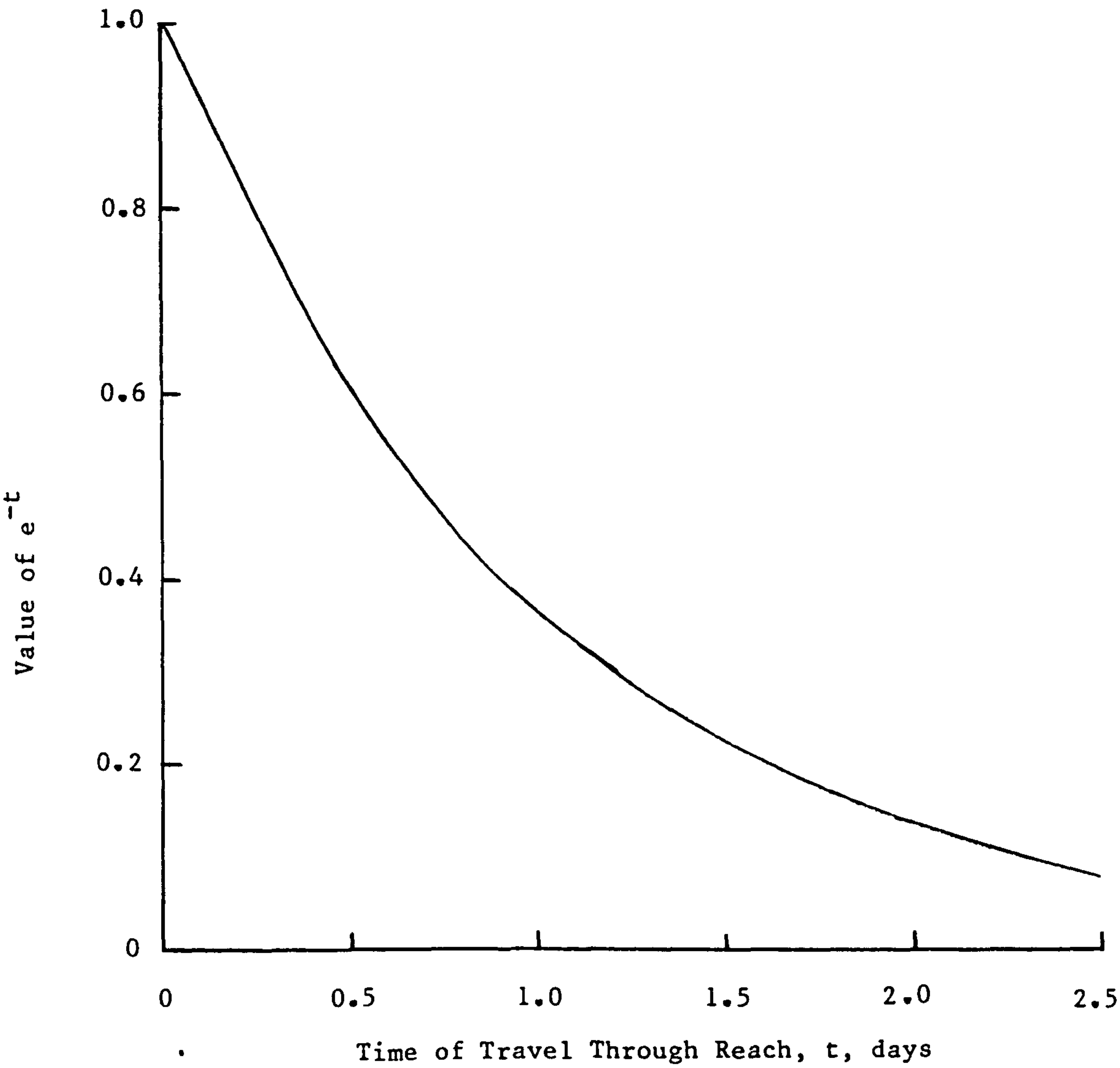

Figure 27. Effect on values of e^{-t} with changing values of t

were in operation) show that the ratio is actually less than this, especially below 40 to 50 cfs. A semi-log plot showing the relationship between the ratio described above (arithmetic) versus the combined discharge value (logarithmic) of the two upstream gaging stations is shown in Figure 28. The discharge data for the comparison is shown in Appendix F, beginning with June 1965.

The results are not totally surprising considering the geology of the area and the location of the Ames well field. The Ames well field extracts water from a buried preglacial alluvial aquifer, which is directly recharged from the Skunk River between Hallett's Quarry and the South Sixteenth Street bridge. Consequently, any withdrawal between the gaging stations located at these sites would result in a lower ratio. Obviously, the withdrawals by the well field cannot be any greater than what would be pumped by the City of Ames, nor should it be implied that all of the water pumped from the well field originates from the river. What is important is the relative magnitude, or recognition of what the upper limit of withdrawal from the river could be. Thus, at a pumping rate of 4.4 to 6.7 mgd, during the years 1965 to 1979, (pumping data from Drustrup, Civil Engineering graduate student, Iowa State University, personal communication, 1984) a flow reduction of 6.8 to 10.4 cfs could be realized. Below 40 to 50 cfs, this represents a sizeable percentage of the combined flow and hence, a very steady and noticeable drop in the ratio appears. Above 40 to 50 cfs, the loss cannot be as easily discriminated from the inherent 10 to, 15% or greater degree of accuracy already in the recorded data.

The approximate mathematical relationships obtained from the

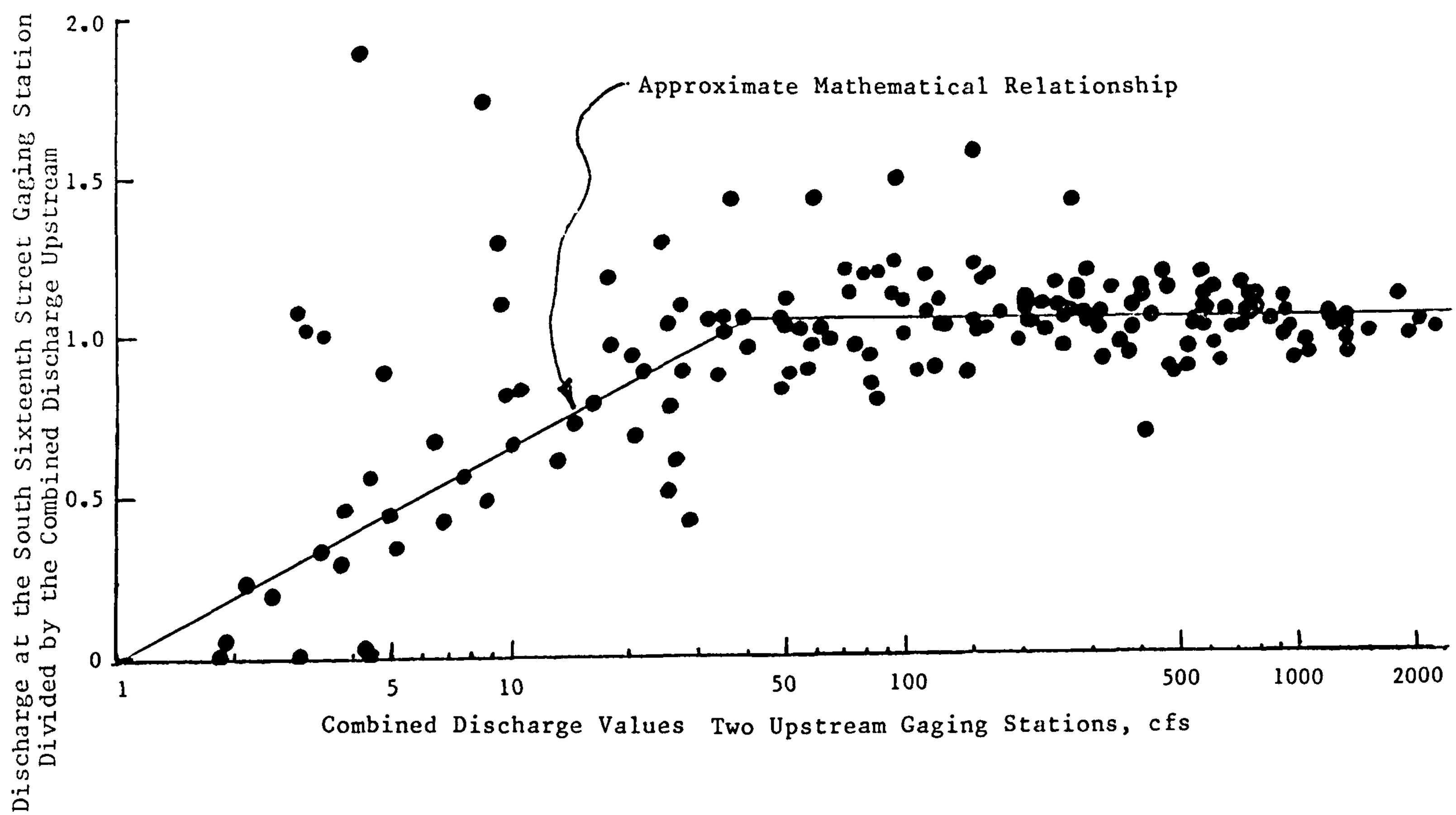

Figure 28. Discharge ratio trend for the discharge south of Ames compared with two upstream gaging station discharges

semi-log plot attempts to historically arrive at the discharge of the discontinued South Sixteenth Street gaging station using the combined discharges of the upstream gaging stations only. The derived equation, from Figure 28, is as follows:

$$D3 = Z \times (D1 + D2)$$

where,

D1 = The discharge (cfs) at the upstream gaging station located north of Ames near Hallett's Quarry on the Skunk River (IGS identification number 05-4700.00),

D2 = The discharge (cfs) at the upstream gaging station located east of the ISU campus at Ames on the Squaw Creek (IGS identification number 05-4705.00),

D3 = The discharge (cfs) expected at the discontinued gaging station located at the South Sixteenth Street bridge at Ames, on the Skunk River (IGS identification number 05-4710.00) and,

Z = A discharge ratio that varies with the combined discharge values of D1 plus D2, with the qualifying combined discharges of (D1 + D2).

< 1 CFS ,	$Z = 0$
1 to 40 CFS ,	$Z = \log [(D1 + D2)] / (1.53)$
> 40 CFS ,	$Z = 1.05$

More scatter is evident in the discharge ratio, as the combined discharge values decrease, such that use of "Z" values below 10 cfs are questionable. Fortunately, the effect of this uncertainty when determinating the average stream velocity using Figure 9 (Dougal, 1969) is small because the increased amount of effluent discharge from the Ames WPCP. The discharge currently averages near 10 cfs compared to 5 cfs in Dougal's 1969 paper. Hence, there would be less variability in average stream velocities today, as the curve becomes progressively flatter as discharges increase.

Low and high outliers were analyzed to see if they were random occurrences or whether some other factors could be employed to help in arriving at values for "Z". Low outliers, above 10 cfs, were found to occur in March after periods of low winter flow conditons. High outliers, above 10 cfs, were found to primarily occur in the fall (or early spring) after previous periods of high flow conditions. Rationally, these observations have a simple explanation if one considers the river to be in union with the groundwater system. More specifically, in extremely dry periods the river would act to recharge the groundwater system, while in extremely wet periods the reverse would occur. Hence, the Skunk River acts as an "effluent" stream during wet periods and as an "influent" stream during dry periods.

Due to the observances of the low and high outliers, the value of "Z" may be changed slightly to compensate for previous dry winter periods (associated with low flows) or for previous wet summer or winter periods (associated with high flows). The change could increase or decrease the value of "Z" by up to 35%, from the value obtained from the relationships earlier. As a guide in altering the values of Z, the following suggestions are made:

1) Decrease March (or other early spring) values of Z by 35%, if the

sum of the upstream monthly average discharges were less than 10 cfs for the last 3 or more months and

2) Increase August or September (or possibly even early spring) values of Z by 35%, if the "current" upstream monthly discharge is less than 100 cfs and the previous 3 or more months were greater than 100 cfs, with at least two of them substantially above 100 cfs, such as 500 cfs.

A plot of the predicted (using the equations and guidelines presented above) versus the actual discharge at the South Sixteenth Street gaging station, for combined discharges over 10 cfs, are shown in Figure 29. The "statistically" best fit line also appears along with the plotted points. The correlation coefficient "r" obtained from the linear regression was 0.996 with a standard error of estimate of 41.0 cfs. The fact that the intercept and slope of the line are so close to 0 and 1, respectively, provides an additional indication of how good the predictions are.

The relationship given for determining the discharge at the discontinued gaging station at South Sixteenth Street represents a "best guess" approach as to what the actual discharge could be. The plus or minus changes in the value of "Z" represents a bracketing of this "best guess" value. Therefore, while not definitive, this method does allow a modeler to arrive at a discharge for this station, which in turn can be used to find a corresponding velocity. The bracketing of discharge values can then be ultimately reevaluated at the upper and lower limits to see if any discernible changes in velocity could be realized.

