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PROCEDURE FOR ESTIMATING THE WASTE

ASSIMILATION CAPACITY OF A RIVER SYSTEM

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF ILLUSTRATIONS

Figure		Page

PROCEDURE FOR ESTIMATING THE WASTE
ASSIMILATION CAPACITY OF A RIVER SYSTEM

CHAPTER I

INTRODUCTION

Little thought was given to river quality conditions in the 19th century. The only design criteria developed in that period was the empirical determination that for every population equivalent discharging to the river a corresponding river flow was needed to keep the river from becoming black and odorous. Perhaps this was adequate for those early times because it was generally a period of relatively small and scattered cities, and the pollutional load from one city seldom greatly affected the river quality or condition when it reached another.

However, beginning around the turn of the century, the increasing urban populations and the great industrial growths soon led to the deterioration of the general water quality of many rivers and brought about the need of stream quality analysis and the necessity of treating sewage and industrial wastes.

Much effort was expended on the development of stream quality analysis and prediction by the U.S. Public Health Service between the years 1915 and 1925, and this work was highlighted by the publication in 1925 of the report "A Study of The Pollution and Natural Purification of the Ohio River" by Streeter and Phelps.(1) Their analysis of the effects of the dissolved oxygen depleting properties of the organic material in sewage and the reaeration, or natural purification, of a river has been, since its publication, the corner stone of the study of stream pollution and stream quality.

The formulas presented by Streeter and Phelps were developed for the purpose of predicting the dissolved oxygen content of the river water as a function of the distance downstream from a point of continual pollutional load. The graph of the values obtained from these equations is known as the Oxygen Sag Curve or the Longitudinal Dissolved Oxygen Profile.

As cities have developed and grown and their waste loads increased, the stretches of rivers between the population centers have proved unable to assimilate or oxidize all the wastes from one city before receiving the wastes of the next one. This has required that the oxygen sag curves be integrated together and has led to the practice of considering as a single point loading the wastes which may be discharged at several points along a stream.

Although this method has worked well for the analysis of stream oxygen conditions in the past, the size and close proximity of present day and expected future pollutional loads has prompted an investigation into a new procedure of analysis which will forecast the ultimate capacity of a river to assimilate waste loadings.

Instead of applying a pollutional load to the river and determining a downstream oxygen profile, the new procedure entails assuming a permissible dissolved oxygen level throughout the river length and then determining the pollutional loading throughout its length which would cause such a condition. In other words for each small segment of the river, the pollutional load will be determined which would depress the oxygen level to the minimum allowed value. Thus when these values for each segment are summed up for an entire river, they will equal the total theoretical pollutional load which the river can assimilate. Since the difference between this allowable loading and any greater predicted loading must be taken up by treatment or increased river flow, these results will make it possible to determine future waste treatment requirements and/or to determine low flow augmentation practices.

The development of the new procedure will consist of three major parts: (1) the determination of the waste assimilation capacity of a river not affected by tides; (2) the determination of the waste assimilation capacity for the tidal zone of the river, and (3) an analysis to correct for non-continuous loading.

The procedure is specifically designed for use on digital computers, and computer programs have been written and utilized for the several applications which will demonstrate the use of the procedure.

The procedure developed in this dissertation is an outgrowth of a method used by Reid in estimating future water requirements for pollution abatement for the U.S. Senate Select Committee on Water Resources. (2) The Reid procedure was developed as a planning type technique for an entire river basin. His technique made no provision for the effect of tides in the tidal zone of the river and had as its primary purpose the determination of dilution water requirements at different sewage treatment levels in the basin.

The new procedure which will be developed herein is considerably different from the Reid procedure in that it will: (1) evaluate either a portion of a river, an entire river, or an entire system of rivers, (2) determine the waste assimilation capacity in terms of pounds of biochemical oxygen demand (BOD) per day, (3) use different computational techniques, (4) utilize much more extensive data, (5) evaluate the effect of tides and salinity in the tidal zone of the river, and (6) make use of a digital computer to carry out the required calculations.

Although some discussion will be presented concerning other pollutants than those requiring oxygen, the procedure outlined herein is directly applicable to only the analysis of oxygen demanding wastes.

In Chapters II and III the theory of the procedure will be discussed for the non-tidal and tidal zones respectively. Chapter IV will discuss the evaluation and modification of the various coefficients utilized in the procedure. Chapter V will include a discussion of the availability of existing stream flow data and its modification for use in the new procedure. Chapters VI and VII will in turn discuss the computer techniques and programs and the applications made to the Ohio River System and the Delaware River. Chapter VIII is the analysis of the effect of non-continuous pollution loading, and in Chapter IX the results will be summarized. The appendix includes computer program notation, flow charts, and computer programs.

CHAPTER II

THEORY - RIVER WITHOUT TIDAL ZONE

A river can be considered as a mass of impure flowing water. From its beginning to its end it is constantly changing. Water is constantly being added and withdrawn, and the multitude of impurities are likewise changing in quantity and form under the influence of the various forces of nature.

The good or harm of the various impurities and properties depends to a large extent on the viewpoint and needs of the beings involved. The majority of human beings would probably agree that they would like streams to be clear and cool, to contain the necessary impurities to support fish, wildlife, and plants, and to be free of the impurities which cause taste, color, and odor, and those impurities, both chemical and of living organisms, which endanger his health. The desirable relative balance of these various impurities must be arbitrarily set by man and is a function of his collective effort to maintain this balance.

From man's, and possibly nature's, point of view the most important impurity which suffers from depletion is the oxygen which is dissolved in water. Most of the wastes which are discharged by a population consist of material which ultimately will be oxidized in the water creating an oxygen deficit. Since fish and other aquatic life depend on this dissolved oxygen for their air supply, they are either killed or forced to leave the area if the dissolved oxygen level falls too low. Fortunately, the dissolved oxygen is replenished by the transfer of oxygen from the air at the stream surface and by other sources. The rates of oxygen use and replenishment are of great importance in stream analysis as will be shown later. Other pollutants such as nitrogenous material, organic chemicals, heat, radioactivity, bacteria, toxic materials, and individual inorganic chemicals are important especially in localized areas. However, since oxygen depletion is at present the controlling stream pollution problem, since it is probably the most difficult to analyze, and since waste treatment processes are primarily designed to reduce this pollutant, the procedures of this dissertation will deal only with this one pollutant. Occasionally some discussion will be directed to the other pollutants, especially to the so called "non-decaying pollutants" which are not assimilated or reduced by the river and therefore, continue to build up.

Derivation Of Equations

The basic equation of stream pollution analysis is Streeter and Phelps' differential equation for the oxygen sag curve:(1)

$$\frac{dD}{dt} = K_1 L - K_2 D \tag{2.1}$$

where D = oxygen deficiency or deficit (P.P.M. or pounds per unit volume)

L = Biochemical Oxygen Demand (BOD) (P.P.M. or pounds per unit volume)

K_1 = decomposition reaction coefficient (day^{-1})

K_2 = reoxygenation coefficient (day^{-1})

and t = time (days).

K_1 and K_2 are not constants but are coefficients dependent on several variables. A large part of Chapter IV will be devoted to their analysis.

In the form shown equation 2.1 is applicable only to a stretch of river with a uniform flow. Illustration 1 Part A shows equation 2.1 graphically for the case where pollutional load is applied at a single point. The resultant line between the deoxygenation and reoxygenation is the classical oxygen sag curve.

Under the concept of this dissertation a fixed minimum dissolved oxygen value called the River Quality Standard (RQS) is arbitrarily set. This RQS is usually set by water quality standards to protect fish and aquatic life. An acceptable value which is often used is four parts per million. The oxygen deficit thus will be the difference between the dissolved oxygen saturation value and the RQS.

$$D_{RQS} = D.O.(saturated) - RQS \tag{2.2}$$

For a segment of river with no temperature or salinity change this deficit will remain constant and therefore:

$$\frac{dD}{dt} = 0 = K_1L - K_2D_{RQS}$$

or

$$K_1L = K_2D_{RQS} \, . \tag{2.3}$$

Equation 2.3 states that the waste oxidation is in equilibrium

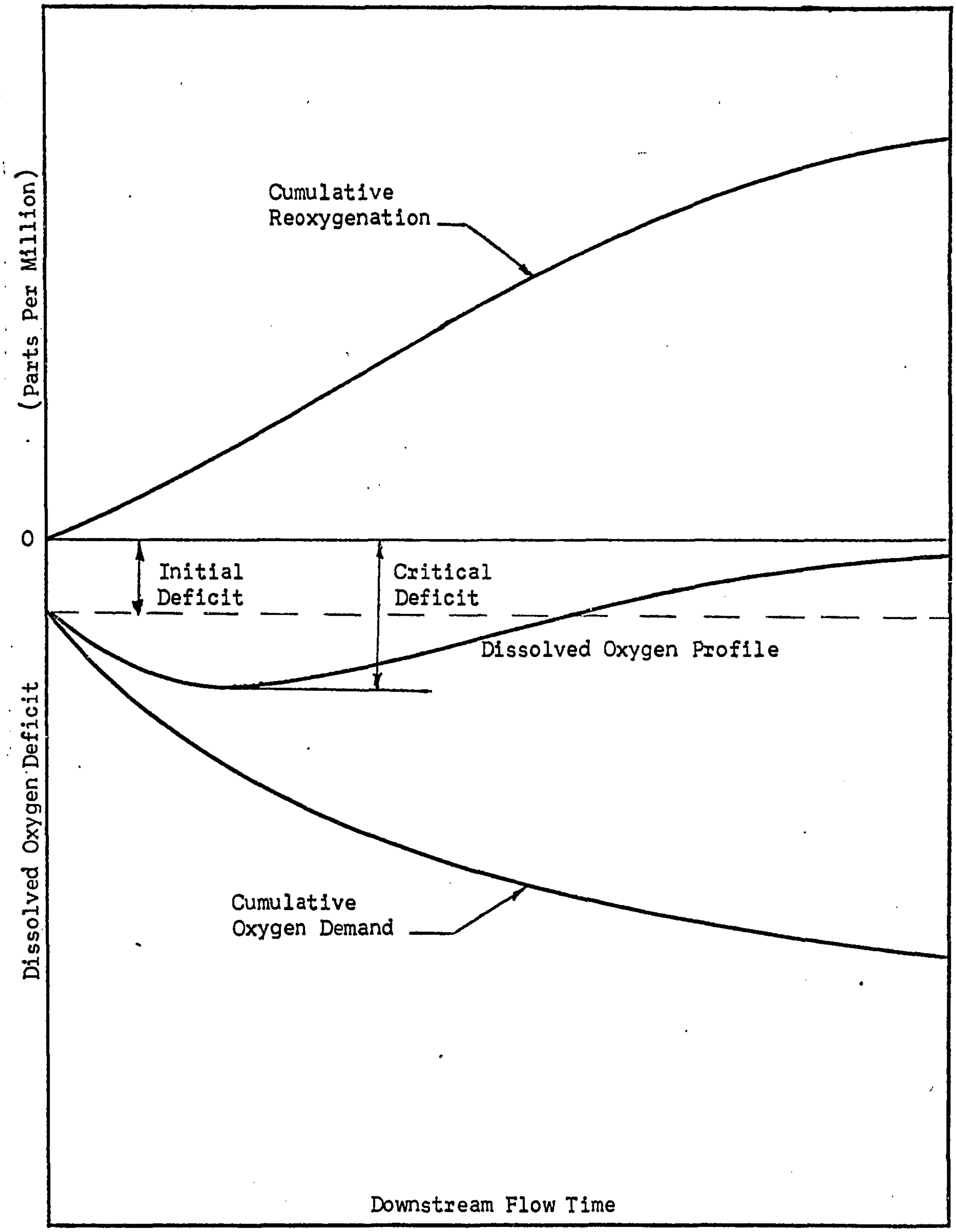


ILLUSTRATION 1 PART A

Oxygen Profile For A Point Waste Loading

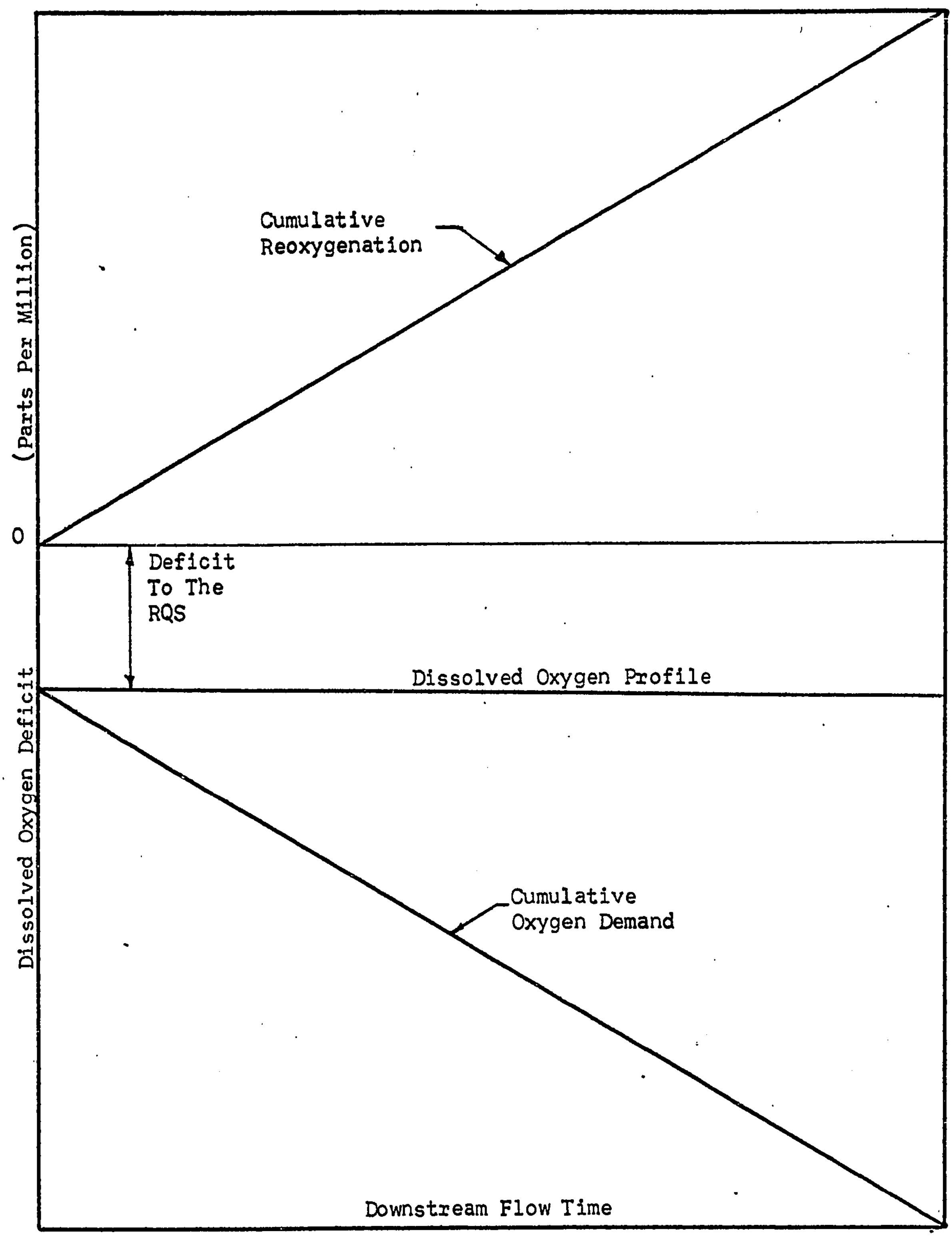


ILLUSTRATION 1 PART B

Oxygen Profile For A Continuous Waste Loading with the reoxygenation. L and D can be expressed in several sets of units, but the most common are milligrams per liter, parts per million, and pounds per unit volume of water. K_1 and K_2 usually represent a fractional change per day.

Illustration 1 Part B shows the case where the dissolved oxygen level is maintained at the RQS. Therefore, the cumulative reoxygenation equals the cumulative BOD loading.

Assuming L and D in units of pounds per unit volume and considering a segment with a volume of V units, equation 2.3 becomes:

$$K_1 LV = K_2 D_{RQS} V. \tag{2.4}$$

Letting $LV = L_{Seg}$ where L_{Seg} is the total BOD per day applied to the segment, equation 2.4 becomes:

$$K_1 L_{Seg} = K_2 D_{RQS} V$$

or

$$L_{Seg} = \frac{K_2 D_{RQS} V}{K_1}. \tag{2.5}$$

The volume V may be computed as either the mean cross section area times the segment length or as the mean flow times the passage time through the segment. Thus:

$$V = A_{mean}(\Delta L) = Q_{mean}(\Delta t). \tag{2.6}$$

The formulation of what happens in a river under the condition of uniform oxygen content can best be shown by considering a small longitudinal segment of the river. A balance of the river flows in and out of the segment as is shown below. These flow values are subscripted with their corresponding dissolved oxygen contents.

$$\text{Inflow}_{(RQS)} + \text{Added Flow}_{(\text{Any D.O.})} = \text{Outflow}_{(RQS)}. \tag{2.7}$$

An oxygen balance may be made for the segment in the same manner.

$$\text{Inflow Oxygen} + \text{Added Oxygen} + \text{Reaeration}$$
$$= \text{Oxygen Utilized} + \text{Outflow Oxygen}. \tag{2.8}$$

These values can all be described mathematically; thus the equation becomes:

$$Q_i(RQS) + \Delta Q(\text{Any DO}) + K_2 D_{RQS} V = K_1 L_{Seg} + Q_o(RQS) \tag{2.9}$$

where Q_i = inflow (million cubic feet per day)

Q_o = outflow (million cubic feet per day)

ΔQ = flow change in segment (million cubic feet per day)

V = volume of the segment (million cubic feet per day)

K_2 = reoxygenation coefficient (day^{-1})

K_1 = decomposition reaction coefficient (dimensionless)

RQS = River Quality Standard (pounds per million cubic feet or (P.P.M.)(62.4))

Any DO = dissolved oxygen of entering flow (pounds per million cubic feet or (P.P.M.)(62.4))

D_{RQS} = deficit to RQS (pounds per million cubic feet or (P.P.M.)(62.4))

and L_{Seg} = BOD allowed in segment (pounds per day).

The second term in equation 2.9 can be revised as follows:

$$\Delta Q(\text{Any DO}) = \Delta Q(RQS) + \Delta Q(\text{Any DO} - RQS). \tag{2.10}$$

If the right hand side of this equation is substituted into equation 2.9 and $Q_o(RQS)$ subtracted from both sides, the equation may be reduced to the following form:

$$\Delta Q(\text{Any DO} - RQS) + K_2 D_{RQS} V + K_1 L_{Seg}. \qquad (2.11)$$

The left hand side of this equation is the total oxygen available for BOD reduction in pounds per day, and henceforth, it shall be referred to as AO. Thus by definition:

$$AO = \Delta Q(\text{Any DO} - RQS) + K_2 D_{RQS} V. \qquad (2.12)$$

From equations 2.11 and 2.12 the total BOD load allowed in the segment may be evaluated as:

$$L_{Seg} = \frac{AO}{K_1}. \qquad (2.13)$$

This total BOD allowed in the segment can either flow into the segment from upstream or it can be added as a pollutional load within the segment. Thus,

$$L_{Seg} = L_{residual} + L_{added}. \qquad (2.14)$$

Letting $L_{residual}$ = HOBOD for hold over BOD and L_{added} = ALOAD for allowable BOD load in the segment, the equation becomes:

$$L_{Seg} = HOBOD + ALOAD \qquad (2.15)$$

where all units are in pounds per day in the segment. Substituting for L_{Seg} in equation 2.13 and solving for ALOAD the following is obtained:

$$\text{ALOAD} = \frac{\text{AO}}{K_1} - \text{HOBOD} \tag{2.16}$$

The general formula for the allowable BOD in a segment is developed as follows. Beginning at the start of the river the allowable BOD loadings may be equated as below. The K_1 values are replaced by $\underline{K}$ to prevent double subscripting.

$$\text{Segment } 1 = \text{ALOAD}_1 = \frac{\text{AO}_1}{\underline{K}_1}$$

$$\text{Segment } 2 = \text{ALOAD}_2 = \frac{\text{AO}_2}{\underline{K}_2} - \frac{\text{AO}_1}{\underline{K}_1}(1-\underline{K}_1)$$

$$\text{Segment } 3 = \text{ALOAD}_3 = \frac{\text{AO}_3}{\underline{K}_3} - (1-\underline{K}_2)\left[\frac{\text{AO}_2}{\underline{K}_2} - \frac{\text{AO}_1}{\underline{K}_1}(1-\underline{K}_1)\right] - \frac{\text{AO}_1}{\underline{K}_1}(1-\underline{K}_1)(1-\underline{K}_2)$$

$$= \frac{\text{AO}_3}{\underline{K}_3} - (1-\underline{K}_2)\,\frac{\text{AO}_2}{\underline{K}_2}$$

The value for segment N then becomes:

$$\text{Segment } N = \text{ALOAD}_N = \frac{\text{AO}_N}{\underline{K}_N} - (1-\underline{K}_{N-1})\,\frac{\text{AO}_{N-1}}{\underline{K}_{N-1}} \tag{2.17}$$

where AO_N = the total available oxygen in segment N

$\underline{K}_N$ = the K_1 value for a particular segment as f (time)

and ALOAD_N = allowable added BOD per segment.

The total waste assimilation capacity for the river, SLOAD in terms of pounds of BOD per day, may be obtained by summing the individual segment loads for the entire river Thus,

$$\text{SLOAD} = \sum_{i=1}^{N} \text{ALOAD}_i = \sum_{i=1}^{N}\left[\frac{\text{AO}_i}{\underline{K}_i} - (1-\underline{K}_{i-1})\frac{\text{AO}_{i-1}}{\underline{K}_{i-1}}\right]. \tag{2.18}$$

The purpose of this dissertation is the evaluation of this summation to the maximum degree possible with available data. It will be shown in Chapter IV that this evaluation can be made in terms of the river flow, flow velocity, cross sectional area, mean depth, temperature, saturation dissolved oxygen, RQS, and channel slope, all of which are usually obtainable from existing records.

CHAPTER III

THEORY - TIDAL ZONE

A river which flows into a saline body of water exhibiting a tidal action is itself affected by both the tidal action and the salinity. The rise and fall of the tides affects both the flow velocity and the depth of the river, and this is important because as will be shown in Chapter IV the reaeration coefficient K_2 is often a function of one or both of these parameters. The salinity is important because density currents are often created between the saline water and the fresh water and because of the effect of salinity on K_1, K_2 and the saturation dissolved oxygen content. In order to provide a background for the tidal zone study the next two sections will present discussions of the tidal phenomena and salinity characteristics respectively, and they will then be followed by a discussion of their effect on waste assimilation capacity.

The Tidal Phenomena

Tides are a phenomena experienced by the ocean waters and, to a lesser extent, by inland bodies of water. Tides, which may be described as the periodical vertical movements of bodies of water, and the associated tidal currents, or horizontal movements, are caused by the gravitational attraction between the waters of the earth and the sun, moon, and planets. The tidal forces produced by these bodies is directly proportional to the mass of the body and inversely proportional to the cube of the distance.(3, 4) By computing the relative importance of these bodies it is found that the moon is 2.25 times more effective than the sun in producing tides and greater than 10,000 times more effective than any of the planets.(3) For all practical purposes, only the tidal forces produced by the moon and sun need be considered.

If the relative motions of the earth, moon, and sun were such that the moon and sun appeared only on the celestial equator and if these bodies remained the same distance from the earth, the tidal forces from each could be resolved into two semidiurnal components. Semidiurnal means twice daily and diurnal once daily where daily refers to the solar or lunar day respectively. The lunar period would be one of 12 hours and 25 minutes, and the solar period would be 12 hours and zero minutes.

These forces are resolved into semidiurnal forces, rather than diurnal forces, because the tide producing force is such that the same force is produced both when the sun or moon is on the meridian and when they are 180° removed from it. The tide producing forces can be assumed as being produced by a real moon, an anti moon, a sun, and an anti sun.

If these bodies remained on the celestial equator and remained equi-distant from the earth, the magnitude of the semidiurnal forces would vary with the latitude; but they would be the only tide producing forces.

Unfortunately for tidal predictions these bodies do not follow the idealized orbits mentioned above. The various variables make necessary the evaluation of a number of tidal components, often as many as 20 or 30, of which the five most important are: (1) semidiurnal lunar component, (2) semidiurnal solar component, (3) component for variation of the moon from the earth, and (4) and (5) components for a change in the moon's declination.(3) A general tidal cycle has been found to repeat every 19 years.