To ultimately arrive at an average velocity in the Skunk River, using the method described above, a modeler would have to know the

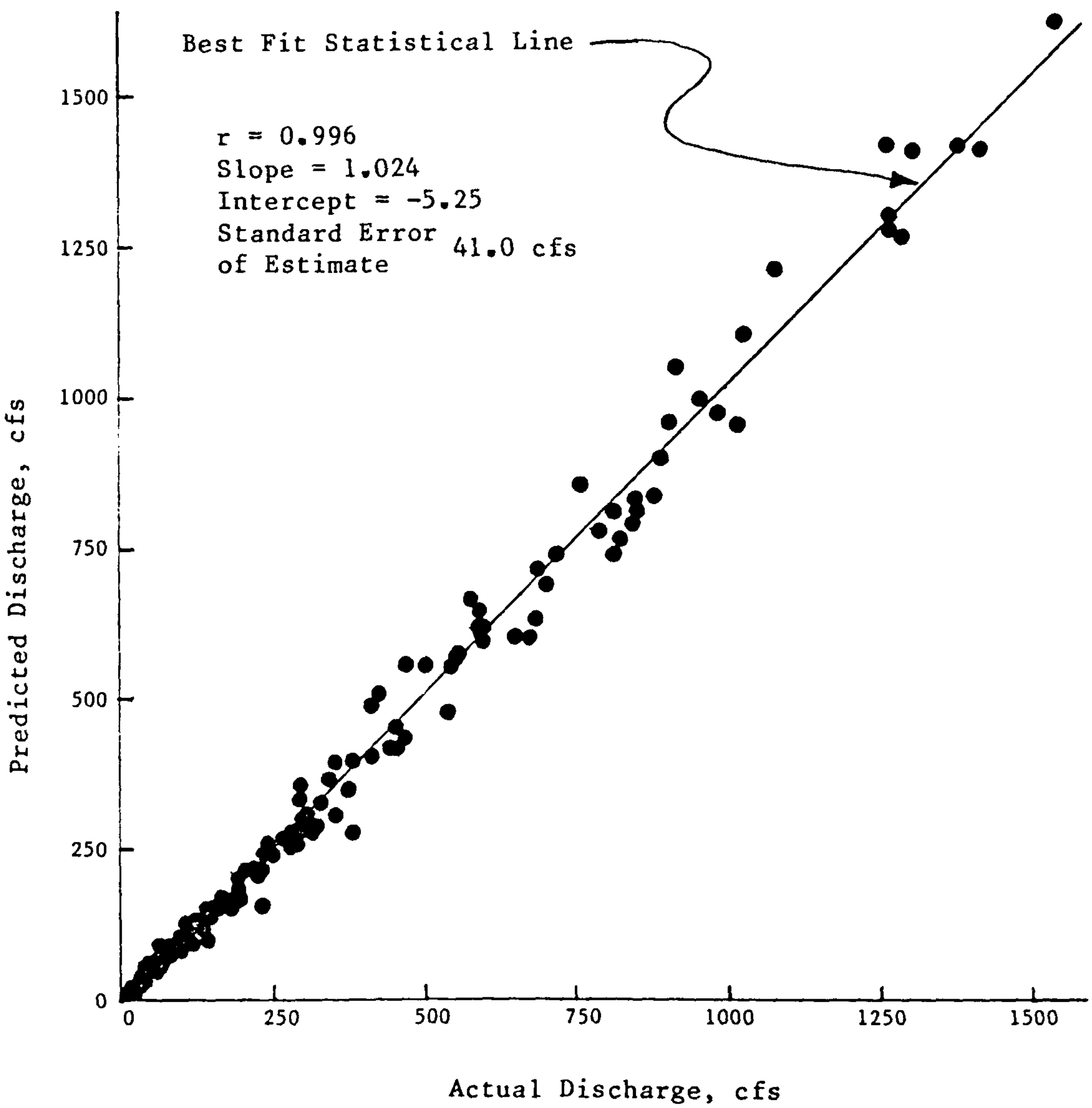

Figure 29. Relationship between predicted and actual discharge south of Ames

discharges of the two upstream gaging stations and the discharge of the Ames WPCP. One drawback to obtaining the discharges at the gaging stations is the time delay required to obtain any official information from IGS, since gage readings are normally collected on a monthly basis. If this time delay is unacceptable, the gage height for each of the stations could be obtained by physically visiting the stations for each day of interest. This represents a fairly simple task, since both gaging stations are near Ames. An even easier method of obtaining the gage height at the Hallet's Quarry exists, as the water-stage recorder is electronically hooked up to the telephone, so that the gage may be telemetered to the nearest tenth of a foot.

This method is so convenient, that an approximation for the discharge at the discontinued South Sixteenth Street gaging station was investigated using the telemetered gage only. A similar method of arriving at the relationship was performed using only the Hallett's Quarry gaging station data. The semi-log plot in Figure 30 shows the relationship obtained between the ratio of the discharge at South Sixteenth Street divided by the discharge at the Hallett's Quarry gaging station (arithimetic), versus the discharge at Hallett's Quarry (logarithmic).

More scatter is evident in this data due to major differences in runoff which occur between the two drainage basins contributing to the combined discharge. The plot shows trends which are very similar to the previous plot. The expected ratio, due to the differences in drainage areas is 1.765 (556 sq. mi./315 sq. mi. = 1.765). The actual ratio above 20 cfs was very close to this value, at just over 1.7.

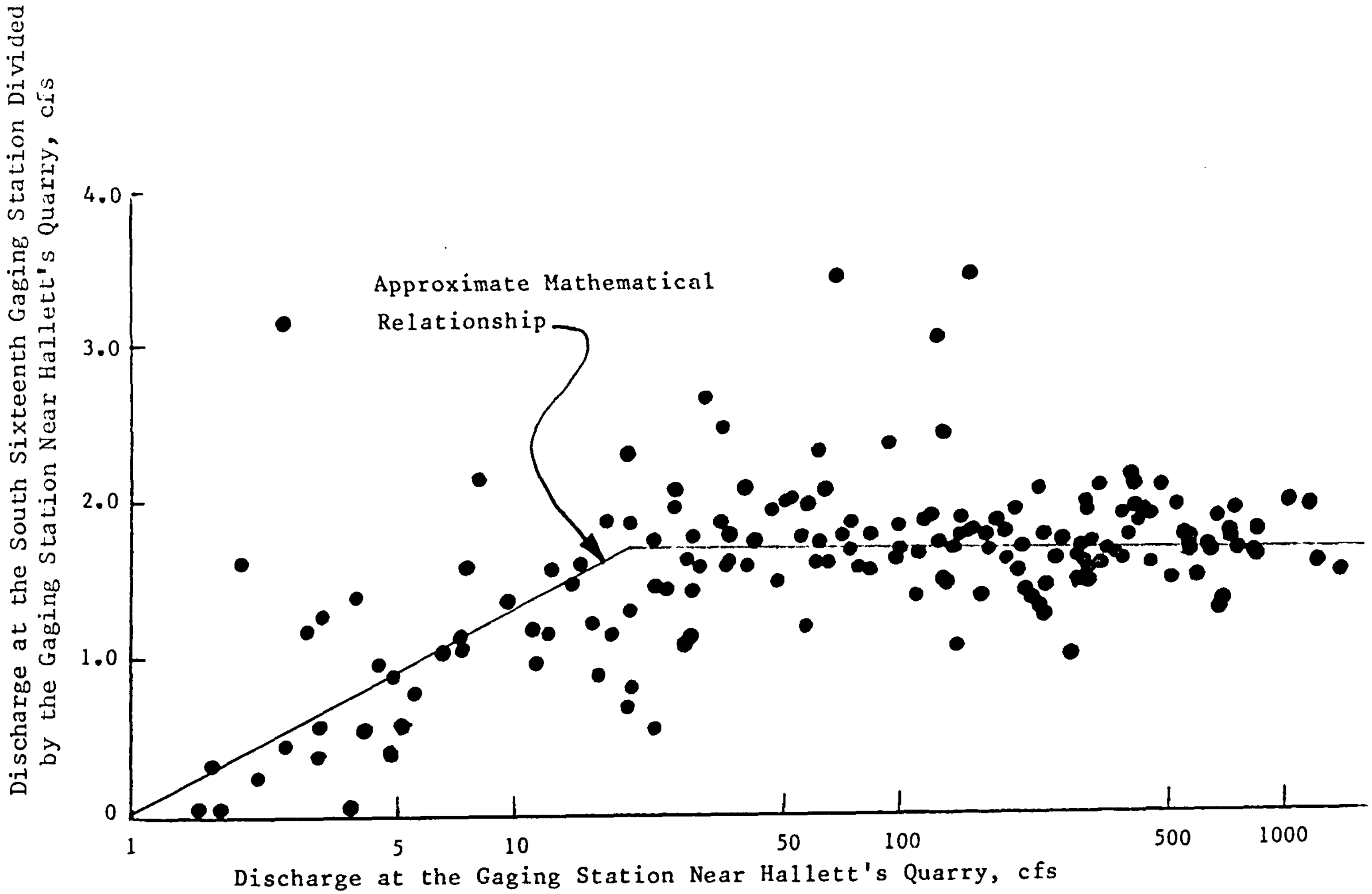

Figure 30. Discharge ratio trend for the discharge south of Ames compared with one upstream gaging station discharge

The approximate mathematical relationship obtained from this second semi-log plot is as follows:

$$D3^{*} = Z^{*} \times (D1)$$

where,

D1 = The discharge (cfs) at the upstream gaging station located north of Ames, near Hallett's Quarry, on the Skunk River (IGS identification number 05-4700.00),

$D3^{*}$ = The discharge (cfs) expected at the discontinued gaging station located at the South Sixteenth Street gaging station, on the Skunk River (IGS identification number 05-4710.00) and,

Z^{*} = A discharge ratio that varies with the discharge of D1, as shown below, with the qualifying discharge values for D1.

< 1 cfs , $Z^{*} = 0$

1 to 20 cfs , $Z^{*} = 1.3$ [log (D1)]

> 20 cfs , $Z^{*} = 1.7$

These relationships are not intended to be used to produce reliable velocity determinations, but instead to be used as an approximation to it, simply by picking up the phone.

Sampling Verification

Data obtained from the four sampling periods are presented below in graphical or tabular form.

Figure 31 shows the discharge and D.O. profile data obtained from the current-meter measuring excursion on July 28, 1983. Average discharge at the South Sixteenth Street bridge and the Ames WPCP flow are

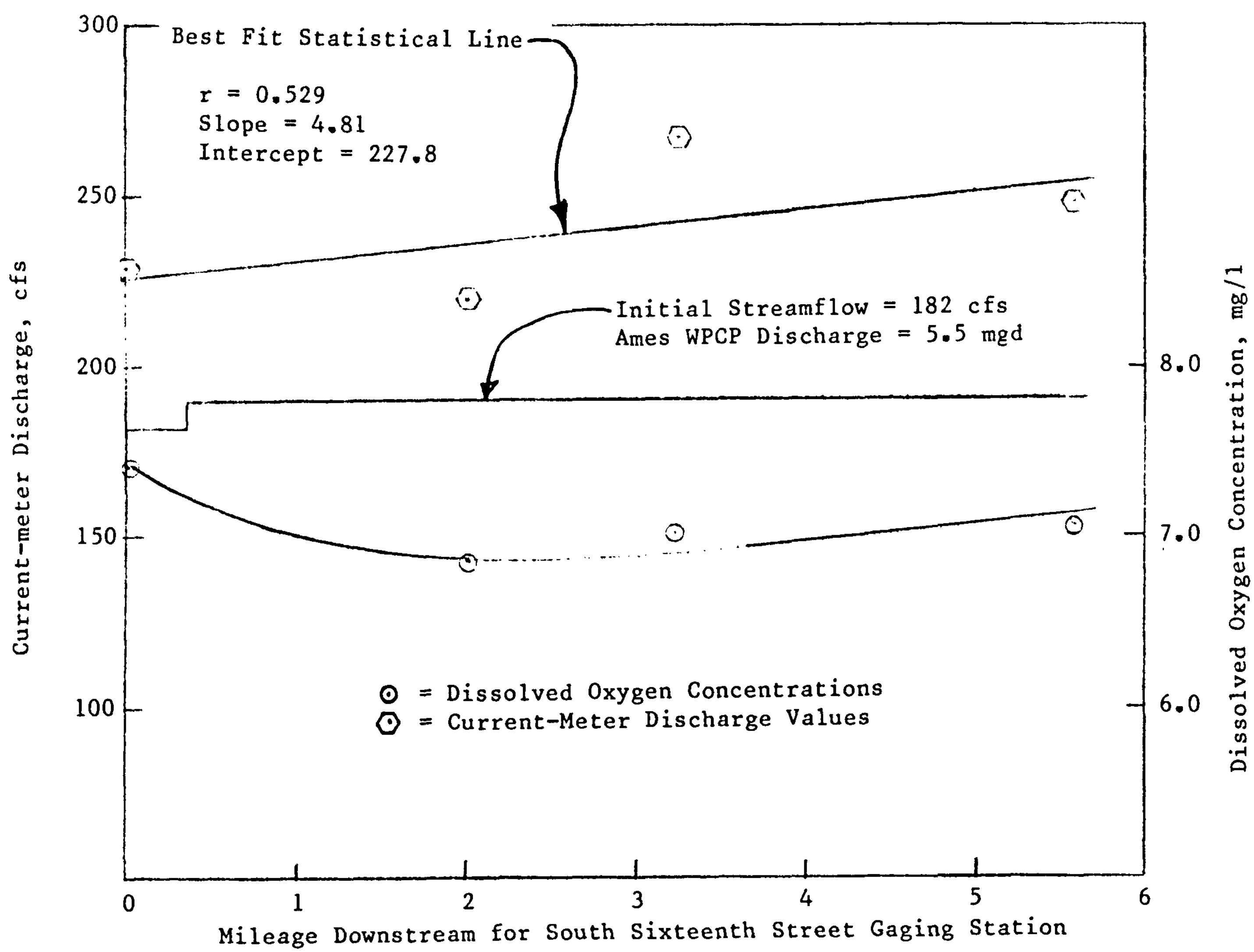

Figure 31. Current-meter discharge and D.O. fluctuations south of Ames on July 28, 1983

indicated on the figure.

Figures 32 and 33 show the results obtained from the August 4th through 5th sampling event. Actual D.O. concentration values are presented in Figure 32 with saturation values shown in Figure 33. Dougal's (1969) July 12th, 1966 data contrasts the more recent data.

Tables 19 and 20 present data obtained from the sampling events of September 14th and October 6th, respectively. Figure 34 shows the logarithmic plot of ammonia and carbonaceous BOD concentrations versus time for both sampling events, as indicated.

Dissolved oxygen data for the October 6th through 7th sampling event appear in Figures 35 and 36. Figure 35 shows the data from the diurnal study while Figure 36 presents the profile portion.

Modeling Analysis or Curve Fitting

A critical step in the WLA procedure is the calibration or adjustment of model input parameters to observed stream data. This calibration or curve fitting exercise was performed on sampling data to aid in evaluation of this important step.

The original "Stanley" model was used in the exercise as adequate sampling data was unavailable for calibration of the more sophisticated models, such as Qual II. The original "Stanley" model was specifically chosen, since it represented an ideal starting point for model calibration in general, due to its simplistic nature.

Sampling data used in the calibration step is present in Table 21 and was obtained from Dougal's (1969) study on the Skunk River. Stream velocity was obtained using Figure 9 and was found to be about 0.2

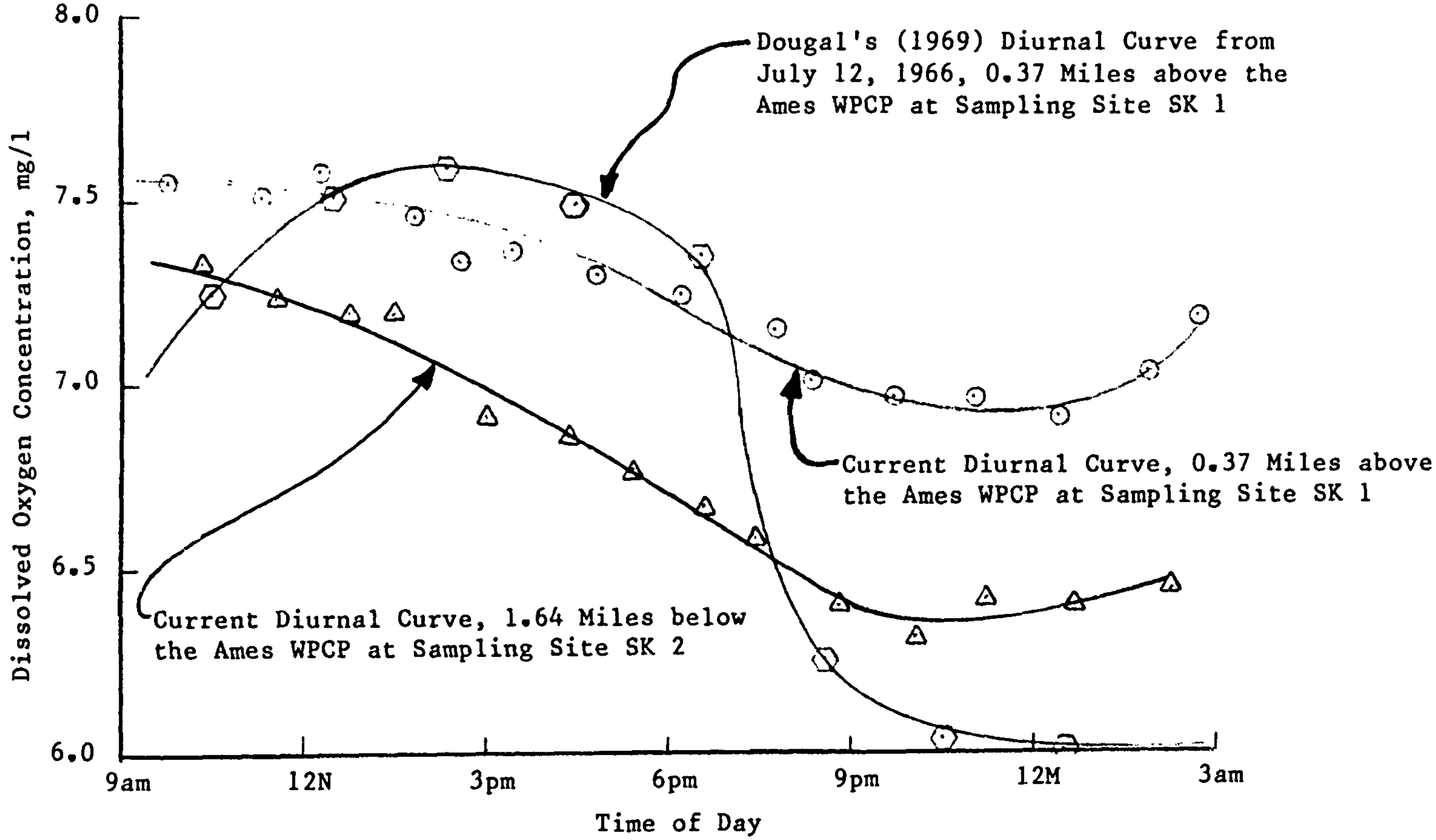

Figure 32. D.O. diurnal variations on August 4 and 5, 1983

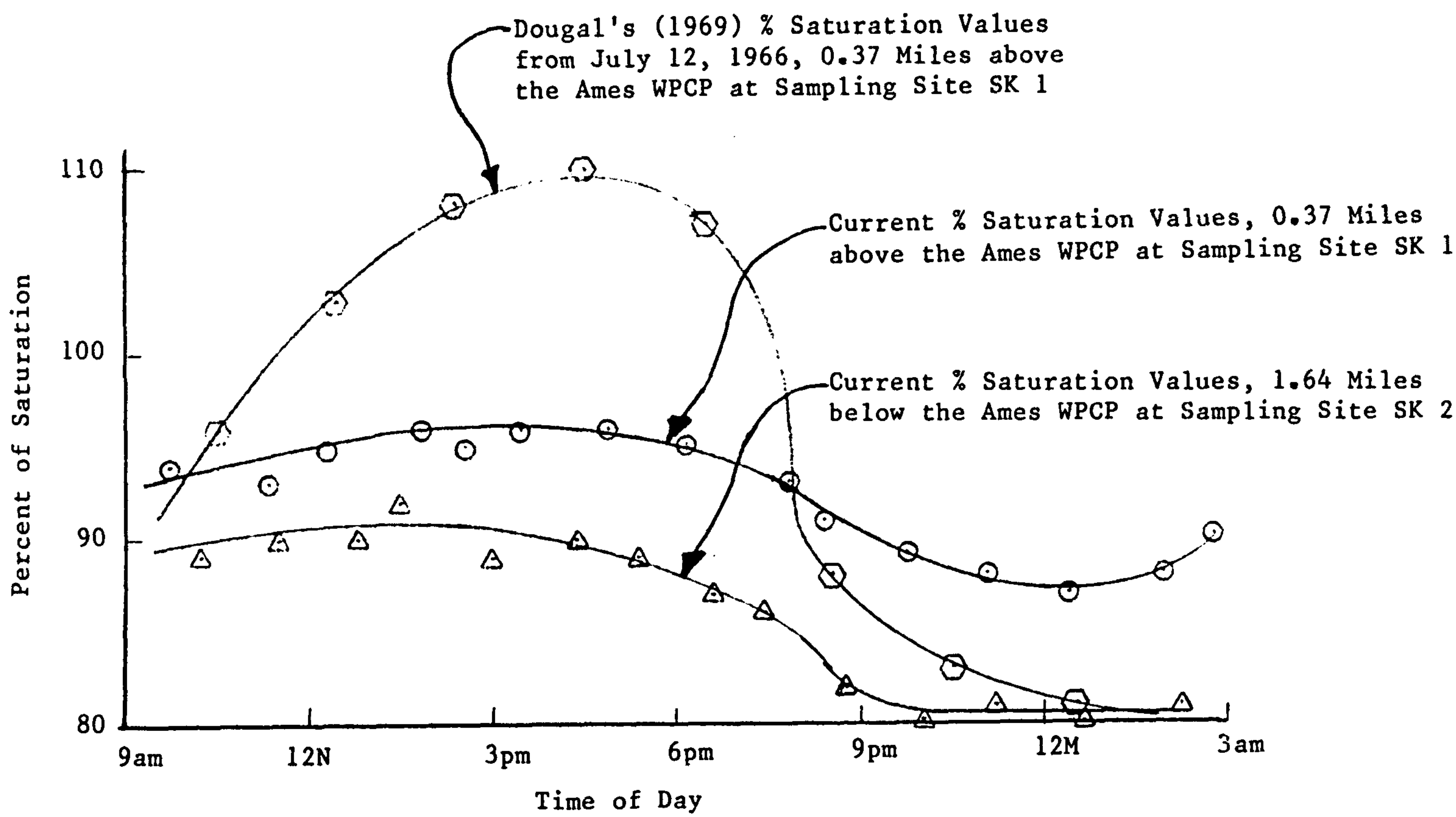

Figure 33. Percent D.O. saturation diurnal variations on August 4 and 5, 1983

Table 19. Sampling data for September 14, 1983

Parameter	Sampling Site				
	SK 1	SK 2	SK 3	SK 4	Units
Total TKN	1.25	4.92	3.38	1.40	MG/L N
Dissolved TKN	.88	4.23	2.90	.94	MG/L N
Ammonia-N	.15	3.43	2.38	.45	MG/L N
NO_3+NO_2-N	5.63	5.71	5.59	4.89	MG/L N
BOD	3.1	5.7	5.0	3.1	MG/L
CHLOR A	14	18	17	17	MG/CU M
CHLOR B	2	2	2	2	MG/CU M
CHLOR C	0	2	1	1	MG/CU M
CORR A	14	17	18	16	MG/CU M
PHEO A	0	2	0	0	MG/CU M

Table 20. Sampling data for October 6, 1983

Parameter	Sampling Site SK 1	SK 2	SK 3	SK 4	Units
BOD	1.5	3.8	3.3	2.3	MG/L
Total TKN	.83	2.74	2.32	1.19	MG/L AS N
Dissolved TKN	.59	2.23	1.92	.82	MG/L AS N
NO_3+NO_2-N	10.8	9.22	9.26	8.89	MG/L AS N
Ammonia-N	.10	1.30	1.68	.35	MG/L AS N
CHLOR-A	6	7	6	6	MG/CU M
CHLOR-B	1	2	2	1	MG/CU M
CHLOR-C	0	0	0	0	MG/CU M
CORR-A	5	4	5	5	MG/CU M
PHEO-A	3	5	2	2	MG/CU M

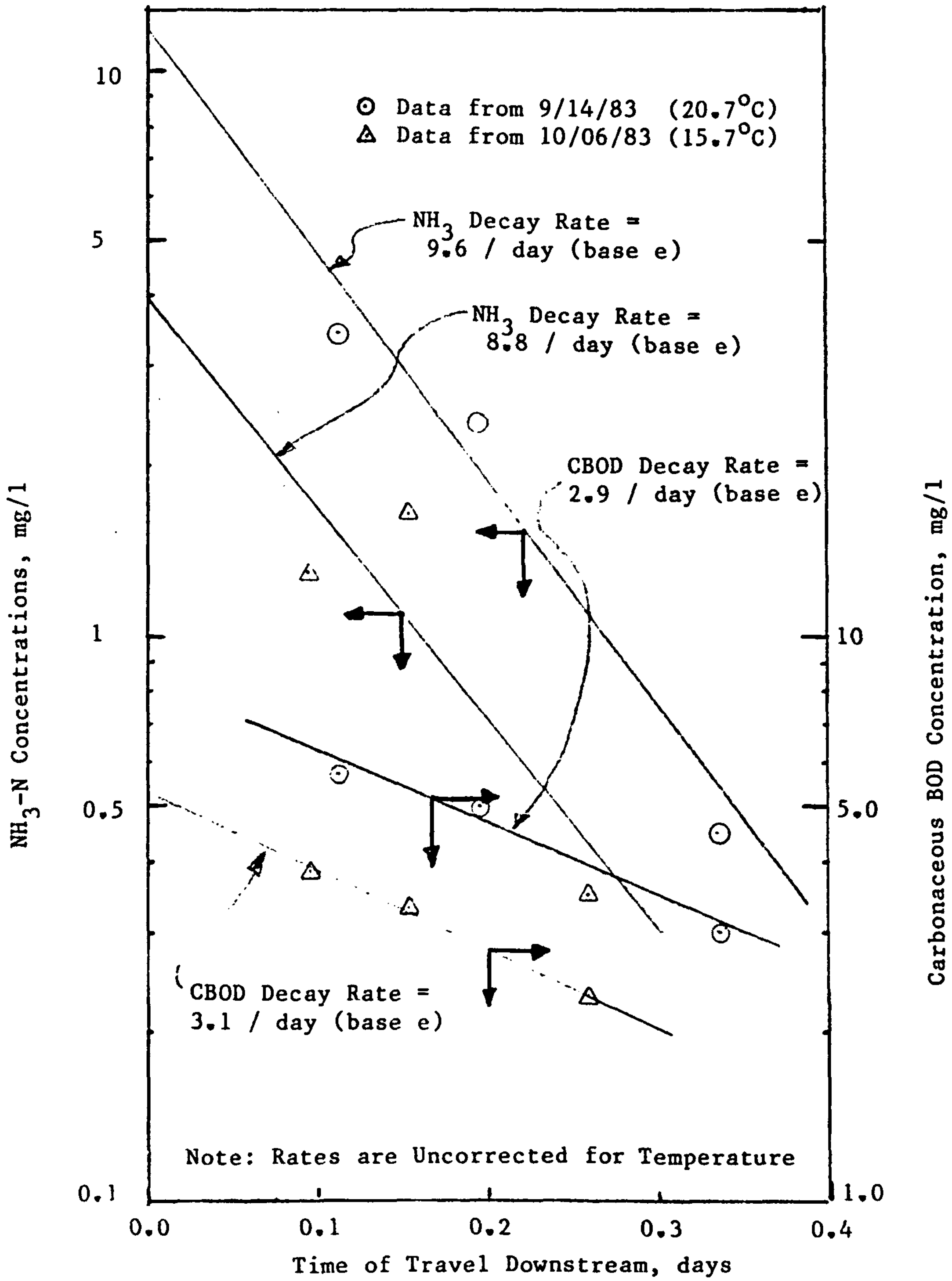

Figure 34. Nitrogenous and carbonaceous deoxygenation removal rates south of Ames