The United States Coast and Geodetic Survey in cooperation with similar agencies in other countries is responsible for the collection of tidal data and for the prediction of tide heights. Their estimates of tidal heights are based on predictions made by the use of tidal theory and extensive past data for points in question. The U.S.C.&G.S. presently issues tide forecasts for the entire world in four volumes. These volumes are: Europe and West Coast of Africa (including the Mediterranean Sea), East Coast, North and South America (including Greenland); West Coast, North and South America; Central and Western Pacific Ocean and Indian Ocean. Together they contain daily predictions for 195 reference ports and for over 6000 other ports which are referenced by differences to the "reference ports."(5, 6)

Examples of the variation in the tide from day to day are shown in Illustration 2 for representative parts along the Atlantic and Gulf coasts of the United States. It will be noted that the range of tide for stations along the Atlantic coast varies from place to place but that the type is uniformly semidiurnal with the principal variations following the changes in the moon's distance and phase. In the Gulf of Mexico, however, the range of tide is uniformly small but the type of tide differs considerably. At certain ports such as Pensacola there is

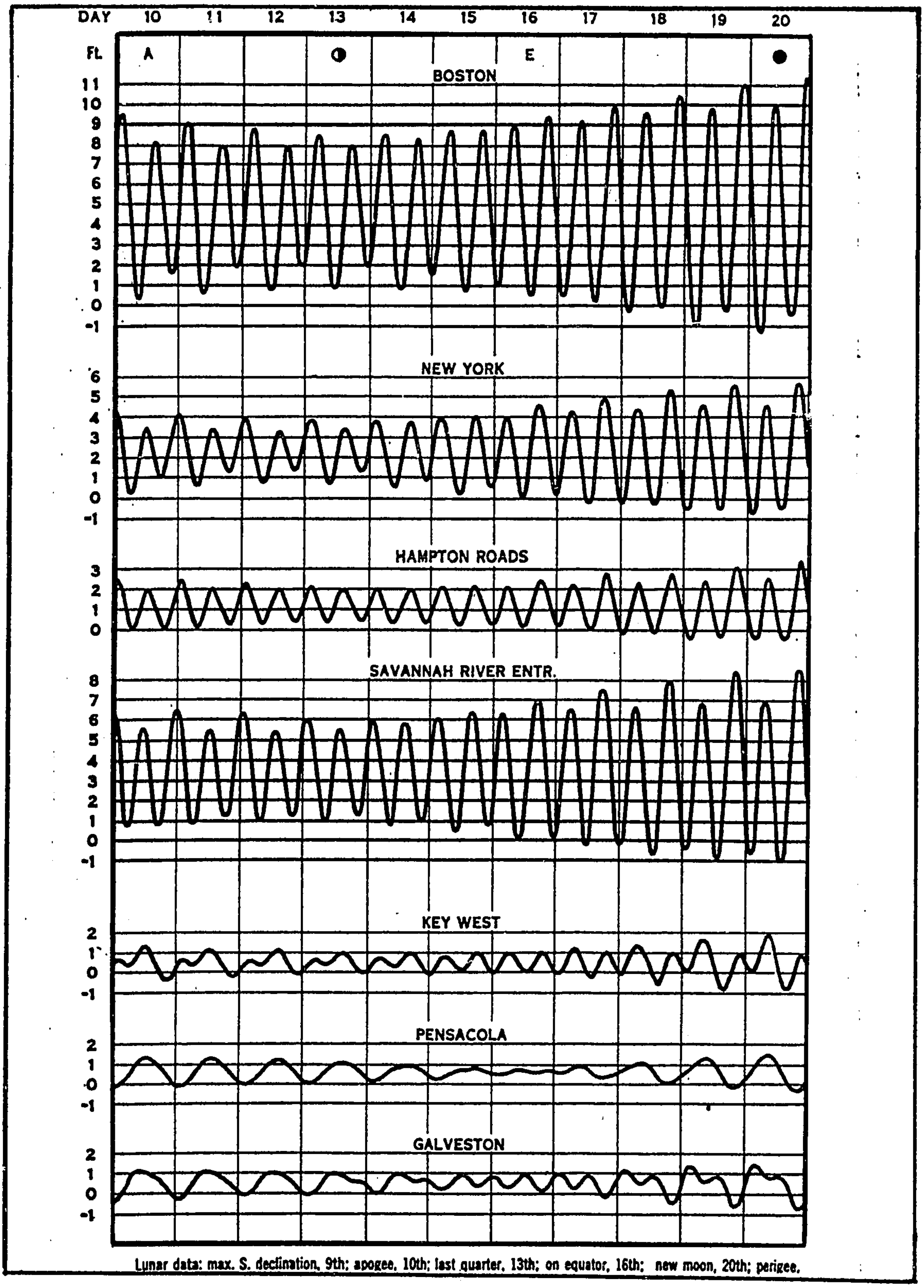


ILLUSTRATION 2

Typical Tide Curves For United States Ports usually but one high and one low water a day, while at other ports such as Galveston the inequality is such that the tide is semidiurnal around the times the moon is on the equator but becomes diurnal around the times of maximum north or south declination of the moon. In the Gulf of Mexico, consequently, the principal variations in the tide are due to the changing declination of the moon. Key West, at the entrance to the Gulf of Mexico, has a type of tide which is a mixture of semi-daily and daily types. Here the tide is semidiurnal but there is considerable inequality in the heights of high and low waters. By reference to the curves it will be seen that where the inequality is large, there are times when there is but a few tenths of a foot difference between high water and low water.(5)

Tidal Effect On Rivers

Where a river runs into the sea its characteristics for a considerable distance upstream are influenced by the rising and falling of the tide at its mouth. The point where the river surface no longer rises and falls in the tidal cycle is called the head of tide or the tidewater point. This point is usually determined by the physical characteristics of the river and river basin or by man made dams or other obstructions. For example the tidewater point of the Delaware River is at a series of rapids at Trenton, New Jersey; and the tidewater point of the Hudson River is at the base of a dam across the river at Troy, New York. For some rivers this point changes as the flow in the river changes.

The tidal height, meaning the difference between mean high and mean low tide, for points upstream from the mouth of a river is dependent on the channel configuration. Often the tidal height upstream from the mouth is greater than at the mouth. For example, the mean tide at the Battery in New York Harbor is 4.4 feet while at Troy, New York, roughly 150 miles upstream on the Hudson River, the mean tide is 4.7 feet. The time of high tide is not the same for upstream locations as it is at the mouth of the river. It has often been found that the high tide progresses upstream at the rate of $V = (gH)^{\frac{1}{2}}$ where g equals acceleration of gravity and H the depth of the water.(3)

Illustration 3 shows the range of tide or mean tidal height, the mean tidal elevation, and the time of the high tide at various points for the Delaware River. The dotted lines show actual values for the river and the solid lines are values obtained by the United States Army Corps of Engineers in their Delaware River Model.(7, 8) This again shows higher mean tides at upstream points than those found at the mouth of the river.

Generally speaking, the volume of a river between high and low tides constitutes a volume which is filled and emptied each tidal cycle. A more exacting statement would be that this volume which is filled and emptied above a particular cross-section is the volume between the water elevations existing upstream from the section in question when it is at low tide and that existing when it is at high tide. It is possible that channel configurations would require even further modification of the above statement to provide the value for the maximum volume which is filled and emptied.

Nevertheless, the water to fill this volume comes into the segment from either upstream or downstream and leaves through the downstream

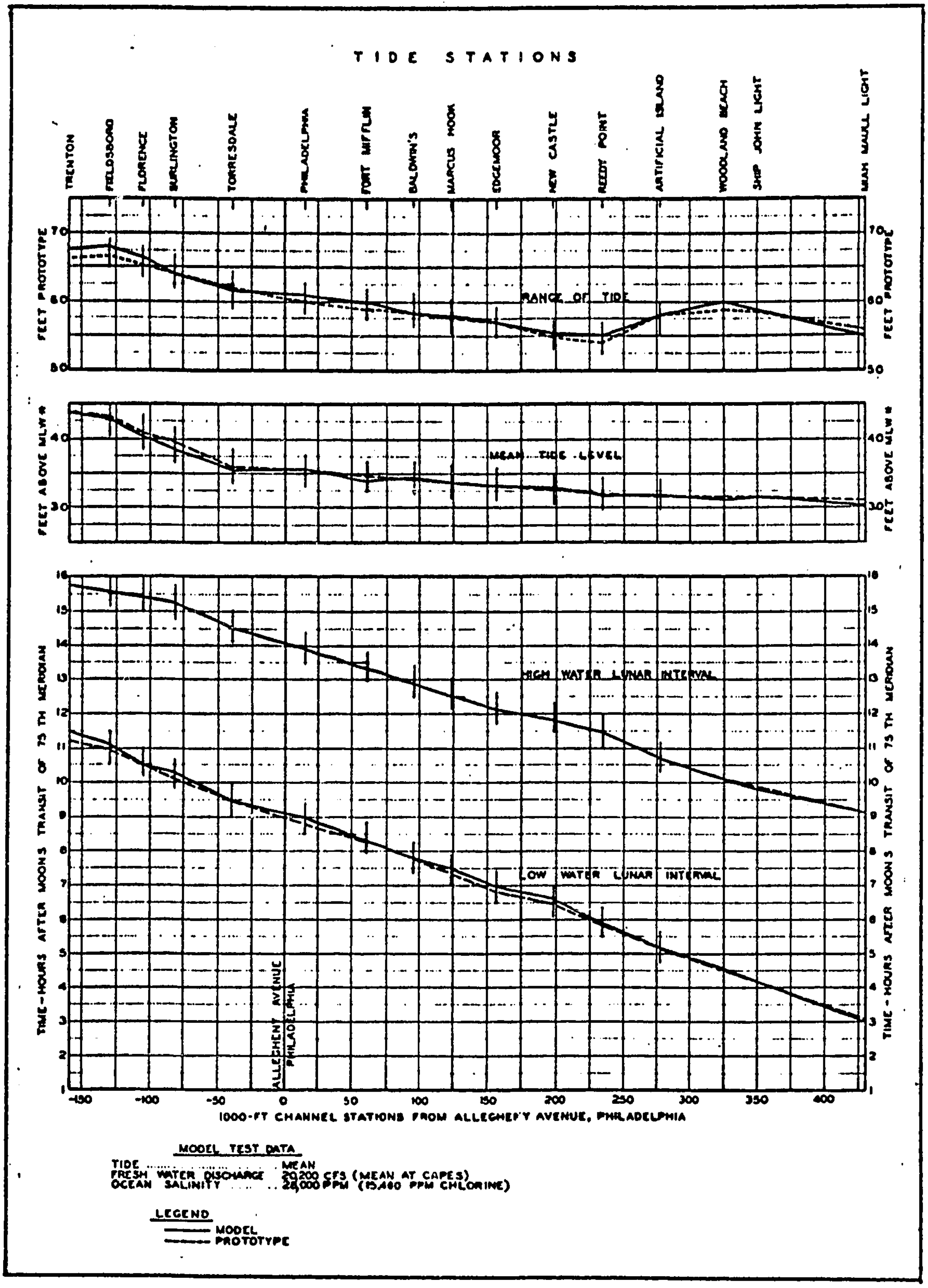


ILLUSTRATION 3

Tidal Profile Of The Delaware River

end. This transfer of water causes tidal currents.

The magnitude of the tidal currents may be obtained in two ways: measurement and calculation. The calculation of the tidal current as a function of time is a lengthy process requiring much stream data and tide records. Beginning at the tidewater point a segment downstream is selected. For a given period of time the change in volume is computed as a function of the tidal rise in the time period and the surface areas before and after the rise. The stream flow is algebraically subtracted from the change and this gives the volume change across the lower boundry. The volume change divided by the mean cross sectional area gives the average current for the time interval. The process then continues downstream by segments, where for these segments the total volume change equals the change in the segment plus the summation of the changes of the upstream segment. This procedure is known as the calculation of cubature.