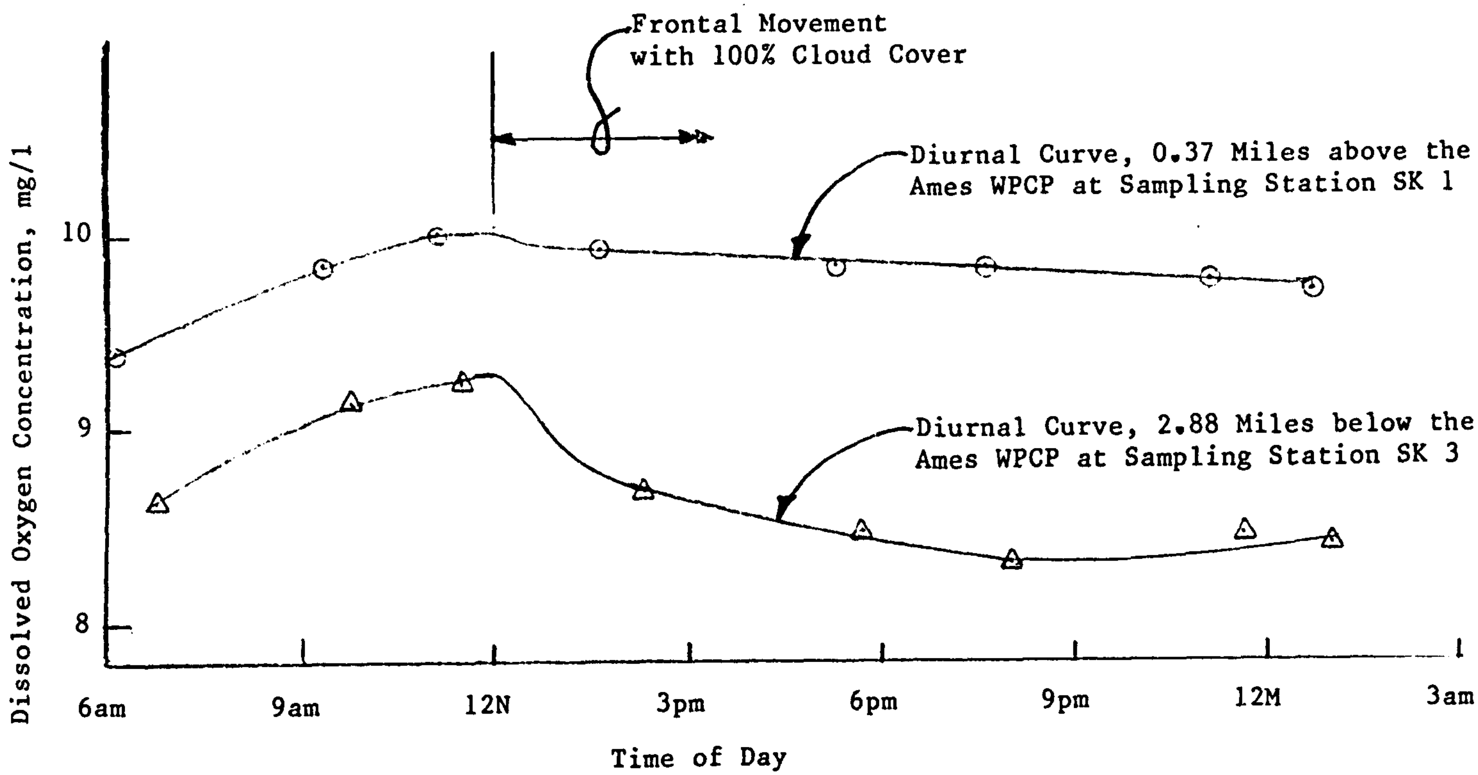

Figure 35. D.O. diurnal variations on October 6 and 7, 1983

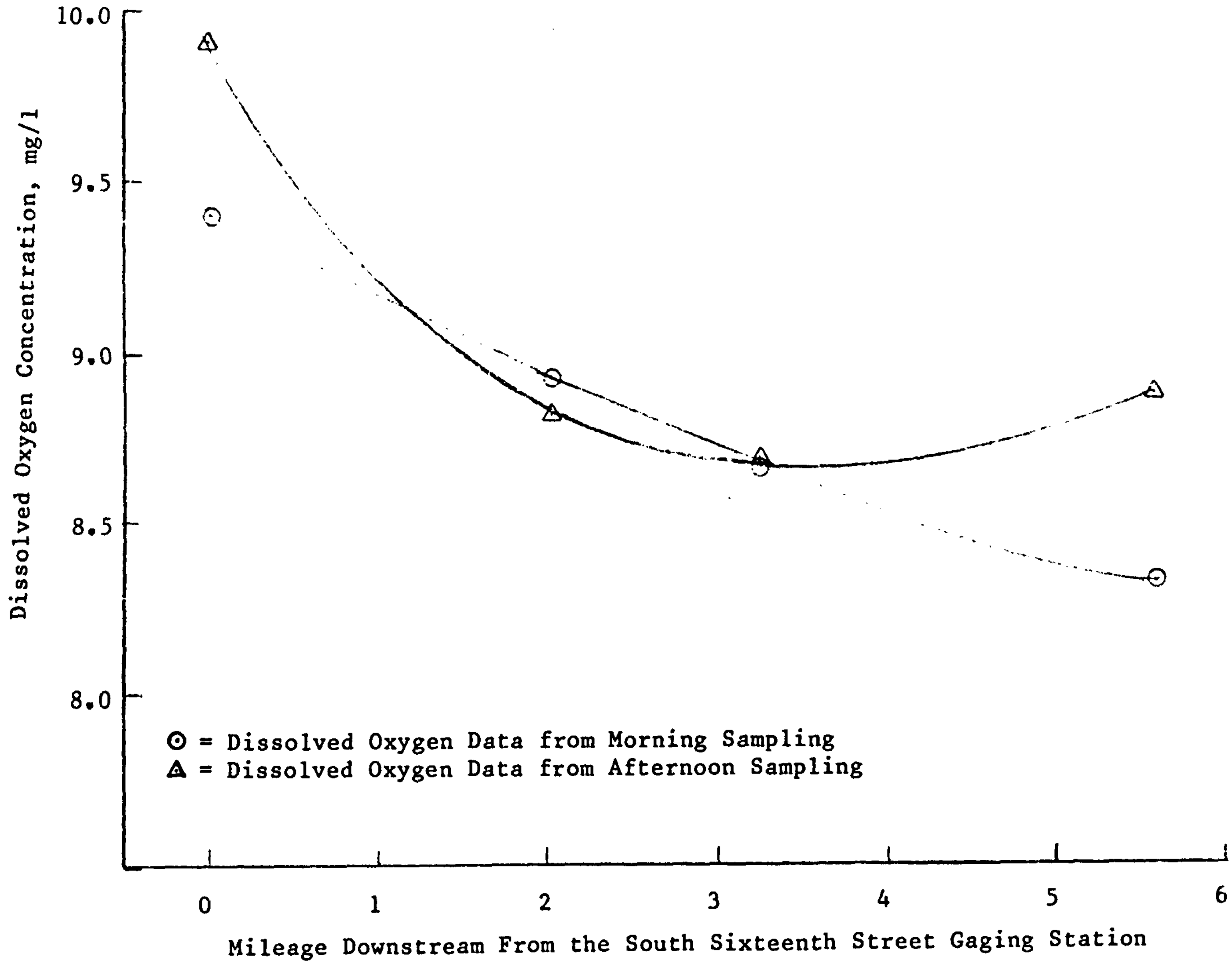

Figure 36. Downstream D.O. profile variations on October 6, 1983

Table 21. Water quality in the Skunk River from Dougal's August 31, 1966 sampling[a]

Parameter	Value at Indicated Reference Mileage[b] Mile 0.00	Mile 0.38	Mile 2.01	Mile 3.25	Mile 5.58	Mile 9.18
Dissolved oxygen mg/1[c]	5.77	-	4.13	3.44	3.75	5.53
Temperature, °C[c]	-	-	21	21	21.5	21.5
Ammonia nitrogen, mg/1	0.5	5.6	1.3	-	1.2	1.1
Carbonaceous BOD, mg/1[d]	-	5.06	2.46	1.97	2.83	2.46

[a]Stream discharge of 9 to 10 cfs with 4.5 to 4.8 cfs from Ames WPCP.

[b]Reference mileage adjusted from mileage reported by Dougal (1969).

[c]Values obtained at night from 0:45 a.m. to 2:30 a.m.

[d]Five day pasteurized and filtered values.

meters/second. These data were chosen for the following reasons:

1) The data were obtained at low flow conditions,

2) The data were essentially complete at each sampling site, and

3) The sampled river reach went beyond the critical dissolved oxygen location.

The input parameters of K_1, K_2, K_N, and stream velocity were changed until a reasonable fit with the observed D.O. data was obtained. The input parameters of initial D.O. deficit and waste loads were kept constant for the curve fitting, except when nitrogenous loads were reduced to simulate algal uptake. Table 22 shows the input parameters for various calibration attempts which have resulted in the reasonable fit of D.O. levels.

Table 22. Input parameter results from curve fitting exercise with Dougal's August 31, 1966 data

K_1, 1/day base e	K_2, 1/day base e	K_N, 1/day base e	V m/s	L mg/l	N mg/l
0.3	3	0.8	0.1	7.5	25
0.3	6	2.0	0.22	7.5	25
0.3	11	3.6	0.40	7.5	25
8	5.5	0.3	0.40	7.5	25
1.3	2.5	0.4	0.08	7.5	25
5	12	2.0	0.40	7.5	25
6	4.4	0.3	0.27	7.5	25
4	3.5	0.3	0.18	8.5	25
2	2.4	0.3	0.10	7.5	25
0.5	1.45	0.3	0.05	7.5	25
0.5	2.1	0.5	0.07	7.5	25
0.8	2.5	0.5	0.08	7.5	25
1.3	7.1	1.7	0.22	7.5	25
1	4.5	3.3	0.22	7.5	12
1.8	4.8	3.1	0.22	7.5	12

DISCUSSION

Iowa's Water Quality Models and the WLAs

The State of Iowa has proposed a sequential modeling procedure to establish Wasteload Allocations (WLA) in Iowa's rivers and streams. Three sequential steps are involved in the procedure and consist of the use of hand calculations, modeling with a revised form of the modified Streeter-Phelps equation, and modeling with a sophisticated model called Qual-II. As a result, very few questions can now be raised concerning the simulative ability of the model, since the use of Qual II represents the current "state of the art" in modeling.

Use of hand calculations represent the most simple approach to establishing WLAs for the State. The legitimacy of the equations appear to be difficult to evaluate, because of the unknowns associated with the 7Q10 discharge and the D.O. deficits. However, these equations can be analyzed by assuming that the 7Q10 discharge is equal to zero cfs and by ignoring the D.O. deficit problem. The 7Q10 assumption of zero cfs is the statistical low flow for the Skunk River south of Ames and actually represents the worst case condition, since other flow contributions would increase the stream capacities.

Use of the above assumptions yields a maximum allowable effluent concentration discharge of 3.7 mg/l for carbonaceous five-day BOD, 2.0 mg/l for summertime ammonia, and 4.5 mg/l for wintertime ammonia. The ammonia limitations are very close to the maximum allowable instream water quality limits for the seasons indicated. Larger ammonia limits could be possible for wintertime ammonia conditions and still be within the State's guidelines. Larger ammonia limits, if allowed under the proposed EPA criteria would allow even further ammonia contributions. The maximum carbonaceous BOD limits of 3.7 mg/l represent an ultimate value of just over 5.5 mg/l. This is less than the value of 6 mg/l assumed for "clean stream" conditions. Hence, the effluent would have to be cleaner than an unpolluted stream, in terms of its carbonaceous BOD concentrations.

The sequential procedure represents a logical approach for DWAWM, in solving the nearly 100 WLAs per year, by focusing modeling attention on those streams where modeling will be most effective. Flexibility in the sequencing process allows the modeler to tailor the "degree of sophistication" required to each discharge location. Unfortunately, the simulative ability of the model and its accurate simulation of the stream are not coincident, but also rely on adequate sampling data and accurate model calibration and verification. Without all three, accurate simulation of a stream response is unlikely.

This fact is most vividly brought out with the introduction of the more sophisticated Qual II model. With the use of this water quality model, the modeler now has the option of using or not using several subroutines to arrive at the desired calibration. Each subroutine, also has several choices in which the input data are arranged or obtained. To complicate the issue even further, most of these options have numerous coefficients and other expressions, which must be simultaneously evaluated to obtain the desired output. Many of these subroutines include the carbonaceous BOD settling rate (K_3), benthic oxygen demands, uniform loading rates, photosynthesis minus respiration effects, and algal uptake of nutrients.

Most of the subroutines and parameters to be evaluated are imprecisely known due to poor sampling, no sampling, or the impossibility of obtaining a value in the first place. As a result, each option has infinite solutions for the same desired end results. While, the end results may match the observed data due to the calibration step, it is unlikely that any of the previous solutions will result in identical end results, when other parameters are changed, as is done is establishing final WLAs.

An ideal example of the problem above deals with the establishment of the algal uptake of ammonia. The equation used in this simulation step, will not be presented again, but consists of several factors that must be obtained from a given range of values reported in the literature. Unfortunately, the ranges given are so large that nearly any of the parameters listed, can dominate the expression and still be within the reported literature value ranges.

Another example deals with the use of uninhibited BODs in establishing carbonaceous oxygen demands. Turkle (Department of Water, Air, and Waste Management, Des Moines, Iowa, personal communication, 1984) suggested that while there appears to be a double counting of some nitrogenous matter, the double counting is adequately taken care of in the calibration step due to an artificial reduction in K_1, and/or K_N rate constant. While this may result in an adequate fit for the calibrated conditions, it is unlikely that these changed rate constants will adequately predict what may occur in another sampling period, such as a low flow condition.

Proposed statewide constants for parameter values also detract from model credibility in establishing WLAs, in the same manner as the uninhibited BOD test in predicting carbonaceous BOD. While, this practice allows for easier model calibration and less sampling expense, there is no guarantee that these constants will adequately describe other flow or waste load conditions.

While, the use of sampling data is suggested to verify or improve parameter values, nothing is mentioned about sampling methodology or time for gathering samples. While it is apparent that adequate procedures must be followed in gathering and analyzing samples, it is not apparent how time will affect these variables. Several questions come to mind that are left unanswered in the WLA procedure and include the diurnal, seasonal, and low flow effects on several of the major factors such as photosynthesis minus respiration, algal uptake of nutrients, or "clean stream conditions." These effects are totally ignored in the WLA procedure, but are vitally important in establishing the critical effluent limitation at low flow conditions.

While several of the equations used in the WLA procedure will be discussed in the upcoming discussion on sensitivity analysis, one important factor, which reduces the reaeration rate deserves special attention at this time. This is the effect of ice cover which reduces the reaeration rate in proportion to the amount of the cover on the river. TenEch (1978b) evaluated this practice and while accepting its use, they concluded that the expression should be experimentally derived in the future. They also mentioned a Minnesota study that indicated substantial ice bridging over the water's surface occurs during low flows, thus allowing for significant aeration. Significant reductions in the wintertime reaeration rate constant values are now occurring, in what essentially amounts to a "wild guess" as to what is actually happening. A significant gain in the reaeration rate constant coupled with an increase in allowable ammonia increases could substantially alter the present day WLAs.

Sensitivity Analysis

TenEch (1978a) reached an overall conclusion based on their sensitivity analysis of the original "Stanley" model. The conclusion obtained was that stream velocity and the reaction rate constants were the most critical parameters affecting model output. (This is a safe conclusion as only initial instream temperature and parameter concentrations remain to be input.)

Stream velocity has a major impact on nearly every equation used in the Iowa water quality models, hence, its accurate determination is particularly important. TenEch (1978a) suggested that the Leopold-Maddock equation be used for velocity determinations instead of the Manning equation, due to the significant impact that the Manning's roughness coefficient has on velocity results. The Leopold-Maddock coefficients have been determined for the Skunk River by Dougal (1969), as reported earlier. The Leopold-Maddock coefficients obtained by Dougal were 0.187 and 0.3442 for a and b, respectively. Since these are relatively low values, their impact on the velocity determination, due to slight variation in the coefficient value would be small.

If the Manning equation were employed for velocity determination, the Manning roughness coefficient should be analyzed in greater detail. The Ames WLA uses a value of 0.05 which is significantly greater than the 0.035 normally assumed in the WLA process. TenEch (1978a) suggested using a back calculation for determining the roughness coefficient "n" from current-meter discharge measurements. However, they reported that this may yield widely varying results due to stream nonuniformity. Using the Ames WLA data presented earlier for slope (S), width (W), and flow rate (Q), it is possible to back out the value for "n" by using Dougal's (1969) Leopold-Maddock coefficients described above. The equation for "n" becomes:

$$n = \frac{0.6}{\sqrt{Q/WV}} \quad \frac{1.5\ W}{Q} \; S^{1/2}$$

where all terms are as defined previously.

Using a Q of 9.9 mgd for the effluent discharge and 1.14 cfs for the upstream low flow discharge results in an "n" value of 0.0476. Thus, 0.05 is quite close.

TenEch (1978a) reported that widths above 20 feet have less of an impact on the velocity determination. They also suggested the following guidelines for measuring stream widths:

width $\leq$ 20' : $\pm$ 3' ,

20' width $\leq$ 50' : $\pm$ 5', and

width > 50' : $\pm$ 10' .

Ames WLA values are 28 feet below the outfall and only 8 feet above it. This suggests that special care should be taken in obtaining reliable measurements for stream widths, especially above Ames. The use of gaging station calibration data for stream is also of questionable accuracy due to the unnatural flow conditions which occur at most gaging stations. A more natural stream segment should yield results of greater value.

Channel slopes affect both velocity and the reaeration rate. Channel slopes become less significant to velocity values as slopes increase, but become more significant to reaeration rates as channel slopes increase. An intermediate value of near 3.5 feet per mile for the Ames WLA has relatively minor impact on modeling output.

The impact on the deoxygenation rates K_1 and K_N are greater when their value is small. Consequently, the Ames WLA values of 0.2 and 0.3 for K_1 and K_N, respectively, suggest that model output could be affected by minor variations in K_1 or K_N. Thus, a fair amount of effort in establishing these values may be worthwhile.

Reaeration rates are strongly affected by higher values of Tsivoglou's gas escape coefficient "C." The Ames WLA data use 0.115 (1/ft) for the value of "C"; hence, its high value suggests that this gas escape coefficient value should deserve closer estimation.