Sample calculations for two segments of the Delaware River as prepared by Pillsbury are shown in Illustration 4.(9)

The second method of determining tidal currents is by actual current measurement. The United States Coast and Geodetic Survey also collects and predicts tidal currents as it likewise makes tide predictions. These are published yearly as tidal current tables in two volumes, one for the Atlantic coast of North America and one for the Pacific coast of North America.(10) In addition to these they also publish tidal current charts for various bays and rivers.(11, 12) These charts and tables give values of the currents as knots per hour for many stations both for the coast and for tidal rivers in the United

ABBREVIATED FORM OF COMPUTATIONS
MEAN CUBATURE OF DELAWARE RIVER

Based on tide records from Sept 1936 to Feb 1937

	Tides					At Lower Station				
Time	Upper Station	Lower Station	Mean	y_1-y_2	$1/2U/\Delta t$	ΔQ	$\Sigma\Delta Q$	Q	X	V
hr	ft.	ft.	ft.	ft.	sq. ft./s.	c.f.s.	c.f.s.	c.f.s.	sq. ft.	ft. per. sec.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
First Reach — Head of Tide to Trenton Municipal Pier, Length — 0.45 mile. Fresh Water Flow — 12,000 c.f.s.										
5:00		2.78								
5:30		2.43		−0.69	725	−500	−500	−12,500	5,620	−2.22
6:00		2.09		−.64		−460	−460	−12,460	5,370	−2.32
6:30		1.79		−.58		−420	−420	−12,420	5,140	−2.42
7:00		1.51		−.27		−200	−200	−12,200	4,950	−2.46
7:30		1.52		+1.97		1,430	1,430	−10,570	4,950	−2.14
8:00		3.48		+2.89		2,100	2,100	−9,900	6,380	−1.55
8:30		4.41		1.68		1,220	1,220	−10,780	7,100	−1.52
9:00		5.16		1.38		1,000	1,000	−11,000	7,650	−1.44
9:30		5.79		1.16		840	840	−11,160	8,150	−1.37
10:00		6.32		1.01		730	730	−11,270	8,590	−1.31
10:30		6.80								
Second Reach — Trenton Municipal Pier to Fieldsboro — 5.8 miles. Fresh Water Flow at Fieldsboro — 12,200 c.f.s.										
5:00	2.78	2.52	2.65							
5:30	2.43	2.16	2.30	−.70	10,900	−7,630	−8,130	−20,330	20,010	−1.02
6:00	2.09	1.81	1.95	−.66	10,700	−7,060	−7,520	−19,720	19,650	−1.00
6:30	1.79	1.50	1.64	−.56	10,530	−5,900	−6,320	−18,520	19,300	−.95
7:00	1.51	1.27	1.39	.15	10,390	1,560	1,360	−10,840	19,070	−.57
7:30	1.52	2.06	1.79	1.87	10,610	19,840	21,270	9,070	19,900	.46
8:00	3.48	3.04	3.26	2.49	11,450	28,510	30,610	18,410	20,960	.88
8:30	4.41	4.16	4.28	1.78	12,020	21,400	22,620	10,420	22,150	.47
9:00	5.16	4.93	5.04	1.40	12,450	17,430	18,430	6,230	23,000	.27
9:30	5.79	5.58	5.68	1.19	12,820	15,260	16,100	3,900	23,690	.16
10:00	6.32	6.14	6.23	1.04	13,130	13,660	14,390	2,190	24,200	.09
10:30	6.80	6.64	6.72							

ILLUSTRATION 4

Example Cubature Calculations

States. These values are usually measured in the navigation channels of the rivers and must be modified to estimate the mean velocity in the cross section.

At the mouth of the river the currents correspond closely with the tides. In other words the maximum flood (inflow) current occurs at about the same time as high tide and the maximum ebb (outflow) at the time of low tide. Further upstream, however, this is not so. Channel configuration and water slope change this relationship and the maximum current occurs before high tide. For example, on the Hudson River the maximum flood current occurs one hour and thirty minutes before high tide at Poughkeepsie, 75 miles above the mouth, and three hours before high tide at Albany, 145 miles above the mouth.(3)

Both the tidal change and current change have been found to approximate the simple cosine curve. This curve is used by the United States Coast and Geodetic Survey to predict interim values.(6, 10)

Illustration 5 shows the variation in tidal currents from day to day for several representative ports along the Atlantic and Gulf coasts of the United States. Flood current is represented by the solid line curve above the zero velocity (slack water) line and ebb currents by the broken line curve below slack water. The curves show clearly that the current cycles along the Atlantic coast are semidiurnal while those on the Gulf Coast are primarily diurnal in nature.

Illustration 6 is a portion of a tidal current chart for the Delaware Bay.(12) This particular chart is for maximum flood tide.

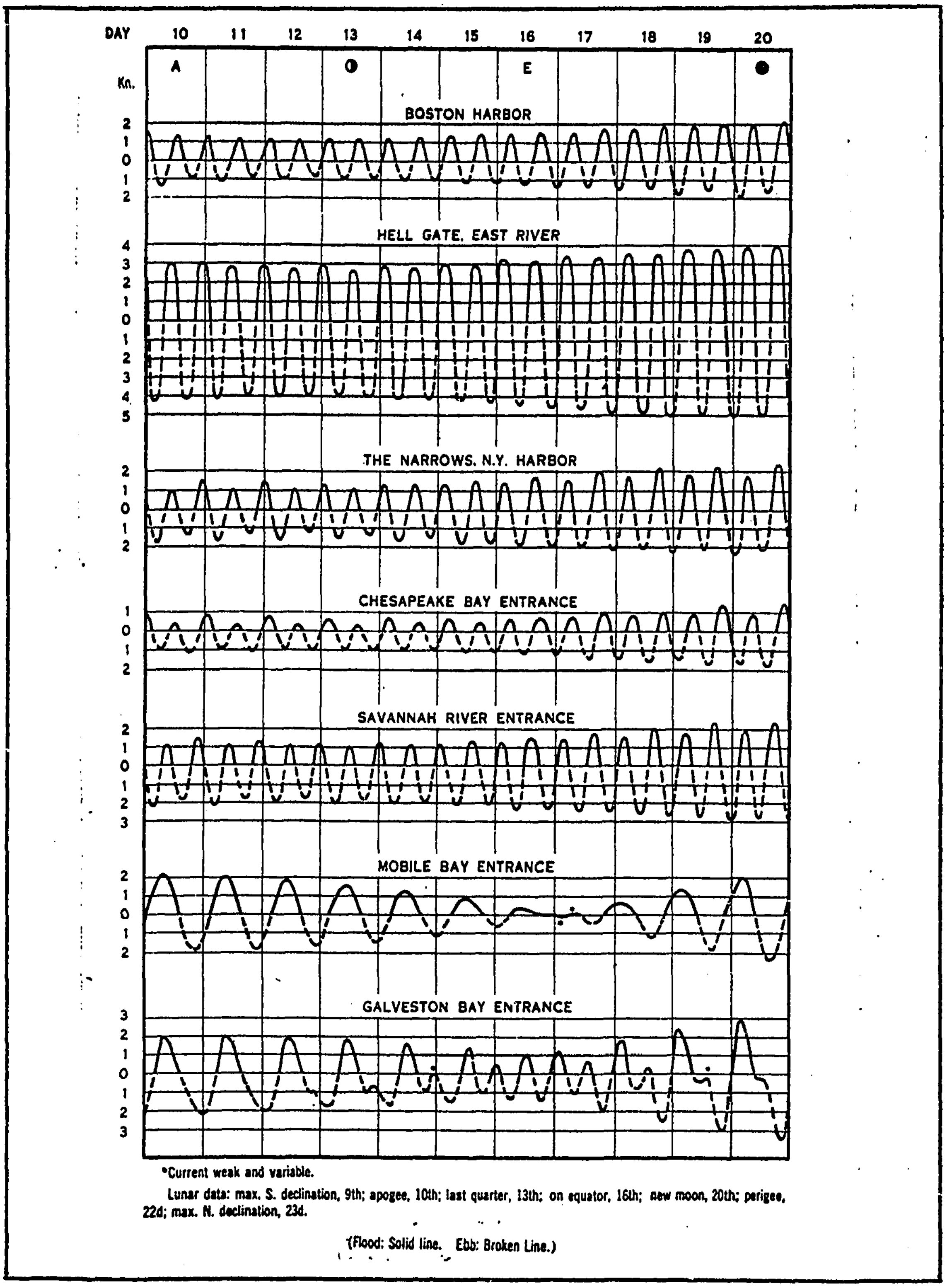


ILLUSTRATION 5

Typical Current Curves

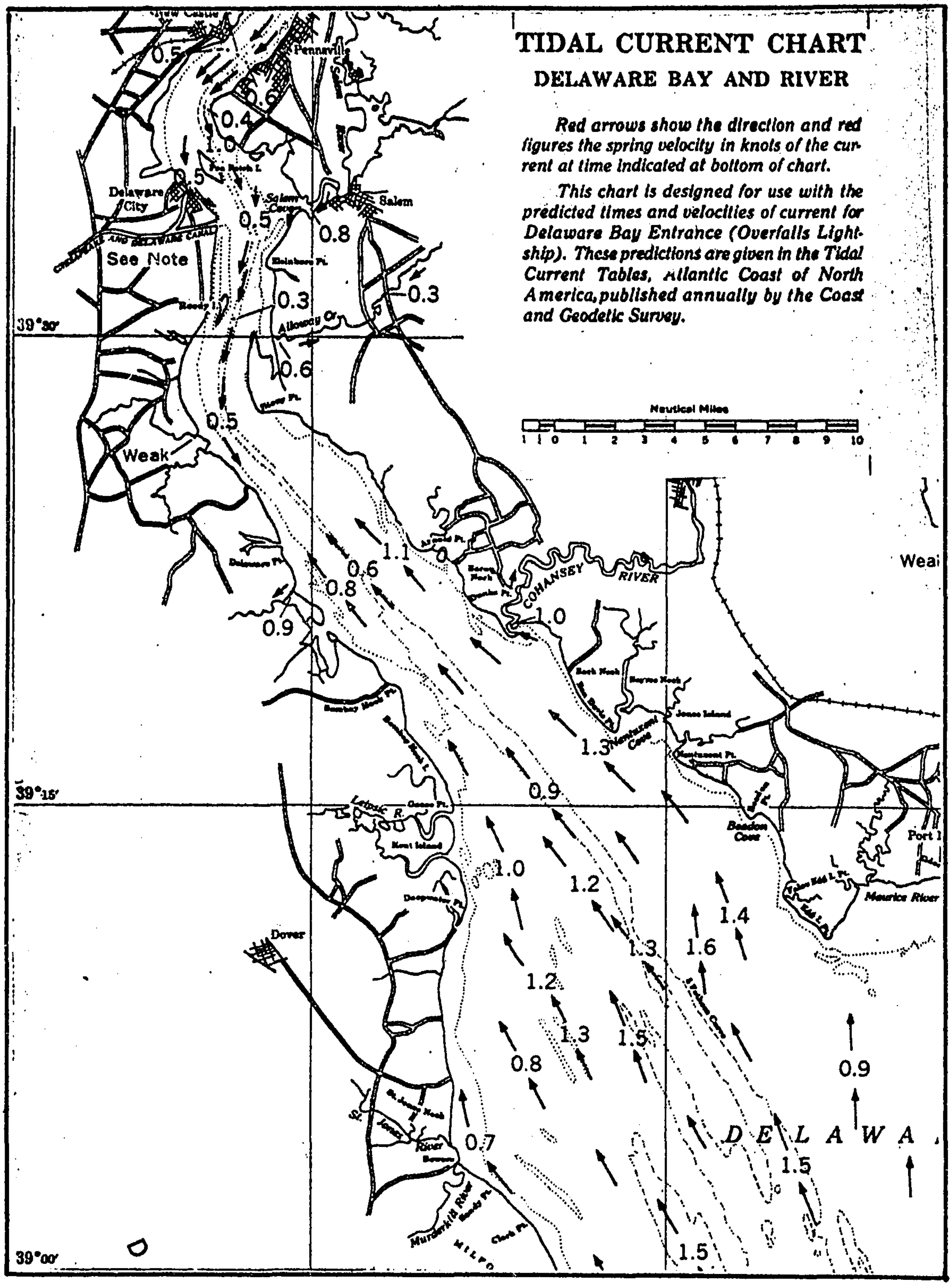


ILLUSTRATION 6

Portion Of A Tidal Current Chart

Salinity Characteristics

Other than tides another major factor must be considered in evaluating the properties of a river which flows into the ocean. That factor is the differences between the relatively pure river water and the more dense saline sea water. Generally the river water contains only a small proportion of pollutants and weighs about 62.4 pounds per cubic foot. Sea water on the other hand, contains about 30,000 parts per million of chemical salts which raise the density to about 64.2 pounds per cubic foot. This density difference coupled with other factors causes estuaries to have various salinity structures and flow patterns.