Reaeration is also influenced by ice cover and low D.O. levels. The percent ice cover has a significant effect on reaeration when ice cover exceeds 80%. The Ames WLA uses 90% ice cover conditions in wintertime. This percentage is extremely sensitive to minor variations. The percentage of ice cover demands <u>close</u> estimations in this range. D.O. levels also affect K_2 rates when D.O. gets below 2 to 3 mg/l. This should not occur in any WLAs as the water quality limitation is 5 mg/l. TenEch (1978a) did find, however, that initial background D.O. levels were critical when reaeration was significantly reduced. This should not pose a problem, however, if modeling is adequate.

TenEch (1978a) makes the assertion that waste load concentrations affect the model output. This appears to be a contradiction to what the further analysis showed, where changes in L_o or N_o produced constant per unit D.O. deficits. The differences, however, are due to TenEch's changing of initial dilution water quantities containing additional waste load. Hence, dilution, not waste load changes affected the model output.

Prior to discussing the modified equations in the JRB model it should be noted that the above results for the "Stanley" model apply equally as well to the modified JRB model, and the Ames WLA parameters for the JRB model have yet to be established. As a result, only a comparison of input parameter value ranges can be discussed.

The two major changes to the original Streeter-Phelps equation include the following:

1. Algal uptake equations for NH_3-N and
2. The (P-R) equations used for D.O. deficit calculations.

The first change involves calculation of a local algal growth rate, (GP) employing Michaelis-Menton (M-M) reduction factors. GP is also used in the calculation for NH_3-N uptake by algae. Use of appropriate M-M half-saturation values, estimates of combined nitrogen (N) and phosphorous (P) values, and published values for average light intensity (LI), result in minimal reduction of algal growth rates ($\bar{u}$). Thus, for Ames, the value of GP will vary with ($\bar{u}$). Crumpton (Department of Botany, Iowa State University, personal communication, 1984) noted that M-M kinetics apply only to single species and that their application to mixed species is not considered appropriate.

The value of NH_3-N uptake (UP) by algae, can be expected to vary as shown in Figures 23, 24, and 25. The value of CHLA will have a significant impact on UP, since its concentration can vary substantially over extended periods. As previously shown, values of (GP-DP) and (t) greater than unity (GP-DP being dimensionless and t in days), will significantly affect UP, since e is directly raised to the (GP-DP)(t) power. This, however, should not occur as values for ANP and/or NF would be reduced in the calibration procedure. Crumpton (Department of Botany, Iowa State University, personal communication, 1984) questioned the use of the algal uptake equation for the reasons indicated below:

1. Values for GP, ANP, NF, DP, K_N, and CHLA will vary diurnally, seasonally, as well as, yearly and

2. NH_3-N sink (positive storage) condition cannot be indefinite, but should very probably be assumed to be zero for all practical purposes.

The significance of these two statements depends on what actually is occurring during low flow conditions. Unfortunately, there is no answer to this at present.

The significance of the addition of the (P-R) term for D.O. deficits in the WLA process depends on the values assigned in the computations for P and R, as well as, the K_2 value.

The preceding sensitivity analysis showed that low K_2 values would produce greater impacts on D.O. deficit values. For Ames, low K_2 values would only occur during winter conditions with large amounts of ice cover. Summer conditions will always produce a larger K_2 value, as ice would not be present.

The greatest impact on the values of P and R appears to be the value of CHLA (chlorophyll-a) concentration in the stream. Thus, the importance of this variable should not be underestimated. Values of CHLA can vary fom 0 to 500^+ ug/l.

Two other variables in the derivation of photosynthesis (P) include GP-DP, OP, and AP. The term GP-DP has been discussed previously and should be directly proportional to values assigned to $\bar{u}$, the maximum algal growth rate for the Ames WLA. Ranges given for OP of 1.4 to 1.8 do not vary significantly, therefore they do not appear to be a great problem. On the other hand, the range given for AP, the amount of chlorophyll-a in ug/l per mg/l of algae, varies from 10 to 100. This range is significant and deserves more attention. Interestingly enough, the value for AP <u>needs</u> even further definition, as the value changes rather significantly depending on whether the algae is measured on a <u>dry weight</u> or <u>wet weight</u> basis. This oversight could pose numerous problems to modelers in the future.

It is difficult to quantitatively say how much a change in one variable, in the modified JRB model, will affect the model output, without prior knowledge of all the other parameters involved. Values for the paramemters however, can change D.O. or NH_3 values by several mg/ls.

Discharge Measurement South of Ames

Researchers and modelers in the future will undoubtedly require discharge measurements immediately upstream of the Ames WPCP effluent discharge pipe. Upon completion of the "new" Ames wastewater facility, near 1987, the mathematical relationships developed will not provide adequate discharge approximations to reflect the additional increase in drainage area. Consequently, a new method of establishing the discharge should be investigated. A plot of the combined discharge values versus their percentage of occurrence is shown in Figure 37. This plot suggests that questionable flow predictions (those under 10 cfs) can be expected, on the average, about 18% of the time. This suggests that any discharge measurement method adopted would frequently be used in low flow ranges. Hence, accuracy in this range would be beneficial.

If reliable discharges will be required in the future, a control structure of some fashion will have to be built. Ideally, IGS would install a permanent gaging station at the new location. Otherwise, some type of manual gage reading device at that location, could be used to obtain the desired discharge information.

The method of combining the two upstream gaging stations to approximate a downstream discharge, may provide quick answers to obtaining discharges or average stream velocities, prior to a more reliable system. The use of the telemetered gage and its relationships may be of some benefit in planning any number of research activities, when flows of a desired magnitude are required. Although Dougal's (1969) dye tracer studies showed that average stream velocities were fairly constant downstream to near Colfax, the location of the new treatment

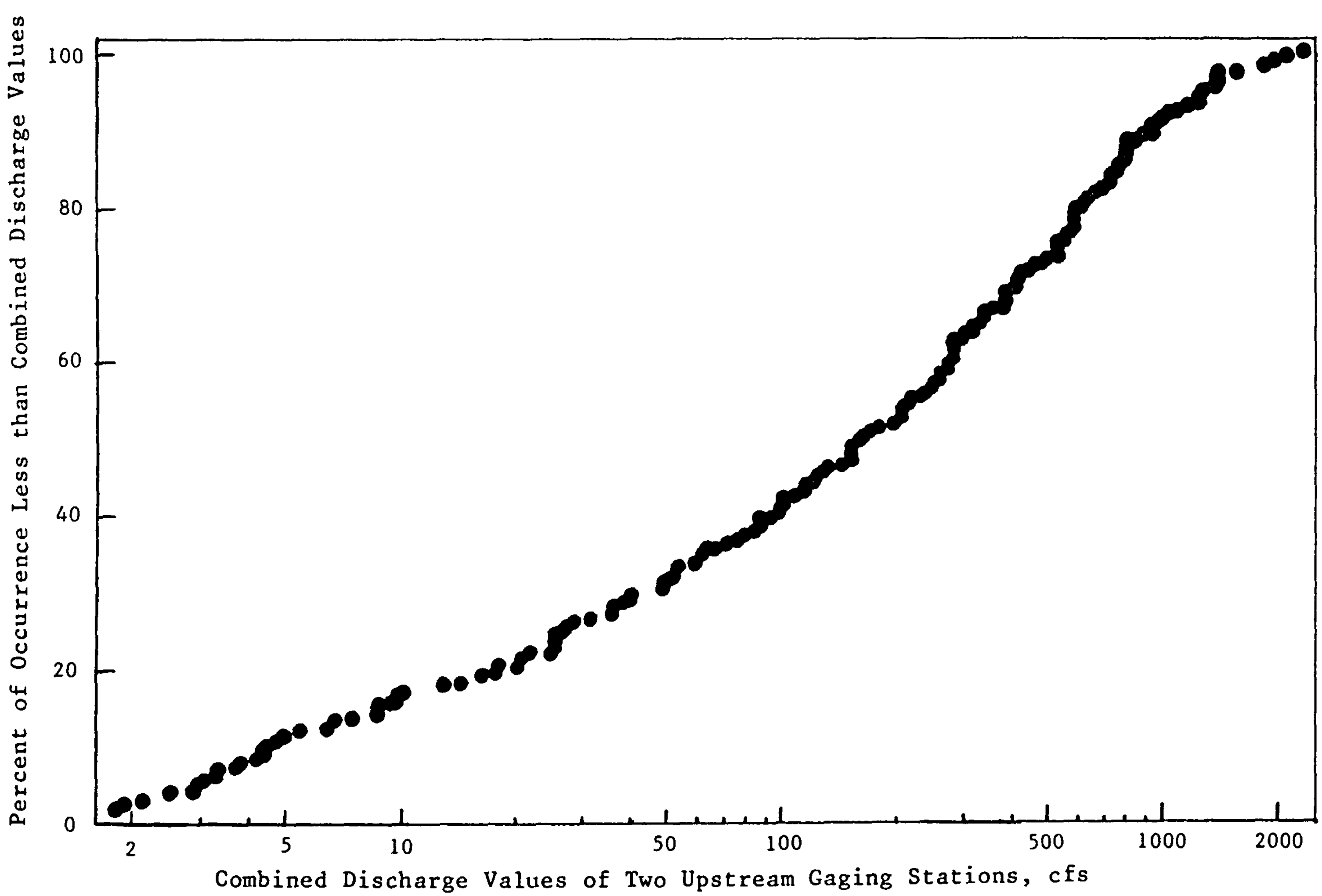

Figure 37. Frequency plot of combined gaging station discharges for Hallett's Quarry and Squaw Creek stations

plant would seem to affect the stream reach velocities between Ames and the new outfall.

Sampling Verification

Results from the current-meter discharge excursion were fairly erratic, suggesting that poor measurement was occurring. Probable sources of error could be attributable to poor measurement technique or to instrument error itself. Regardless of the source of error, the results distinctly show that this method is time consuming and therefore not applicable to numerous research events.

Results from the preliminary D.O. profile study suggest that D.O. recovery downstream from the Ames WPCP is occurring in a relatively short section of the river and that the actual D.O. deficit is fairly small. These results coincide with what would be expected below any treatment plant with ample dilution on a bright, sunny day. Another possible explanation for the observed D.O. profile, could be attributed to the beginning of an "oxygen bulge," resulting from stimulated algal photosynthetic activity. This however, is unlikely considering the amount of dilution water already in the river. D.O. levels ranged from near 80% to 90% of saturation values throughout the reach in question.

D.O. diurnal effects were investigated on August 4th and 5th, above and below the wastewater treatment plant. The results obtained from the study clearly show that some degradation of D.O. *is* occurring below the Ames WPCP, as D.O. levels downstream are consistently less than the upstream D.O. levels. The D.O. discrepancy between the two stations varied from as little as 0.25 mg/l at noon, to as much as 0.6 mg/l near midnight. This variance in D.O. levels indicates the possibility that photosynthetic activity is occurring at a somewhat accelerated rate immediately downstream of the effluent discharge location. Flow variation at the Ames WPCP does exist and it could be possible for diurnal D.O. fluctuations, due to the influx of more oxygen demanding material. However, the typical peak waste discharge occurs in the early morning hours, which would result in the peak load occurring simulataneously downstream with the peak oxygen level in the day.

A comparison of the diurnal data obtained in 1983 with that by Dougal (1969) in 1966, shows a significant difference in the reported time of day for peak D.O. levels. The actual peak occurred nearly 5 hours earlier from the 1966 value, which appeared at about 3:00 p.m.! Although the shift in actual peak concentrations was great, the discrepancy can be explained by the consideration of percent saturation values. This comparison shows that essentially no difference exists between the 1983 and 1966 times for peak saturation values.

Peak saturation values were less than that obtained during the 1966 sampling, probably due to differences in stream discharge or some other physical reason.

The sampling results compare favorably to Dougal's (1969) data obtained nearly 15 years ago. The presence of high K_1 and K_N "river" reaction rates immediately below the treatment plant provides physical evidence of the fantastic assimilative ability of the stream. It also questions the validity of assuming typical reacton rates, which are an order of magnitude less than what were found.

The higher reaction rates found below the treatment facility at Ames may be due to greater biological activity, which increases deoxygenation rates, and/or those "nondeoxygenating" effects, such as the settleability of CBOD, or the K_3 rate constant. The fact that K_N is affected in a like manner, suggests that K_3 term also exists for the nitrogenous matter, as well.

Modeling Analysis or Curve Fitting

The results obtained from the curve fitting exercise indicate that numerous combinations of model input can result in an acceptable match with sampled data. To arrive at one set of input parameters that can be used for establishing the WLA, a modeler must assign values to all but one or two input parameters, based on assumptions or gathered data. Based on the sampling data available, it appears that the reaeration rate constant, K_2 is the most likely candidate for adjustment, since there is not independent measurement for its value (at present). Adjustment for K_1 and K_N may also be considered for adjustment, because of many factors that may affect their apparent river decay rate. These factors were discussed in the literature review, but primarily are due to settling for K_1 and algal uptake for K_N.

Considering the results obtained from the curve fitting exercise, it becomes evident that high K_2 values will not result in reasonable values for stream velocity, K_1, or K_N. Consequently, Dougal's (1969) use of high reaeration rates is not consistent with what this simplified model indicates. It also becomes evident that use of high values of K_N, which would correspond to observed NH_3 decreases in the river, will not result in reasonable values for stream velocity.

Therefore, a substantial reduction in ammonia is occurring, other than nitrification.

A reasonable explanation for the discrepancy in reaeration rates could be attributed to the simplistic nature of the model used in this curve fitting exercise <u>or</u> to Dougal's (1969) misapplication of the higher rate constant in his own curve fitting attempts. The simplistic nature of the model will not be totally discounted, but it would seem that such a wide discrepancy would not be attributed to an over simplification alone. Dougal (1969) arrived at his expression for the reaeration rate constant in the Skunk River, by fitting a line to the average reaeration values from three previously published equations. Hence, there was not an independent check on the value for reaeration, but rather a curve fitting attempt using the higher reaeration rate.

SUMMARY AND CONCLUSIONS

Summary

The State of Iowa has proposed a sequential modeling procedure to establish Wasteload Allocations (WLA) in Iowa's rivers and streams. Three sequential steps are involved in the procedure and consist of the use of hand calculations, modeling with a revised form of the modified Streeter-Phelps equation, and modeling with a computer model called Qual-II. The procedure ultimately establishes a limit on the quality of waste effluent which may be discharged to the waterway.

The WLA procedure theoretically involves the accurate calibration and verification of model parameters which are used to model specific low flow conditions where water quality criteria must be maintained.

Successful WLAs depend on the collection of data for input into the model. This thesis examined various geological, physical, hydrological, and limnological characteristics for the Skunk River basin. Interesting points reviewed included a review of the historic water quality upstream of the Ames Water Pollution Control plant (WPCP) and an investigation into how discharge measurements could be obtained, south of Ames. A sensitivity analysis examined how model input affects model output for two versions of the modified Streeter-Phelps equation. The thesis also gathered important sampling data for the Skunk River, initializing a data base for future modeling studies. In addition, a calibration using collected data was attempted.