The estuary is here defined as a semi-enclosed coastal body having a free connection with the open sea and within which sea water is measurably diluted with fresh water runoff.

Within most estuaries the fresh water flow exhibits a considerable seasonal variation which reflects itself in changes in the distribution of salinity in the estuary. These changes in salt content over a yearly period may be quite large in absolute magnitude, but in considering times of lower flows or just a single tidal cycle other factors are large in proportion to the net time change.

Pritchard states that the local time rate of change of salt content is equal to the sum of the effects of the advective and nonadvective processes.(13) The advective processes are defined as those processes in which both salt and water transfer from the sea water to the fresh water, and the nonadvective processes are those where only salt is transferred.

Assuming that observations have been made of the salt content and current velocities within a segment of an estuary over a period of one or more tidal cycles, the salt balance at a given point within this segment may be given as

$$\frac{dS}{dt} = -V_1\frac{\delta S}{\delta X_1} - V_2\frac{\delta S}{\delta X_2} - V_3\frac{\delta S}{\delta X_3} + \frac{\delta}{X_1}\left[K_1\frac{\delta S}{\delta X_1}\right] + \frac{\delta}{\delta X_2}\left[K_2\frac{\delta S}{\delta X_2}\right] + \frac{\delta}{\delta X_3}\left[K_3\frac{\delta S}{\delta X_3}\right] \qquad (3.1)$$

where X_1, X_2, and X_3 are the longitudinal, verticle, and lateral coordinates respectively. S is the mean salt content averaged over one or more tidal cycles, and V_1, V_2, and V_3 are the three components of the mean velocity. The terms K_1, K_2, and K_3 are the eddy diffusivities in the three coordinant directions.

It has been stated by Pritchard that in any particular estuary two or more of the terms on the right side of the equation dominate the salt balance.(13) There exists an approximate dynamic equilibrium between several opposing processes, the result of which is a relatively slow net time change in salt content.

This does not mean that estuaries are always in a steady state, however, in most cases, two or more of the terms on the right side of the equation are each an order of magnitude larger than the net rate of change of salt concentration.

The estuaries found in the United States were almost all formed by the drowning of the lower reaches of river valleys. However, these estuaries show a wide variation in the character of the circulation pattern which is the factor which controls the salt balance structure.

These types range from the highly stratified salt wedge estuary to the well mixed vertically homogenous estuary. These different types and the characteristics found in each will be discussed in turn.

It is assumed that the estuary to be considered is an "ideal" coastal plain estuary. Thus it is an elongated indenture in a coastline with a single river source of fresh water at its upper end and a free connection with the sea at its lower end. It is then assumed that there is in this ideal estuary an absence of tidal action and other mixing forces. In such a case fresh river water would flow out over the surface of the sea water and the sea water would protrude under as a wedge under the outflowing fresh water.

If the fresh water flow were infinitely small, if there was no mixing due to wind or other external forces, and if there was no transfer of salt water across the interface, the salt water wedge would be stationary and would extend to the line where the sea level plane intersected the bottom of the river. Naturally this is a condition which is not found in nature because some mixing forces always exist.

As the flow in this idealized system becomes greater interfacial waves will form at the interface between fresh and salt water. At some critical relative speed these waves will become unstable and break, leading to an entrapment of some salt water from the salt water wedge into the upper fresh water layer. It has been found that in this situation there is little or no admixture of fresh water into the salt water wedge, but only the upward movement of salt water into the fresh water layer. The upper layer now increases its salt content as it moves seaward. Since the net salt content at the section approximates a steady state within the tidal cycle, there must be an upstream movement of saline water.

The hydraulic head of the fresh water and the increased frictional drag that exists between the two layers restricts the upstream protrusion so it no longer extends as far upstream as the intersection of the sea level and the river bottom. The greater the river flow the further downstream the upper edge of this wedge is found. Within the wedge increased upstream flow occurs to compensate for the increased loss of salt to the upper layer.

It has been shown that the breaking of the waves at the intersurface face results in the entrapment of both salt and water mass in the upper layer but not the converse which would be the entrapment of fresh water in the salt water. Therefore, the phenomena at the interface is an advective one in contrast to the nonadvective one which would have been the case if no net mass had transferred across the interface.

Once the salt water is in the upper layer, it is spread throughout that layer by turbulent mixing which is a nonadvective force. In such an estuarine system the horizontal and vertical advections are the two dominant terms in the salt balance equation. Therefore, assuming an instantaneous steady state, the equation would reduce to

$$0 = -V_1\frac{\delta S}{\delta X_1} - V_2\frac{\delta S}{\delta X_1} \quad . \tag{3.2}$$

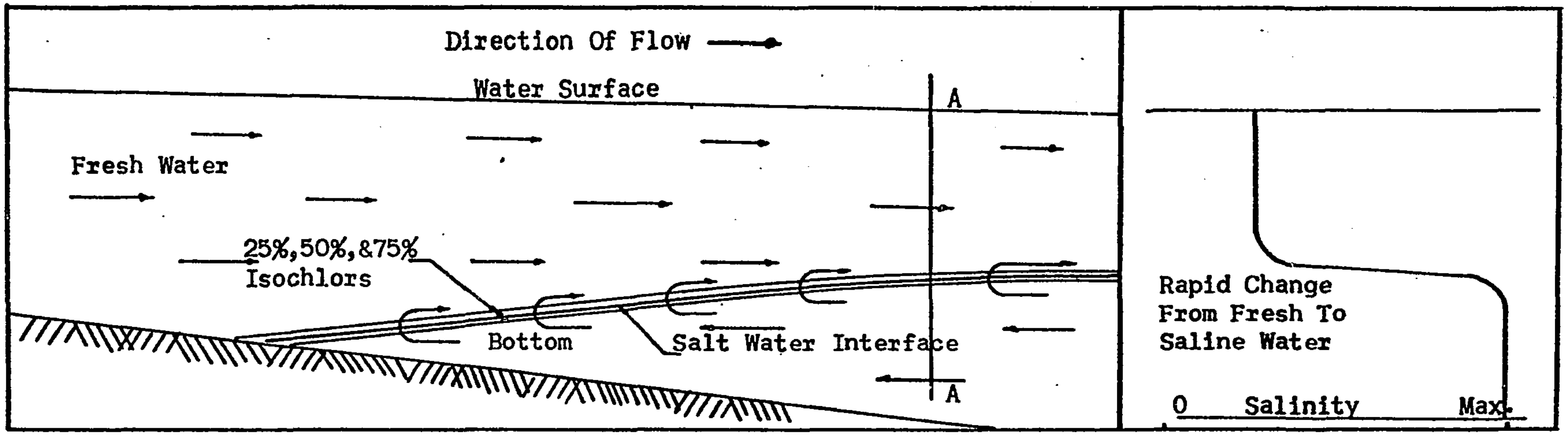
In other words, if considering the gross salt flux, the vertical advection (or flow) must be balanced by the longitudinal advection. Within the top layer the vertical mixing term $\frac{\delta}{\delta X} K_2\frac{\delta S}{\delta X_2}$ would also be significant.

The mouth of the Mississippi River and the Rhone River Estuary are typical of this highly stratified type of estuary.(13, 14) Such estuaries occur when either the ratio of river flow to tidal flow is relatively large or when the ratio of width to depth is relatively small.

Illustration 7 shows a typical longitudinal profile of a highly stratified estuary and a plot of the salinity against depth.(15, 16, 17, 18) Pritchard calls this highly stratified estuary a type A estuary.(13)

Illustration 8 is a plot of the flow velocity against depth for the condition discussed above in order to show the effect of density currents. If there were no density difference between the sea water and the river water, the flow would be distributed more or less equally from top to bottom as indicated by line a. If we consider that there is a density difference but that no advection is allowed across the fresh water-salt water interface, the flow would be confined to the upper layer as indicated by line b. If advection is allowed at the interface, a velocity pattern similar to the one shown by line c develops whereby there is a downstream flow in the upper layer and upstream flow in the bottom layer.

Several changes take place when the estuary is exposed to tidal action. The tidal velocities, which must be nearly invariable with depth, will result in eddy motions as a consequence of both internal turbulance and boundary induced turbulance. Mixing now occurs between the upper fresh water layer and the lower saline layer. In this case,



Longitudinal Salinity Profile

Salinity At Section A-A

ILLUSTRATION 7

Highly Stratified Estuary

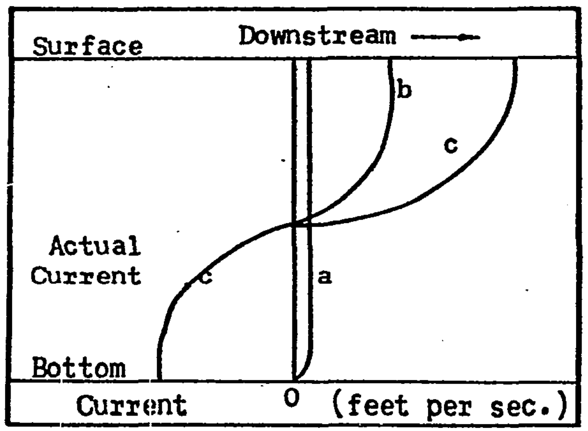


ILLUSTRATION 8

Current Profile Of A Highly Stratified Estuary not only does the salinity of the upper layer increase in the seaward direction, but also the salinity of the lower layer decreases in an upstream direction. In addition to this vertical mixing salt water is still advected to the upper layer as in the case of the stratified estuary.

In this type of estuary there would be three major terms in the salinity balance equation under steady state conditions. The equation becomes

$$0 = -V_1\frac{\delta S}{\delta X_1} - V_2\frac{\delta S}{\delta X_2} + \frac{\delta}{\delta X_2}\left[K_2\frac{\delta S}{\delta X_2}\right]. \tag{3.3}$$

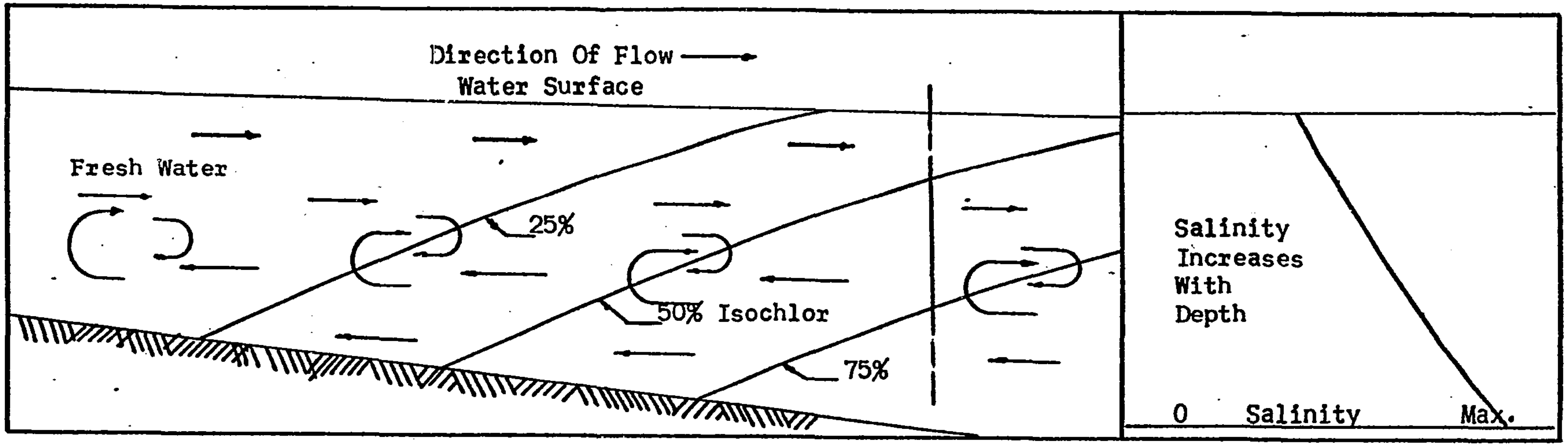
Estuaries of this type are called partially mixed or type B estuaries.(13, 15)

The net circulation pattern in this type estuary involves flow volumes many times the volume of the fresh water flow. Pritchard states that the net outflow can be as much as 40 times the river flow with a corresponding upstream flow of 39 times the river flow.(13) These flows would be in addition to those determined by cubature calculations.