Conclusions

Conclusions reached at in this study are:

1. Use of model sequencing represents a reasonable integration of the technical, economical, and institutional factors,

2. Use of hand calculations in determining waste load allocations yield conservative results for carbonaceous BOD, based on the 7Q10 value for Ames,

3. Use of the revised version of the modified Streeter-Phelps equation does represent an improvement to the original model, but has components which are difficult to justify. Adoption of the Qual-II model allows the State of Iowa to use a "state-of-the-art" water quality modeling tool for establishing Wasteload Allocations. However, the effectiveness of providing acceptable modeling results is hindered by the unavailability of sampling data, inaccurate model calibrations, and a difficulity in modeling all the river interactions,

4. Effects of low flow discharge assumptions and time variations may produce significant errors in modeling,

5. The discharge south of Ames may be approximated with use of stream gaging data upstream of Ames,

6. Velocity measurements, vitally important in the modeling procedure should be determined using a Leopold-Maddock approach employing tracer studies,

7. Reaction rate constants which are also important to the modeling procedure, should be determined by direct measurements,

8. Reaction rate constants in the Skunk River are an order of magnitude greater than those typically used by the State of Iowa,

9. Recent sampling results compare favorably to data obtained 15 years earlier,

10. "Clean water" conditions upstream of the Ames WPCP at low flow conditions collected over the last six years by the Ames WPCP personnel are about 5 mg/l for carbonaceous BOD, 4 mg/l for NH_3-N, 2 mg/l for PO_4-P, and 0.5 mg/l for NO_3-N,

11. Greater photosynthetic activity occurs downstream of the Ames WPCP effluent discharge than upstream,

12. Use of high reaeration rates, as indicated in Dougal's (1969) study, were found to be incompatible with the current calibration of the modified Streeter-Phelps equation because of the associated high velocities which were required to obtain a reasonable fit,

13. Numerous input parameter combinations exist for model calibration or curve fitting exercises, suggesting a need for greater data collection, to arrive at a truer calibration.

RECOMMENDATIONS FOR FUTURE STUDY

Recommended areas for future actions and studies are:

1. Initiation of water sampling studies by the State of Iowa to obtain accurate and reliable information at low flow conditions for use in estalishing WLAs. This could be financed by the WLA applicant in hopes of a more favorable WLA,

2. An investigation of the water quality effects of low flow discharges and time variations on WLAs,

3. An analysis of nitrogen transformations including the effects of biostimulation, nitrification, and ammonia toxicity, and

4. Investigation into greater use of the historical weekly water quality data at the Ames WPCP and other treatment plants.

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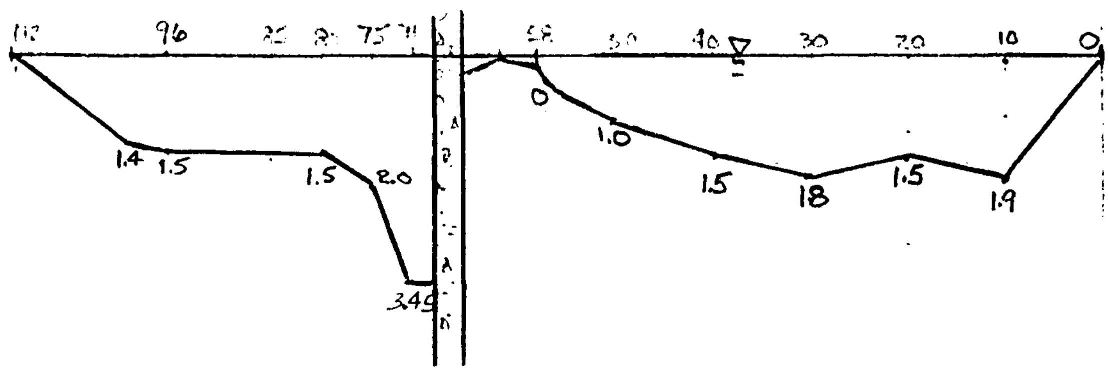
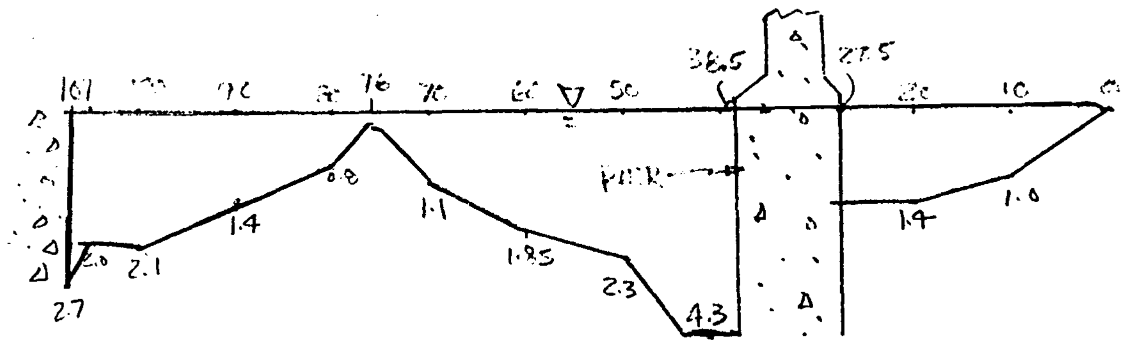
APPENDIX A: CURRENT-METER DISCHARGE CALCULATIONS

Mile 0.00



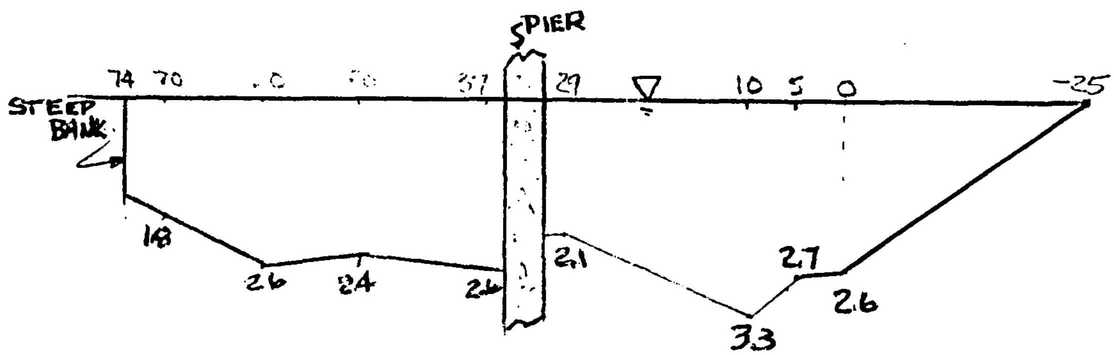
Location (1)	(2)	Depth D1 (ft)	D2 (ft)	Avg. Depth (ft)	Area (ft^2)	Velocity V1 (fps)	V2 (fps)	Avg. Velocity (fps)	Skew Angle (deg)	Corrected Discharge (cfs)
0	10	0.0	1.9	0.95	9.5	0.0	1.62	0.81	30	6.66
10	20	1.9	1.5	1.7	17.0	1.62	2.22	1.92	30	28.27
20	30	1.5	1.8	1.65	16.5	2.22	2.33	2.28	30	32.58
30	40	1.8	1.5	1.65	16.5	2.33	1.87	2.1	30	30.01
40	50	1.5	1.0	1.25	12.5	1.87	1.99	1.93	30	20.89
50	58	1.0	0.0	0.5	4.00	1.99	0.0	1.0	30	3.46
69	71	3.45	3.45	3.45	6.9	2.05	2.05	2.05	0	14.15
71	75	3.45	2.0	2.73	10.9	2.05	1.98	2.02	20	19.02
75	80	2.0	1.5	1.75	8.75	1.98	2.08	2.03	20	16.69
80	85	1.5	1.5	1.5	7.5	2.08	2.12	2.10	20	14.80
85	96	1.5	1.5	1.5	16.5	2.12	1.93	2.03	20	31.40
96	100	1.5	1.4	1.45	5.8	1.93	1.11	1.52	30	7.63
100	112	1.4	0.0	0.7	8.4	1.11	0.0	0.55	30	4.00
									Total	229.6

Mile 2.01



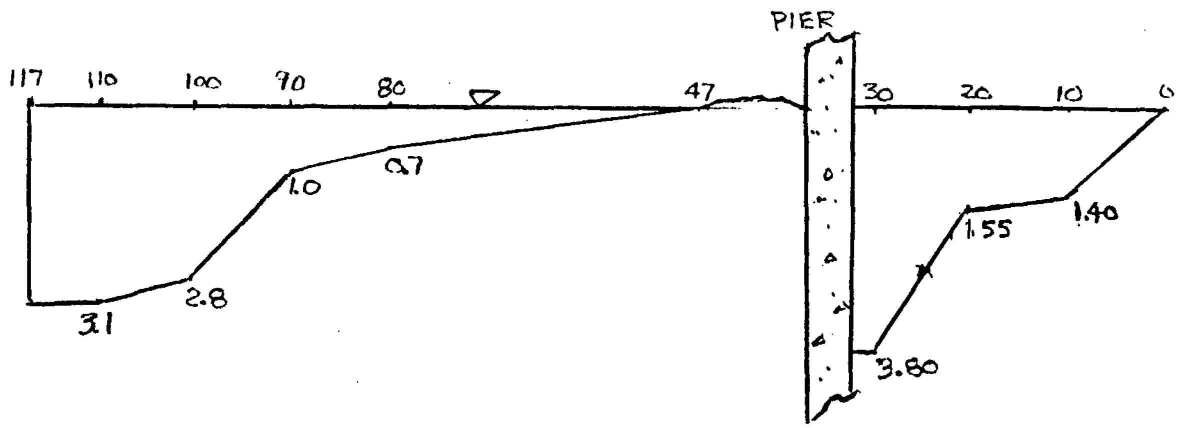
Location (1)	(2)	Depth D1 (ft)	D2 (ft)	Avg. Depth (ft)	Area (ft^2)	Velocity V1 (fps)	V2 (fps)	Avg. Velocity (fps)	Skew Angle (deg)	Corrected Discharge (cfs)
0	10	0.0	1.0	0.5	5.0	0.0	0.57	0.29	0	1.45
10	20	1.0	1.4	1.2	12.0	0.57	1.12	0.85	0	10.2
20	27.5	1.4	1.4	1.4	10.5	1.12	1.12	1.12	0	11.76
38.5	40	4.3	4.3	4.3	6.45	1.52	1.52	1.52	0	9.80
40	50	4.3	2.3	3.3	33	1.52	1.92	1.72	0	56.76
50	60	2.3	1.85	2.08	20.8	1.92	2.15	2.04	0	42.43
60	70	1.85	1.1	1.48	14.8	2.15	1.96	2.06	0	30.49
70	76	1.1	0.0	0.55	3.3	1.96	0.0	0.98	0	3.23
76	90	0.0	1.4	0.7	9.8	0.0	1.66	0.83	0	8.13
90	100	1.4	2.1	1.75	17.5	1.66	1.42	1.54	0	26.95
100	105	2.1	2.0	2.05	10.3	1.42	1.05	1.24	0	12.71
105	107	2.0	2.7	2.35	4.7	1.05	1.05	1.05	0	4.94
									Total	218.9

Mile 3.25



Location (1)	(2)	Depth D1 (ft)	D2 (ft)	Avg. Depth (ft)	Area (ft2)	Velocity V1 (fps)	V2 (fps)	Avg. Velocity (fps)	Skew Angle (deg)	Corrected Discharge (cfs)
-25	0	0	2.6	1.3	32.5	0.0	0.84	0.42	0	13.65
0	5	2.6	2.7	2.65	13.3	1.85	2.01	1.86	0	24.64
5	10	2.7	3.3	3.0	15.0	2.01	2.45	2.23	0	33.45
10	29	3.3	2.1	2.7	51.3	2.45	1.44	1.95	0	100.04
29	31	2.1	2.1	2.1	4.2	1.44	1.44	1.44	0	6.05
35	37	2.6	2.6	2.6	5.2	1.79	1.79	1.79	0	9.31
37	50	2.6	2.4	2.5	32.5	1.79	0.79	1.29	0	41.92
50	60	2.4	2.6	2.5	25.0	0.79	0.83	0.81	0	20.25
60	70	2.6	1.8	2.2	22.0	0.83	0.49	0.66	0	14.52
70	74	1.8	1.8	1.8	7.2	0.49	0.49	0.49	0	3.53
									Total	267.36

Mile 5.58



Location (1)	(2)	Depth D1 (ft)	D2 (ft)	Avg. Depth (ft)	Area (ft^2)	Velocity V1 (fps)	V2 (fps)	Avg. Velocity (fps)	Skew Angle (deg)	Corrected Discharge (cfs)
0	10	0.0	1.4	0.7	7.0	0.0	2.39	1.2	20	7.89
10	20	1.4	1.55	1.48	14.8	2.39	2.61	2.5	20	34.77
20	30	1.55	3.80	2.68	26.8	2.61	2.17	2.39	20	60.19
30	32	3.80	3.80	3.80	7.6	2.17	2.17	2.17	0	16.49
47	80	0.0	0.7	0.35	11.6	0.0	1.53	0.77	30	7.70
80	90	0.7	1.0	0.85	8.5	1.53	1.74	1.64	30	12.07
90	100	1.0	2.8	1.9	19.0	1.74	1.86	1.80	30	29.62
100	110	2.8	3.1	2.95	29.5	1.86	1.54	1.70	0	50.15
110	117	3.1	3.1	3.1	18.5	1.54	1.54	1.54	0	28.41
									Total	247.3

APPENDIX B: INFORMATION ON THREE GAGING STATIONS NEAR AMES

Identification Number: 05-4700.00

Title: South Skunk River Near Ames, Iowa

(Prior to October 1966, published as Skunk River near Ames)

Location: Latitude 42° 04' 05", Longitude 93° 37' 02"; in NW 1/4 of SW 1/4 of Section 23, T. 84 N., R. 24 W., Story County; Hydrologic Unit 07080105; on left bank (looking downstream), 2.5 miles (4.0 km) north of Ames, 3.5 miles (5.6 km) downstream from Keigley Branch, 5.2 miles (8.4 km) upstream from Squaw Creek, and at mile 228.1 (367.0 km) upstream from the mouth of the Skunk River.

Drainage Area: 315 square miles (816 sq. km)

Discharge History: (Based on 57 years of data to September 1982)

Period of Record; July 1920 to September 1927, October 1932 to current year.

Median Discharge; 120 cfs, 5.20 inches/year

Average Discharge; 151 cfs, 6.51 inches/year

Maximum Discharge; 8630 cfs, June 10,1954

Minimum Discharge; No flow many years, 1934, 1937, 1953-1957, and 1977

Identification Number: 05-4705.00

Title: Squaw Creek at Ames, Iowa

Location: Latitude 42° 01' 21", Longitude 93° 37' 45"; in NE 1/4 of NW 1/4 of Section 10, T. 83 N., R. 24 W., Story County; Hydrologic Unit 07080105; on the left bank (looking downstream), 65 feet (180 m) downstream from bridge on Lincoln Way in Ames, 0.2 mile (0.3 km) downstream from College Creek (Revised 1982), and 2.4 miles (3.9 km) upstream from its mouth (Revised 1982).