Illustration 9 shows the longitudinal profile of a partially mixed estuary and a plot of the salinity for a section of the profile.

The magnitude of the currents in this type of estuary can be obtained by plotting the velocities throughout the cross section for the ebb and flood tides for a tidal cycle and then plotting their difference. Illustration 10 shows this. The Hudson, Savannah, and Charleston estuaries are typical of the partially mixed type.(19)

Although the terms are of second order magnitude in the partially mixed estuary, there is another factor which acts in this type and



Longitudinal Salinity Profile

Salinity At Section A-A

ILLUSTRATION 9

Partially Mixed Estuary

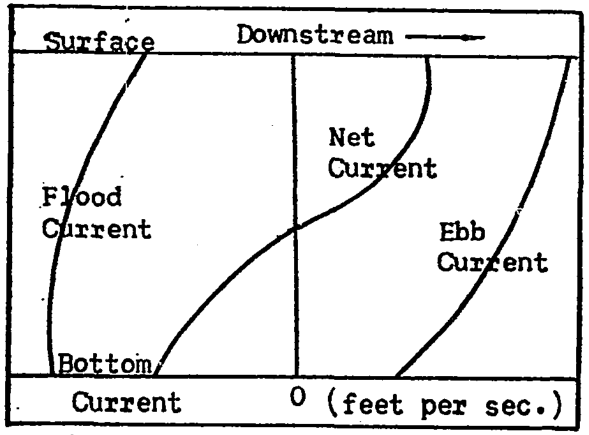


ILLUSTRATION 10

Current Profile Of A Partially Mixed Estuary which becomes increasingly important in the homogeneous estuaries which will be mentioned next. This factor is the effect of the earth's rotation. This rotation causes a slight lateral salinity gradient. In other words the fresh water-salt water interface is not parallel to the water surface in a cross section but is instead slightly tilted. The upper layer extends to greater depths and the net downstream flow is greater on the one side of the estuary than on the other looking downstream. On the United States east coast the right side or southern side would have the greater fresh water flow.(13, 14)

The third general class of estuaries is the highly mixed or homogeneous estuary. In this type tidal velocities occur which cause the salt water and fresh water to be so completely mixed that in effect a vertical homogeneity results. Since this has occurred there can be no vertical transfer of salt by either eddy diffusion or vertical advection since there is no vertical salinity gradient. There does remain, however, a longitudinal gradient with increasing salt concentration toward the sea and in some cases the coriolis or rotation effect mentioned above with a higher salt content on one side of the estuary.

If the coriolis effect is evident and assuming a steady state, the general equation takes the form

$$0 = -V_1\frac{\delta S}{\delta X_1} - V_3\frac{\delta S}{\delta X_3} + \frac{\delta}{X_3}\left[K_3\frac{\delta S}{\delta X_3}\right] \qquad (3.4)$$

where the first term is the downstream flow of salt water, the second term is the lateral advection of salt water from one side of the estuary to the other, and the third term is the eddy diffusion from one side of the estuary to the other.

Pritchard calls this a typ(? estuary and says that the wider portion of the Delaware and Raritan estuaries have these characteristics.(13)

In estuaries which are relatively narrow no upstream advection develops on one side of the estuary. If such an estuary is sufficiently mixed to be homogeneous, it is called a type D estuary by Pritchard.(13) In this homogeneous estuary there is only the longitudinal nonadvective flux or upstream eddy diffusion to balance the downstream salt flow. Thus the general equation reduces to the form:

$$0 = -V_1\frac{\delta S}{\delta X_1} + \frac{\delta}{\delta X_1}\left[K_1\frac{\delta S}{\delta X_1}\right]. \qquad (3.5)$$

In such an estuary there is no additional flow contributed by the longitudinal density difference.

Illustration 11 shows a longitudinal profile of a homogeneous estuary without rotation effect.

Illustration 12 shows the velocity characteristics of a vertical section for an ebb and flow tide and their mean value.

The Delaware, Thames, and Gironde estuaries are typical type D homogeneous estuaries.(14, 20)

The distribution of the longitudinal salinity has been studied by Aron and Stommel.(21) The equation they developed is for an estuary with a homogeneous cross section perpendicular to the longitudinal axis and a uniform width and cross section. The equation is

$$s = Se^{F(1-L/X)} \qquad (3.6)$$

or in log form

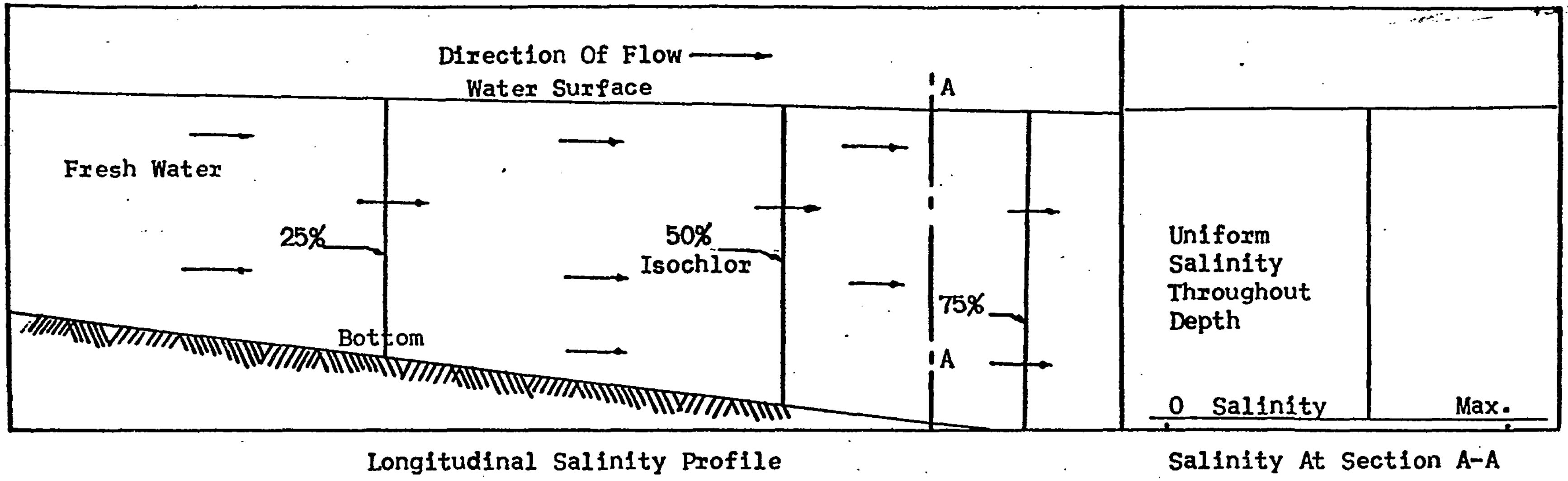


ILLUSTRATION 11

Homogeneous Estuary

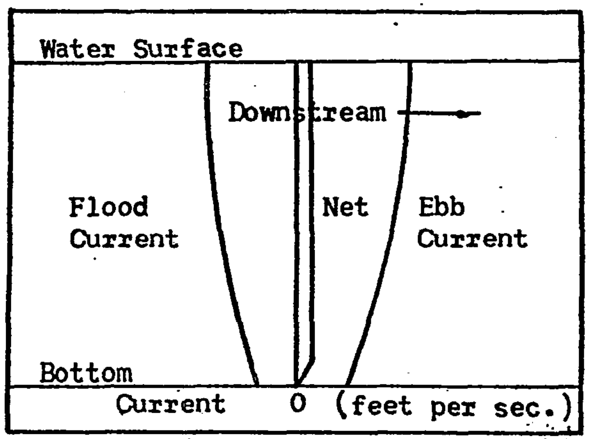


ILLUSTRATION 12

Current Profile Of A Homogeneous Estuary

$$\text{Ln}\ \frac{S}{s} = F\left(\frac{L}{X} - 1\right) \tag{3.7}$$

where S = ocean salinity (any unit)

s = salinity at any point at distance X (same units as S)

L = length of estuary (any unit)

X = distance from upstream end (same units as L)

and F = a dimensionless coefficient dependent on numerous variables.

It should be noted that equation 3.7 is linear in F.

Illustration 13 shows the plot of Ln S/s vs (L/X - 1) for the Delaware, Thames, and Gironde estuaries. Although these estuaries do not have uniform depths and widths, it can be seen from the graphs that for the middle reaches the plots do tend to be straight lines. A very straight line was obtained from the Thames River data. The data used and the values calculated for use in this graph are listed in Table 1. (7, 14, 20)

In Table 2 the salinities are calculated by means of the Aron-Stommel equation for an estuary with F value of one and a 30,000 P.P.M. sea water salt content.

Illustration 14 shows a plot of the salinity versus X/L for the Gironde and Thames estuaries and the plot of salinity versus X/L for the estuary with 30,000 P.P.M. sea water salt content and an F value of unity. It can be noted that this line closely approximates the Delaware line and falls between the Thames and Gironde. This is in accordance with the F values determined from Illustration 12.

The total salt profile moves upstream and downstream with the tide, but for uniform tide and fresh water flow the mean value will