Drainage Area = 204 square miles (528 sq. km)

Discharge History: (Based on 25 years of data to September 1982)

Period of record; May 1919 to September 1927, May 1965 to current year

Average Discharge; 118 cfs, 7.86 inches/year

Median Discharge; 95 cfs, 6.30 inches/year

Maximum Discharge; 11,300 cfs, June 27, 1975

Minimum Discharge; No flow during most years

Identification Number: 05-4710.00

Title: South Skunk River Below Squaw Creek Near Ames, Iowa

(Prior to October 1966, published as Skunk River Below Squaw Creek near Ames)

Location: Latitude 42° 00' 31", Longitude 93° 35' 37"; in NE 1/4 of NW 1/4 of Section 13, T. 83 N., R. 24 W., Story County; on the right bank (looking downstream) 15 feet (5 m) from bridge on county highway (South Sixteenth Street) bridge, 0.2 mile (0.3 km) downstream from Squaw Creek, 0.2 mile (0.3 km) upstream from bridge on U.S. Highway 30, 2 miles (3.2 km) SE of Ames, and at mile 222.6 (358.2 km) from the mouth of the Skunk River.

Drainage Area: 556 square miles (1440 sq. km)

Discharge History (Based on 27 years of data to September 1979)

Period of Record; October 1952 to September 1979 when discontinued

Average Discharge; 301 cfs, 7.35 inches/year

Maximum Discharge 14,700 cfs, June 27, 1975

Minimum Discharge; No flow many years, 1934, 1937, 1953-1957, and 1977

APPENDIX C: INFORMATION ON 1982 AMES WASTELOAD ALLOCATION

DATE: January 29, 1982

TO: Tom Newman

FROM: Ralph Turkle

RE: Ames WLA Data

Enclosed is Chapter 4 of the Basin Plan Support Document and current USGS gaging data for the gage stations in the Ames area. Additionally, the following are the site specific values used as modeling input for the river reach at Ames.

1. Stream width above Ames discharge - 8', and stream width below Ames discharge - 28'.

2. Stream bed slope above and below Ames discharge = 0.000676 ft/ft.

3. Roughness coefficient is 0.05 for this sandy braided stream bed.

4. Tsivoglou gas escape coefficient = 0.115 1/ft.

5. Average winter stream temperature for first reach below Ames (1.8 miles) is 4°C. For the second reach below Ames (5.6 miles) winter temperature is 2°C.

   Average summer stream temperature for first and second reach is 26°C.

6. Background stream flow is lost at a rate of 0.06 cfs/mile.

7. Modeled stream values prior to Ames discharge.

Parameters	Winter	Summer
Flow	1.14 cfs	1.14 cfs
NH_3-N	3.6 mg/l	0.65 mg/l
BODu	26.9 mg/l	17.2 mg/l
D.O.	6.39 mg/l	6.60 mg/l
D.O. Saturation	13.9 mg/l @ 2°C	8.0 mg/l @ 26°C
Temperature	2°C	26°C

Memo to Tom Newman
From Ralph Turkle
January 29, 1982
Page 2

Parameters	Winter	Summer
% Ice Cover	90%	0%
Stream Velocity	0.5 ft/sec	0.5 ft/sec
Ave. water Depth	0.3 ft	0.3 ft
Reaeration Rate Constraint	0.215 1/day @ $2^{o}C$	3.812 1/day @ $26^{o}C$
Carbonaceous Deoxygenation Rate Constraint	0.087 1/day @ $2^{o}C$	0.263 1/day @ $26^{o}C$
Nitrogenous Deoxygenation Rate Constraint	0.0 at $2^{o}C$	0.404 1/day @ $26^{o}C$

8. Ames discharge characteristics: Q = 9.9 mgd; D.O. (summer) = 5.0 mg/l; D.O. (winter) = 6.0 mg/l.

9. Continuous discharges upstream of Ames.

Facility	Stream Miles to Ames	Discharge Flow	Comments
Story City	15.5	0.83 mgd	Domestic Waste
Ames Power Plant	2.8	0.043 mgd	Cooling H_2O
Sunstrand	2.8	0.015 mgd	Cooling H_2O
Gilbert	13.3	0.20 mgd	Domestic Waste

Please feel free to contact us if any question arise.

RT:bsb/OCWO28J10.02

APPENDIX D: PROGRAM SOURCE CODE FOR THE JRB MODEL

```
C REVISED IOWA MODEL   JWB ASSOCIATES  MARCH 1983
C EXISTING IOWA MODEL MODIFIED TO INCLUDE:
C   1. NEW TEMP. CORRECTION FUNCTION FOR KN
C   2. NEW TEMP. CORRECTION FUNCTION FOR R
C   3. PREFERENTIAL UPTAKE OF AMMONIA
C   4. CORRECTION FUNCTION FOR KN AT LOW DO LEVELS
C   5. ADDITION OF (P-R) TERM
C VELOCITIES CALCULATED BY LEOPOLD-MADDOCK EQUATION
      REAL K(35),KN(35),LEN(35),NOD(20),MILE(20),NH3(20),ICE(35),NF,KMN
      DIMENSION R20(35),R(35),TEM(35),CQ(35),TDO(35),TBOD(35),TNH3(35),
     1S(35),TQ(35),Q(35,20),DOSAT(35),DT(35),DLEN(35),BOD(20),DO(35),
     1D(20),DAY(20),C(35),SL(35),ITITLE(17),JTITLE(35,4),
     1ALPHA(35),BETA(35),TRIB(35,4),P(35),RESP(35),DELTA(35),CHLA(35),
     1CKN(35),GP(35),DP(35)
      DOUBLE PRECISION DATE
      DO 99 I=1,35
      DO 99 J=1,4
   99 TRIB(I,J)=0.
    1 READ(2,2,END=500)N,NF,ANP,KMN,(ITITLE(I),I=1,12)
    2 FORMAT(I8,3F8.0,12A4)
      NS=N/100
      N=N-(NS*100)
      IF(N) 3,500,3
    3 IF((N.LE.35).AND.(N.GE.1)) GO TO 5
      WRITE(3,4) N
      PRINT 4,N
    4 FORMAT(' N OF ',I3,' OUTSIDE RANGE OF 1 TO 25',/,
     1' RUN TERMINATED')
      GO TO 500
    5 READ(2,6) Q(1,1),DO(1),BOD(1),NH3(1),TEM(1),CDO,CBOD,CNH3,AP,OP,
     1(LEN(I),S(I),K(I),KN(I),ALPHA(I),BETA(I),SL(I),C(I),GP(I),DP(I),
     1TDO(I),TBOD(I),TNH3(I),TQ(I),TEM(I),CQ(I),ICE(I),CHLA(I),
     1(JTITLE(I,J),J=1,4),I=2,N)
    6 FORMAT(10F8.0,/,(10F8.0,/,8F8.0,4A4))
      DO 44 I=2,N
      IF((S(I).LE.19.).AND.(S(I).GE.1.)) GO TO 44
      WRITE(3,45) I,S(I)
      PRINT 45, I,S(I)
   45 FORMAT(' S(',I3,') = ',F4.0,' OUTSIDE OF RANGE 1 TO 19',/,
     1' RUN TERMINATED')
      GO TO 500
   44 CONTINUE
      IF(NS.EQ.0) GO TO 88
      GO TO 91
   88 DO 89 I=1,N
      IF(TRIB(I,4).NE.0) GO TO 90
      GO TO 89
   90 TDO(I)=TRIB(I,1)
      TBOD(I)=TRIB(I,2)
      TNH3(I)=TRIB(I,3)
      TQ(I)=TRIB(I,4)
   89 CONTINUE
   91 CONTINUE
      WRITE(3,7) (ITITLE(I),I=1,10)
      PRINT 7, (ITITLE(I), I=1,10)
    7 FORMAT(//,5X,10A4,/,4X,'THE INPUT DATA ARE')
      PRINT 8, N,Q(1,1),DO(1),BOD(1),NH3(1),TEM(1),CDO,CBOD,CNH3,NF,ANP,
     1KMN,AP,OP
      WRITE(3,8)N,Q(1,1),DO(1),BOD(1),NH3(1),TEM(1),CDO,CBOD,CNH3,NF,
     1ANP,KMN,AP,OP
    8 FORMAT(' N = ',I3,'  Q(1,1) =',F7.2,'  DO(1,1) =',F6.2,
     1'  BOD(1,1) =',F6.2,'  NH3(1,1) =',F6.2,'  TEM(1) =',F6.2,
     1'  CDO =',F6.2,/,' CBOD =',F6.2,' CNH3 =',F6.2,' NF =',F6.2,
     1' ANP =',F9.5,'   KMN=',F6.2,'   AP=',F6.2,'   OP=',F6.2)
```

```
      WRITE(3,9)
      PRINT 9
    9 FORMAT('  I LEN(I)  S(I) K20(I) KN20(I) A(I) B(I)  SL(I)    C(I)',
     1'   CHLA(I) TDO(I) TBOD(I) TNH3(I)  TQ(I)  TEM(I) CQ(I)  '
     1,'   REACH')
      DO 10 I=2,N
      PRINT 11, I,LEN(I),S(I),K(I),KN(I),ALPHA(I),BETA(I),SL(I),C(I),
     1CHLA(I),TDO(I),TBOD(I),TNH3(I),TQ(I),TEM(I),CQ(I),
     1(JTITLE(I,J),J=1,4)
   10 WRITE(3,11) I,LEN(I),S(I),K(I),KN(I),ALPHA(I),BETA(I),SL(I),C(I),
     1CHLA(I),TDO(I),TBOD(I),TNH3(I),TQ(I),TEM(I),CQ(I),
     1(JTITLE(I,J),J=1,4)
   11 FORMAT(1X,I2,F7.2,F6.2,F7.2,F6.2,F7.2,F5.2,F9.6,F6.2,F10.2,
     1F7.2,2F8.2,F7.2,F8.2,F7.2,4A4)
      WRITE(3,12)
      PRINT 12
   12 FORMAT(/,' VALUES FOR EACH REACH ARE ',/,'  I',6X,'FPS',9X,
     1'R20',7X,'R',9X,'K',8X,'KN',5X,'DOSAT',3X,'ICE',6X,'P',6X,'RESP',
     15X,'G-RATE',4X,'D-RATE')
      M=1
      DO 17 I=2,N
      Q(I,1)=TQ(I)+Q(I-1,M)
      DLEN(I)=LEN(I)/S(I)
      DQ=DLEN(I)*CQ(I)
      M=S(I)+1
      DO 13 J=2,M
   13 Q(I,J)=Q(I,J-1)+DQ
      QM=(Q(I,1)+Q(I,M))/2.
      FPS=ALPHA(I)*(QM**BETA(I))
      DH=5280.*LEN(I)*SL(I)
      V=16.37*FPS
      TIME=LEN(I)/V
      R20(I)=ICE(I)*C(I)*DH/TIME
      DT(I)=DLEN(I)/V
      R(I)=R20(I)*(1.0159**(TEM(I)-20.))
      K(I)=K(I)*(1.047**(TEM(I)-20.))
      KN(I)=KN(I)*1.080**(TEM(I)-20.0)
      GP(I)=GP(I)*1.047**(TEM(I)-20.0)
      P(I)=OP*(GP(I)-DP(I))*CHLA(I)/AP
      RESP(I)=0.025*CHLA(I)
      IF(KN(I)) 14,15,15
   14 KN(I)=0.
   15 TEMF=32.0+1.8*TEM(I)
      DOSAT(I)=24.89-0.426*TEMF+.00373*(TEMF**2.)-0.0000133*(TEMF**3.)
      PRINT 16, I,FPS,R20(I),R(I),K(I),KN(I),DOSAT(I),ICE(I),P(I),
     1RESP(I),GP(I),DP(I)
      WRITE(3,16) I,FPS,R20(I),R(I),K(I),KN(I),DOSAT(I),ICE(I),P(I),
     1RESP(I),GP(I),DP(I)
   16 FORMAT(1X,I2,5F10.3,F9.3,F7.3,2F8.2,2F10.2)
   17 CONTINUE
      WRITE(3,18)
      PRINT 18
   18 FORMAT(/,'   I   J     MILE',3X,'Q(CFS)',3X,'DAYS',7X,'CKN',4X,
     1'NH3-N',3X,'BOD-U',5X,'DO',6X,'D')
      M=1
      DO 23 I=2,N
      MILE(1)=MILE(M)
      DAY(1)=DAY(M)
      NH3(1)=(TNH3(I)*TQ(I)+NH3(M)*Q(I-1,M))/Q(I,1)
      BOD(1)=(TBOD(I)*TQ(I)+BOD(M)*Q(I-1,M))/Q(I,1)
      DO(1)=(TDO(I)*TQ(I)+DO(M)*Q(I-1,M))/Q(I,1)
      CKN(1)=KN(I)*(1.0-EXP(-0.52*DO(1)))
      D(1)=DOSAT(I)-DO(1)
```

```
      M=S(I)+1
      CD=DOSAT(I)-CDO
      DQ=DLEN(I)*CQ(I)
      J=1
      PRINT 19, I,J,MILE(1),Q(I,1),DAY(1),CKN(1),NH3(1),BOD(1),DO(1),
     1D(1),(JTITLE(I,KK),KK=1,4)
      WRITE(3,19) I,J,MILE(1),Q(I,1),DAY(1),CKN(1),NH3(1),BOD(1),DO(1),
     1D(1),(JTITLE(I,KK),KK=1,4)
   19 FORMAT(2X,2(I2,2X),F6.2,2X,F7.2,2X,F6.3,4X,F6.2,4(2X,F6.2),2X,4A4)
      DO 23 J=2,M
      MILE(J)=MILE(J-1)+DLEN(I)
      DAY(J)=DAY(J-1)+DT(I)
      NOD(J-1)=NH3(J-1)*4.33
      PN=1.0-EXP(-0.52*DO(J-1))
      CKN(J)=KN(I)*PN
      A=K(I)*BOD(J-1)/(R(I)-K(I))
      B=CKN(J)*NOD(J-1)/(R(I)-CKN(J))
      BOD(J)=BOD(J-1)*EXP(-K(I)*DT(I))
      NOD(J)=NOD(J-1)*EXP(-CKN(J)*DT(I))
      D(J)=A*(EXP(-K(I)*DT(I))-EXP(-R(I)*DT(I)))+B*(EXP(-CKN(J)*DT(I))-
     1EXP(-R(I)*DT(I)))+D(J-1)*EXP(-R(I)*DT(I))+(RESP(I)*(1.0-EXP(
     1-R(I)*DT(I))))/R(I)-(P(I)*(1.0-EXP(-R(I)*DT(I))))/R(I)
      IF(D(J).LE.DOSAT(I)) GO TO 33
      D(J)=D(J-1)*EXP(-R(I)*DT(I))
      AF=BOD(J-1)-BOD(J)
      BF=NOD(J-1)-NOD(J)
      BOD(J)=BOD(J-1)-(D(J-1)-D(J))*AF/(AF+BF)
      NOD(J)=NOD(J-1)-(D(J-1)-D(J))*BF/(AF+BF)
      D(J)=DOSAT(I)
   33 CONTINUE
      IF(CQ(I).LE.0.) GO TO 20
      D(J)=(D(J)*Q(I,J-1)+CD*DQ)/Q(I,J)
      BOD(J)=(BOD(J)*Q(I,J-1)+CBOD*DQ)/Q(I,J)
   20 NH3(J)=((NOD(J)/4.33-GP(I)*ANP*NF*CHLA(I)/(GP(I)-DP(I)+KN(I))*(
     1EXP((GP(I)-DP(I))*DT(I))-EXP(-KN(I)*DT(I)))))*Q(I,J-1)
     1+CNH3*DQ)/Q(I,J)
      IF(NH3(J).LT.KMN) NH3(J)=(NOD(J)/4.33*Q(I,J-1)+CNH3*DQ)/Q(I,J)
      DO(J)=DOSAT(I)-D(J)
      IF(DO(J)) 21,22,22
   21 DO(J)=0.
   22 WRITE(3,24) I,J,MILE(J),Q(I,J),DAY(J),CKN(J),NH3(J),BOD(J),DO(J),
     1D(J)
   23 PRINT 24, I,J,MILE(J),Q(I,J),DAY(J),CKN(J),NH3(J),BOD(J),DO(J),
     1D(J)
   24 FORMAT(2X,2(I2,2X),F6.2,2X,F7.2,2X,F6.3,4X,F6.2,4(2X,F6.2))
      IF(NS.NE.0) GO TO 66
      GO TO 67
   66 TRIB(NS,1)=DO(J)
      TRIB(NS,2)=BOD(J)
      TRIB(NS,3)=NH3(J)
      TRIB(NS,4)=Q(I,J)
   67 CONTINUE
      GO TO 79
   77 DO 79 I=1,35
      DO 78 J=1,4
   78 TRIB(I,J)=0.
   79 CONTINUE
      GO TO 1
  500 STOP
      END
```