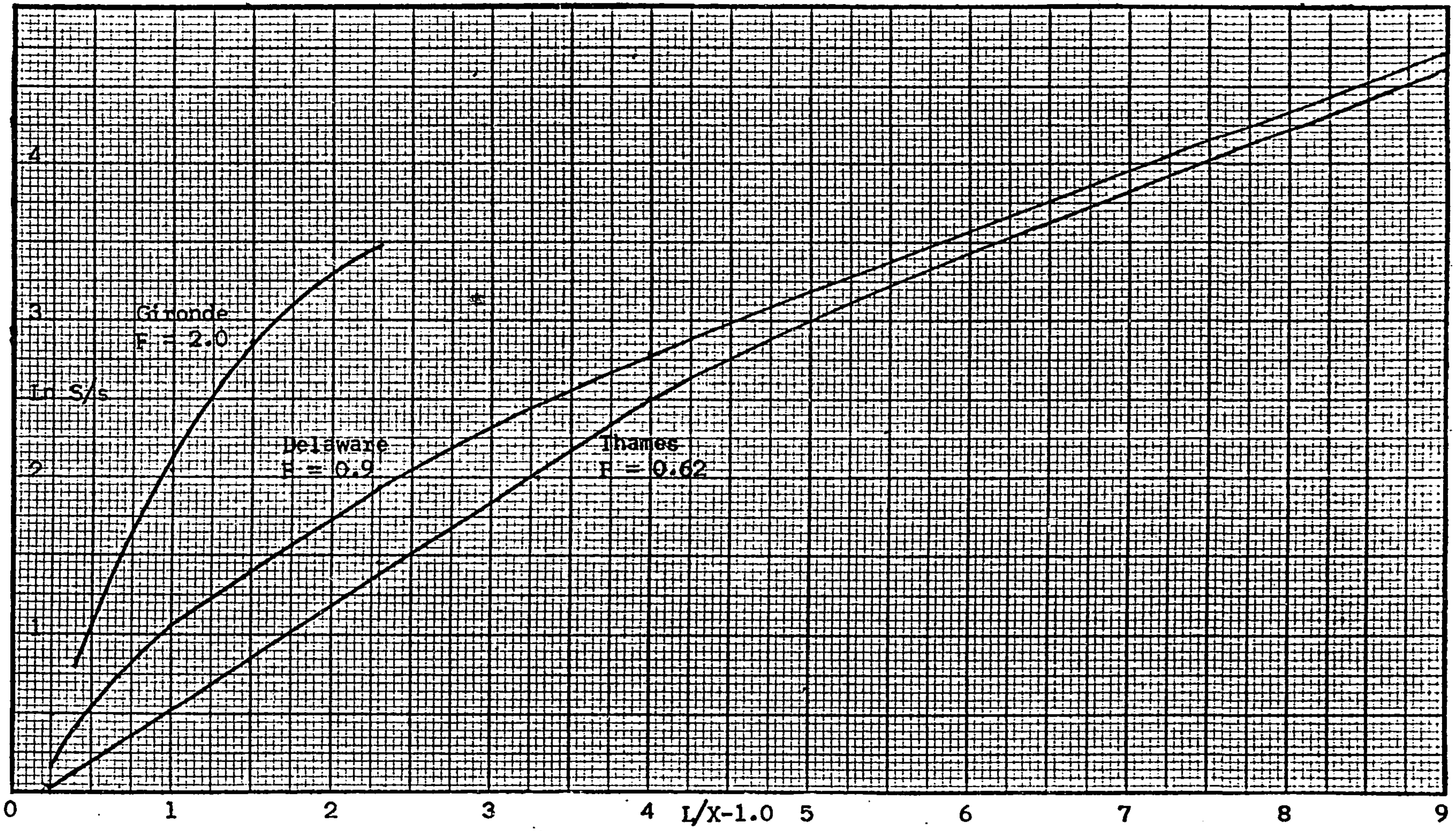


ILLUSTRATION 13

Plot To Determine Value F For Aron-Stommel Equation

TABLE 1

DATA FOR SALINITY ANALYSIS

		Estuary								
		Delaware (U.S.) L = 350,000 ft.			Gironde (France) L = 105 KM			Thames (England)		
X/L	L/X-1	s	S	Ln S/s	s	S	Ln S/s	s	S	Ln S/s
0	. .	0	28,000	. .	. .	28,000	. .	0	30,000	. .
.1	9.00	270	28,000	4.63	. .	28,000	. .	300	30,000	4.6
.2	4.00	1,800	28,000	2.78	. .	28,000	. .	2,400	30,000	2.5
.3	2.33	4,000	28,000	1.94	900	28,000	3.46	7,000	30,000	1.45
.4	1.5	6,900	28,000	1.40	1,600	28,000	2.86	12,800	30,000	.84
.5	1.0	9,600	28,000	1.07	3,600	28,000	2.05	18,000	30,000	.51
.6	.67	13,400	28,000	.74	6,200	28,000	1.50	22,000	30,000	.31
.7	.43	18,100	28,000	.44	12,000	28,000	.84	26,500	30,000	.12
.8	.25	23,500	28,000	.17	. .	28,000	. .	29,000	30,000	.03
.9	.11	. .	28,000	. .	. .	28,000	. .	29,500	30,000	. .
1.0	0	28,000	28,000	0	28,000	28,000	0	30,000	30,000	0

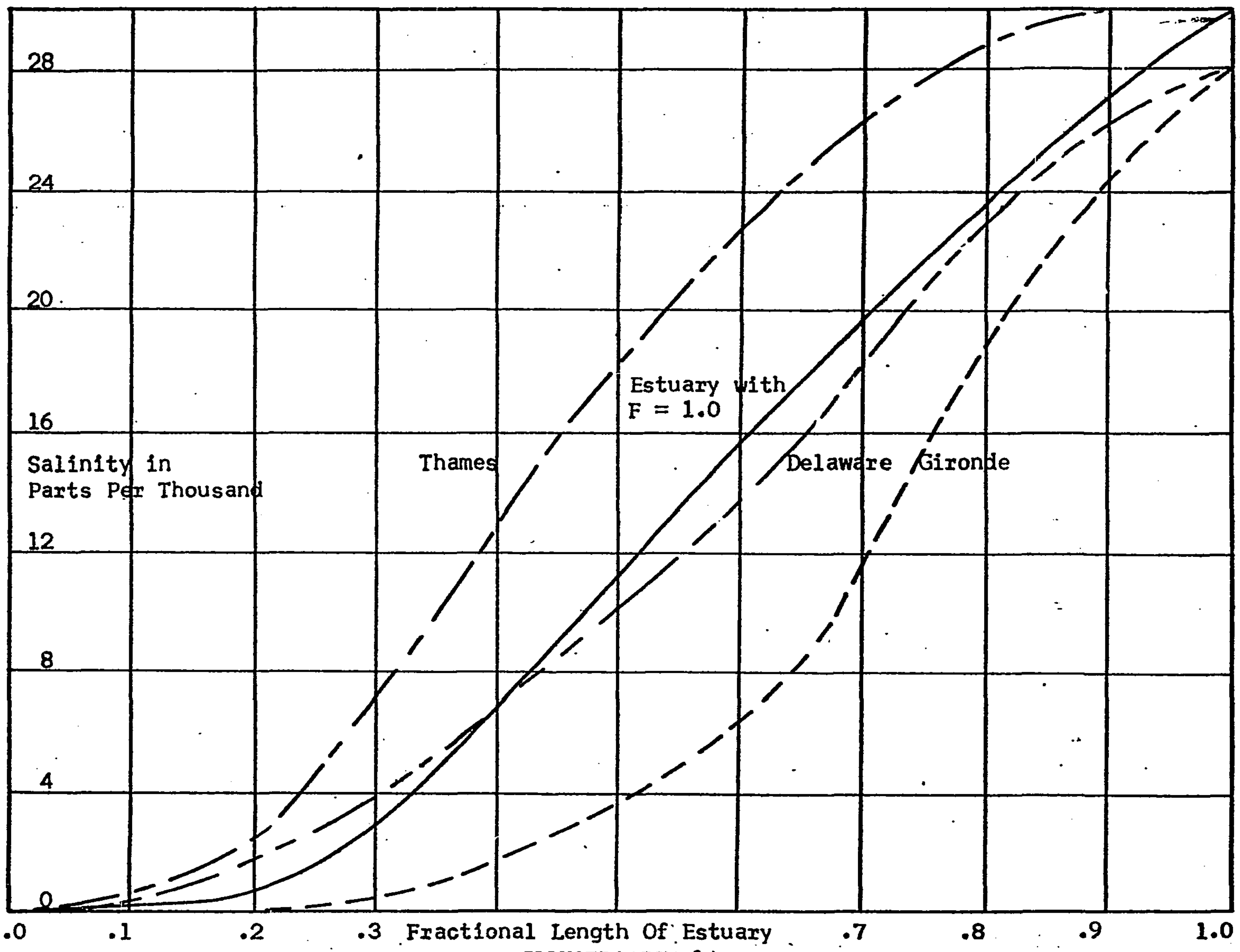


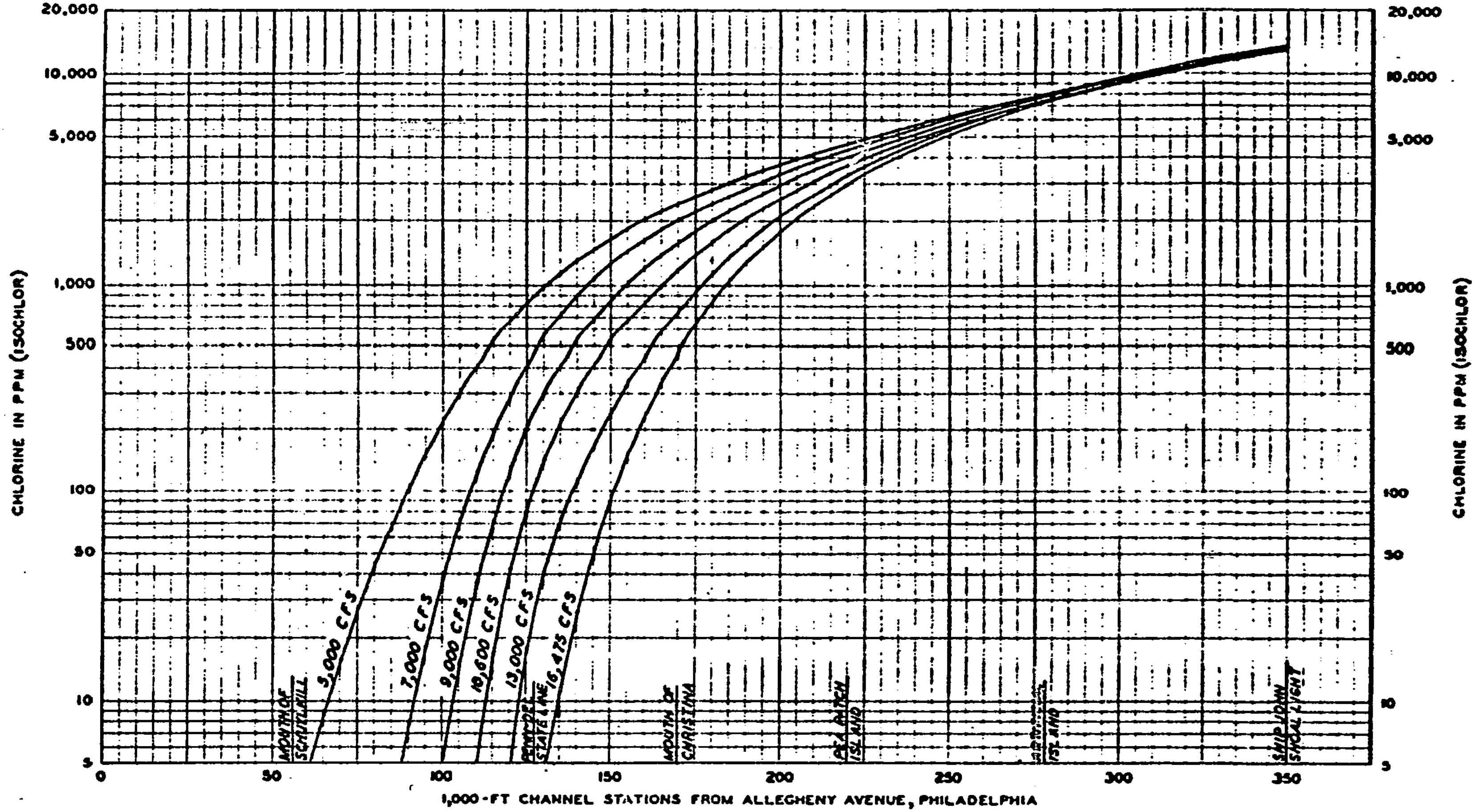
ILLUSTRATION 14

Salinity Profiles Of Several Estuaries

TABLE 2

SALINITY VALUE FOR ESTUARY WITH F = 1 AND S = 30,000

X/L	L/X-1	Ln S/s	S/s	s
.0				0
.1	9.0	9.0	8000.0	30
.2	4.0	4.0	54.5	550
.3	2.33	2.33	10.25	2,930
.4	1.5	1.5	4.48	6,700
.5	1.0	1.0	2.718	11,000
.6	.67	.67	1.95	15,400
.7	.43	.43	1.53	19,600
.8	.25	.25	1.28	23,400
.9	.11	.11	1.11	27,000
1.0	0	0	0	30,000



NOTE: FRESH-WATER DISCHARGE REFERRED TO IS THE TOTAL FRESH-WATER DISCHARGE AT AND INCLUDING THE SCHUYLKILL RIVER. SUMP SALINITY = 15,460 PPM CHLORINE. TO CONVERT PPM CHLORINE TO PPM TOTAL SALT MULTIPLY BY 1.81.

ILLUSTRATION 15

EFFECTS OF FRESH-WATER DISCHARGE ON BOTTOM SALINITY AT HIGH-WATER SLACK

MEAN TIDE

remain in the same place.

The effect of changing the fresh water inflow is shown in Illustration 15 which shows the results of a United States Army Corps of Engineers study.(7)

The Effect Of Tides And Salinity On Waste Assimilation Capacity

Fundamentally the tidal zone analysis presents only three major differences from the upstream river when considering the assimilation capacity for oxygen utilizing wastes. These are: (1) the effect of tides on flow velocities, (2) the presence and effect of salinity and salinity density currents, and (3) the flow of water in and out of the mouth of the river with the ebb and flood of the tide. In considering other wastes, especially those wastes which are not reduced in the river, another factor called the flushing time becomes important.