APPENDIX E: PROGRAM SOURCE CODE FOR THE "STANLEY" MODEL

```
200     K1=0.3/KN=0.3/K2=0.25/DO=0/LO=40/NO=80/V=0.5/X=1/N=15
210     PRINT "ENTER C, P, OR Q"/INPUT R$
215     PRINT
220     IF R$="C" THEN 1000
230     IF R$="P" THEN 2000
240     IF R$="Q" THEN 9999
250     PRINT "TYPE C, P, OR Q"/GOTO 190
1000    PRINT "PARAMETEER VALUES"
1010    PRINT "-----------------"
1020    PRINT "K1 = ";K1;" 1/DAYS"
1030    PRINT "K2 = ";K2;" 1/DAYS"
1040    PRINT "KN = ";KN;" 1/DAYS"
1050    PRINT "DO = ";DO;" MG/L"
1060    PRINT "LO = ";LO;" MG/L"
1070    PRINT "NO = ";NO;" MG/L"
1080    PRINT " V = ";V;" M/S"
1090    PRINT " X = ";X;"  KM"
1100    PRINT " N = ";N
1110    PRINT/PRINT
1120    PRINT "CHANGES:"
1130    PRINT "ENTER R,O TO EXIT"/INPUT C$,C
1140    IF C$="K1" THEN K1=C/GOTO 1000
1150    IF C$="K2" THEN K2=C/GOTO 1000
1160    IF C$="KN" THEN KN=C/GOTO 1000
1170    IF C$="DO" THEN DO=C/GOTO 1000
1180    IF C$="LO" THEN LO=C/GOTO 1000
1190    IF C$="NO" THEN NO=C/GOTO 1000
1200    IF C$="V" THEN V=C/GOTO 1000
1210    IF C$="X" THEN X=C/GOTO 1000
1220    IF C$="N" THEN N=C/GOTO 1000
1230    IF C$="R" THEN GOTO 210
1240    PRINT "ENTER PARAM,VALUE"/GOTO 1000
2000    PRINT "DIST, KM","DEFICIT, MG/L"
2010    PRINT "--------","-------------"
2020    IF K1=K2 THEN K2=K1-0.00001
2030    IF KN=K2 THEN KN=K2+0.00001
2040    F1=K1*LO/(K2-K1)
2050    F3=KN*NO/(K2-KN)
2060    TI=X/(V*86.4)
2070    FOR I=0 TO N
2080    T=I*TI
2090    F2=EXP(-1*K1*T)-EXP(-1*K2*T)
2100    F4=EXP(-1*KN*T)-EXP(-1*K2*T)
2110    F5=DO*EXP(-1*K2*T)
2120    F=F1*F2+F3*F4+F5
2130    PRINT X*I,F
2140    NEXT I
2150    PRINT/PRINT/GOTO 210
9999    END
```

APPENDIX F: DISCHARGE DATA FOR THE THREE GAGING STATIONS NEAR AMES

Table F1. Discharge data for the three gaging stations near Ames[a]

		Monthly Average Discharge In Ft^3/sec			
Year	Month	Hallett's Quarry[b]	Squaw Creek[c]	Combined Discharge[d]	South 16th Street[e]
65	6	340	238	578	652
	7	62	17.8	79.8	94.8
	8	7.21	2.39	9.6	10.6
	9	539	144	683	689
	10	246	87.3	333.3	380
	11	120	44.9	164.9	196
	12	221	75.5	296.5	355
66	1	130	73.6	203.6	225
	2	131	64.6	195.6	191
	3	171	108	279	298
	4	158	84.6	242.6	280
	5	322	205	527	547
	6	402	353	755	847
	7	53.0	40.1	93.1	105
	8	14.6	3.37	17.97	21.2

[a]Sources (USGS, 1965 to 1982) and preliminary data for 1983.

[b]Gaging station near Hallet's Quarry above Ames, Iowa Geological Survey (IGS) Identification Number 05-4700.00.

[c]Gaging station on Squaw Creek at Ames, IGS # 05-4705.00.

[d]Combined discharge data from Hallett's Quarry and the Squaw Creek gaging stations.

[e]Gaging station near South Sixteenth Street bridge at Ames, IGS # 05-4710.00.

Table F1. continued

		Monthly Average Discharge In Ft^3/sec			
Year	Month	Hallett's Quarry	Squaw Creek	Combined Discharge	South 16th Street
66	9	2.92	0.46	3.38	3.41
	10	1.64	0.51	2.15	0.49
	11	3.11	0.63	3.74	1.07
	12	2.19	0.30	2.49	0.49
67	1	3.14	0.69	3.83	1.79
	2	3.79	0.57	4.36	0.10
	3	20.1	5.53	25.63	13.0
	4	11.2	5.42	16.62	13.1
	5	6.64	3.61	10.25	6.71
	6	850	504	1354	1383
	7	74.2	40.2	114.4	123
	8	25.6	9.89	35.49	35.9
	9	3.38	1.37	4.75	4.24
	10	4.55	1.89	6.44	4.31
	11	5.62	1.93	7.55	4.25
	12	5.18	1.63	6.81	2.88
68	1	2.60	0.72	3.32	1.09
	2	4.13	0.79	4.92	2.21
	3	20.6	6.15	26.75	16.2
	4	48.5	35.3	83.8	70.8
	5	30.6	23.3	53.9	47.9
	6	349	244	593	640
	7	143	64.8	207.8	214
	8	27.6	11.7	39.3	41.3
	9	15.3	12.0	27.3	24.2
	10	83.9	67.2	145.1	127
	11	60.7	47.4	108.1	95.6
	12	43.1	32.9	76	73.6
69	1	29.0	19.7	48.7	40.6
	2	28.1	23.0	51.1	44.9
	3	767	585	1352	1268
	4	363	256	619	595
	5	315	227	542	553
	6	711	531	1242	1270
	7	1430	648	2078	2138
	8	224	47.4	271.4	298
	9	59.3	35.6	94.9	116

Table F1. continued

		Monthly Average Discharge In Ft^3/sec			
Year	Month	Hallett's Quarry	Squaw Creek	Combined Discharge	South 16th Street
69	10	36.6	29.1	65.7	64.5
	11	72.1	50.2	122.3	125
	12	29.7	20.5	50.2	52.0
70	1	12.8	9.15	21.95	19.7
	2	50.9	49.0	99.9	98.9
	3	147	111	258	272
	4	118	94.8	212.8	220
	5	158	297	455	540
	6	94.9	107	201.9	222
	7	26.2	22.8	49.0	51.4
	8	65.5	49.2	114.7	135
	9	32.4	39.6	72	86.3
	10	173	95.8	268.8	288
	11	206	107	313	315
	12	101	61.2	162.2	164
71	1	36.4	23.0	59.4	57.6
	2	410	381	791	851
	3	576	376	952	959
	4	144	84.9	228.9	250
	5	122	88.8	210.8	229
	6	84.7	47.4	132.1	146
	7	133	45.4	178.4	191
	8	7.77	1.60	9.37	12.1
	9	2.85	0.071	2.921	3.16
	10	4.85	3.93	8.78	4.34
	11	16.2	4.31	20.51	19.3
	12	11.6	2.97	14.57	10.7
72	1	4.76	0.70	5.46	1.84
	2	3.93	4.71	8.64	15.1
	3	280	131	411	277
	4	57.0	29.1	86.1	68.0
	5	191	105	296	305
	6	378	189	567	601
	7	112	39.9	151.9	152
	8	600	261	861	891
	9	260	126	386	416
	10	457	203	660	707
	11	726	491	1217	1270
	12	292	197	489	426

Table F1. continued

Year	Month	Hallett's Quarry	Squaw Creek	Combined Discharge	South 16th Street
		Monthly Average Discharge In Ft^3/sec			
73	1	315	275	590	599
	2	534	465	999	919
	3	821	526	1347	1313
	4	867	682	1549	1546
	5	693	518	1211	1293
	6	385	324	709	723
	7	179	135	314	331
	8	62.7	57.2	119.9	107
	9	349	289	638	578
	10	653	505	1158	1079
	11	234	179	413	472
	12	203	177	380	384
74	1	127	151	278	318
	2	199	203	402	452
	3	397	311	708	816
	4	463	330	793	888
	5	747	604	1351	1421
	6	1243	691	1934	1910
	7	242	113	355	345
	8	234	49.5	283.5	299
	9	26.1	11.3	37.4	53.7
	10	66.7	33.2	99.9	105
	11	238	82.8	320.8	294
	12	102	57.0	159.0	187
75	1	62.8	34.1	96.9	144
	2	35.1	24.9	60.0	85.5
	3	432	309	741	790
	4	624	434	1058	1027
	5	303	227	530	504
	6	1189	1107	2296	2304
	7	304	227	531	474
	8	40.3	21.4	61.7	62.6
	9	20.6	4.80	25.40	26.4
	10	7.54	2.18	9.72	7.81
	11	18.0	7.77	25.77	20.0
	12	23.6	8.54	32.14	33.6

Table F1. continued

		Monthly Average Discharge In Ft^3/sec			
Year	Month	Hallett's Quarry	Squaw Creek	Combined Discharge	South 16th Street
76	1	1.98	1.07	3.05	3.15
	2	8.28	9.79	18.02	17.5
	3	215	124	339	296
	4	438	295	733	828
	5	324	225	549	560
	6	519	295	814	759
	7	55.9	30.4	86.3	96.6
	8	2.57	1.68	4.25	8.09
	9	0.081	0.23	0.311	0.16
	10	0.76	1.12	1.88	0.090
	11	1.48	2.90	4.38	0.005
	12	0.00	0.001	0.001	0.003
77	1	0.00	0.00	0.00	0.00
	2	1.72	0.093	1.813	0.00
	3	23.1	5.77	28.87	11.9
	4	16.6	4.33	20.93	14.4
	5	7.30	5.95	13.25	8.1
	6	0.011	2.87	2.981	0.00
	7	0.017	4.39	4.41	2.45
	8	687	231	918	908
	9	310	92.9	402.9	449
	10	267	166	433	457
	11	152	97.6	249.6	269
	12	98	52.3	150.3	156
78	1	36.3	15.1	51.4	57.1
	2	17.4	7.41	24.81	32.1
	3	127	142	269	383
	4	421	353	774	818
	5	211	166	377	353
	6	239	230	469	418
	7	132	146	278	317
	8	69.1	81.2	150.3	236
	9	484	446	930	988
	10	140	92.5	232.5	236
	11	138	117	255	243
	12	72.8	56.0	128.8	132

Table F1. continued

		Monthly Average Discharge In Ft^3/sec			
Year	Month	Hallett's Quarry	Squaw Creek	Combined Discharge	South 16th Street
79	1	23	17.6	40.6	39.3
	2	20.7	15.1	35.8	37.6
	3	1034	777	1811	2026
	4	531	387	918	1022
	5	320	260	580	622
	6	455	320	775	855
	7	399	204	603	691
	8	332	241	573	678
	9	114	36.3	150.3	183
	10	118	71.8	189.8	199[f]
	11	217	116	333	350
	12	93.2	52.8	146	153
80	1	98.9	72.5	171.4	180
	2	75.5	57.5	133.2	140
	3	190	132	322	338
	4	179	100	279	293
	5	63.6	75.9	139.5	146
	6	561	282	843	885
	7	45.5	28.5	74.0	77.7
	8	19.9	15.8	35.7	36.2
	9	6.63	4.03	10.66	7.2
	10	2.12	1.35	3.47	1.2
	11	3.85	4.10	7.95	4.7
	12	3.44	5.35	8.79	5.42
81	1	0.84	0.54	1.38	0.1
	2	14.4	11.1	25.5	23.4
	3	6.35	2.51	8.86	5.5
	4	24.6	7.77	32.37	31.9
	5	7.15	1.42	8.57	5.2
	6	186	88.4	274.4	288.1
	7	45.3	29.9	75.2	79.0
	8	15.4	9.98	25.38	23.3

[f]Predicted values for the South Sixteenth Street bridge, for this month and all values hereafter, obtained from the equations developed in the discharge section.

Table F1. continued

		Monthly Average Discharge In Ft^3/sec			
Year	Month	Hallett's Quarry	Squaw Creek	Combined Discharge	South 16th Street
81	9	10.4	2.60	13.00	9.5
	10	12.5	2.12	14.62	11.1
	11	17.4	5.71	23.11	20.6
	12	15.7	7.74	23.44	21.0
82	1	5.53	2.62	8.15	4.9
	2	254	151	405	425
	3	574	325	899	944
	4	222	172	394	414
	5	591	531	1122	1178
	6	438	367	805	845
	7	567	430	997	1047
	8	71.6	47.0	118.6	124.5
	9	69.0	33.2	102.2	107.3
	10[g]	117	42.1	159.1	167.1
	11	270	133	403	423
	12	548	193	741	778
83	1	260	196	456	479
	2	596	438	1034	1086
	3	578	373	951	999
	4	1081	757	1838	1930
	5	748	514	1262	1325
	6	470	420	890	934
	7	730	366	1096	1151
	8	84.5	22.7	107.2	112.6
	9	135	24.2	159.2	167.2
	10	259	108	367	385
	11	494	335	82.9	87.0

[g] Discharge data obtained from preliminary data obtained from IGS, for this month and all months hereafter.