In considering the first difference, the effect of tides on flow velocities, the tidal zone can be considered divided into two different zones. These zones are shown in Illustration 16 for a typical estuary. The first and most upstream section, which will be called Zone 1, is the zone which is affected by tides but where the flow in the river is sufficiently high to fill the entire volume of the prism between high tide and low tide. The tidal effect in this zone is the quickening and slowing of the river flow velocity during the tidal cycle. This is illustrated in the first segment of the example cubature calculations for the Delaware River, Illustration 4. It can be noted in the example that at the municipal pier in Trenton, New Jersey, the downstream flow velocity varies from 2.46 to 1.31 feet per second with a mean value for

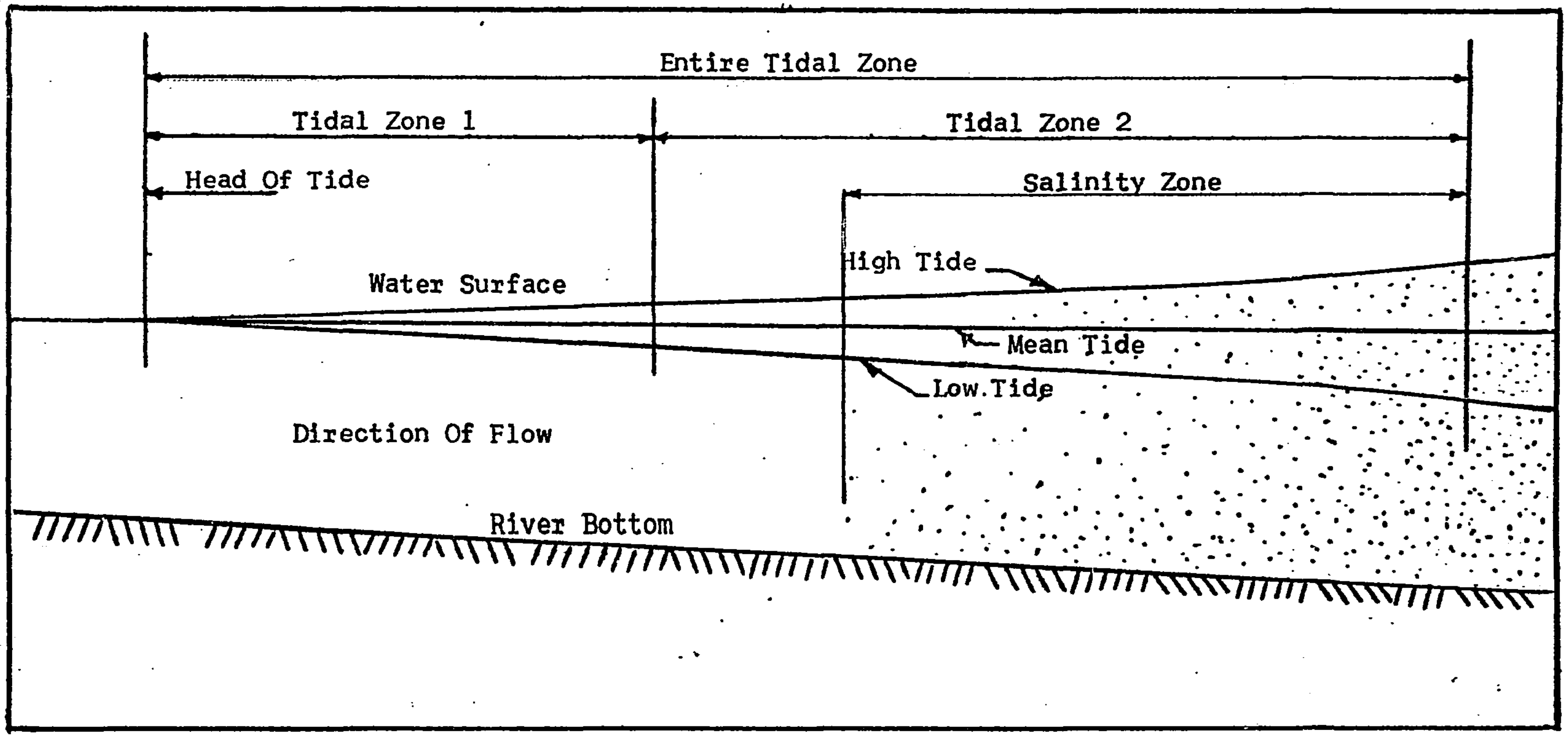


ILLUSTRATION 16

Longitudinal Profile Of Estuary Showing Different Tidal Zones the six hour period of 1.885 feet per second.

The end or downstream limit of this Zone 1 is the point at which a zero downstream flow occurs at some time in the tidal cycle or in other words, it is the point at which there is an infinitely small upstream flow at some time during the cycle. At this point the maximum stream flow will be twice the mean flow, and the flow in the twelve hour and 25 minute tidal cycle varies as a cosine function as shown in Illustration 17.

This variation in velocity has one major affect on waste assimilation capacity. It is that the variation of velocity from the mean can affect the reoxygenation coefficient, K_2.

If the reaeration coefficient K_2 is a function of the river flow velocity raised to a power of less than one, then the coefficient would be smaller for this velocity distribution than for a uniform flow condition. It will be shown in the next chapter that this is indeed the case, because the preferred Dobbins-O'Conner formula is a function of the velocity raised to the $\frac{1}{2}$ power. The difference in the reaeration coefficient for the most downstream point is evaluated in Table 3.

From the analysis in Table 3 it may be seen that if the mean velocity were used at the downstream end of Zone 1, there would be an over estimation in the reaeration coefficient of 1.0/.90 or 11.1%. A velocity of $(.90)^2$ times the mean velocity would give an effective velocity which would yield the proper reaeration coefficient.

The above development is a special case of the general formula for this zone which is developed below from Illustration 18.

It is assumed that the values for the maximum velocity (V_1) and

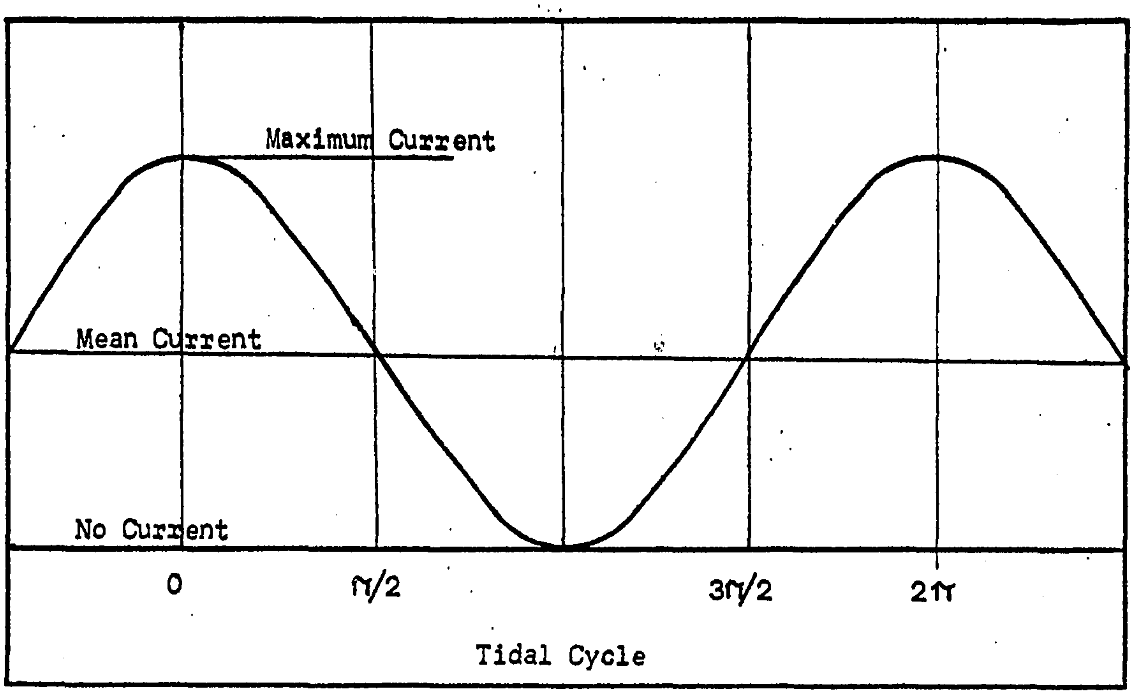


ILLUSTRATION 17

Currents At The End Of Tidal Zone 1

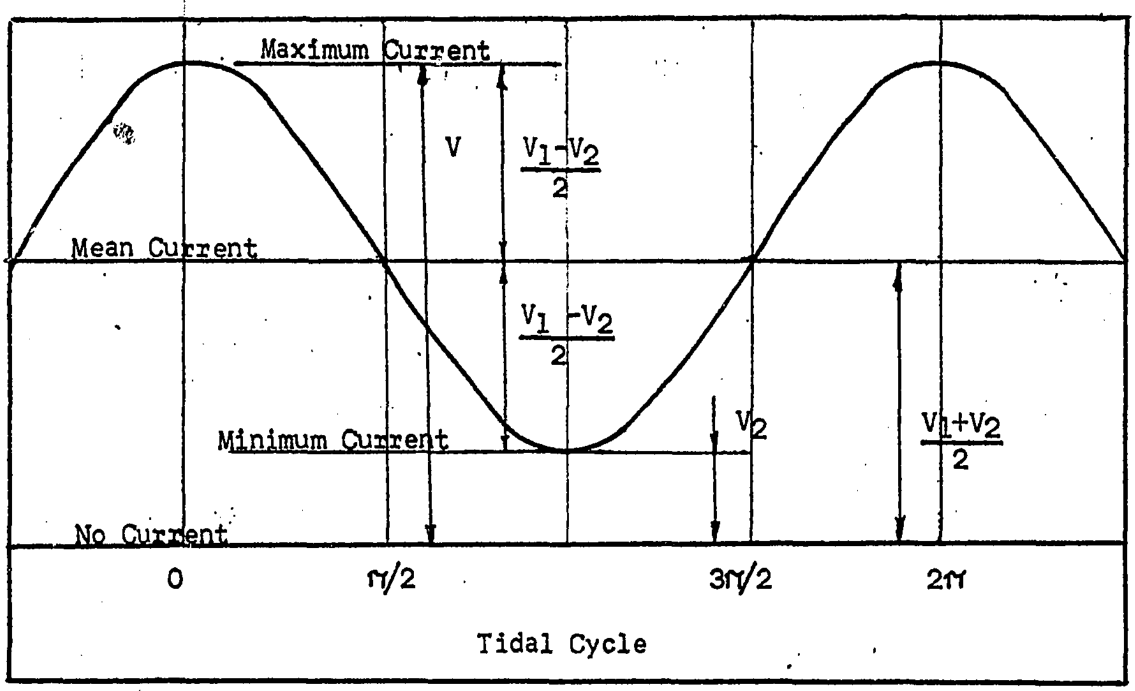


ILLUSTRATION 18

Currents In Tidal Zone 1

TABLE 3

EFFECT OF VELOCITY VARIANCE ON REAERATION COEFFICIENT AT END OF TIDAL ZONE 1

End of Segment in Degrees	Mean of Segment X	ΔX	Cos X	1+Cos X	$(1+\cos X)^{\frac{1}{2}}\Delta X/(\pi/18)$
0					
10	5	$\pi/18$	+.997	1.997	1.414
20	15	$\pi/18$	+.965	1.966	1.403
30	25	$\pi/18$	+.906	1.906	1.380
40	35	$\pi/18$	+.819	1.819	1.348
50	45	$\pi/18$	+.707	1.707	1.306
60	55	$\pi/18$	+.574	1.574	1.255
70	65	$\pi/18$	+.423	1.423	1.194
80	75	$\pi/18$	+.259	1.259	1.122
90	85	$\pi/18$	+.087	1.087	1.043
100	95	$\pi/18$	-.087	.913	.955
110	105	$\pi/18$	-.259	.741	.861
120	115	$\pi/18$	-.423	.577	.759
130	125	$\pi/18$	-.574	.426	.652
140	135	$\pi/18$	-.707	.293	.541
150	145	$\pi/18$	-.819	.181	.425
160	155	$\pi/18$	-.906	.094	.306
170	165	$\pi/18$	-.965	.035	.187
180	175	$\pi/18$	-.003	.055	.055
Total		π			16.206

K_2 Velocity Term = Mean of $(1+\text{Cos } X)^{\frac{1}{2}} V_{Mean}^{\frac{1}{2}} = 16.206(\frac{\pi}{18})\ \frac{1}{\pi}(V_M)^{\frac{1}{2}} = .90V_M^{\frac{1}{2}}$

Effective Velocity = $(K_2 \text{ Velocity})^2 = (.90V_M^{\frac{1}{2}})^2 = .81V_M$

minimum velocity (V_2) are given and that the velocity varies around the mean value as the cosine curve with a period equal to the tidal cycle.

The actual reaeration coefficient velocity term may be found by integrating the following function for a $\frac{1}{2}$ tidal period.

$$K_2(\text{Velocity term}) = \int_{X=0}^{X=\pi} \left(\frac{V_1 + V_2}{2} + \frac{V_1 - V_2}{2} \cos X\right)^{\frac{1}{2}} \qquad (3.8)$$

The equivalent velocity may then be evaluated as the square of this term. This integral can be quickly evaluated on the computer by means of the numerical integration by segments used above for the special end point case.

In times of relatively low flow Zone 1 is very short in regions with relatively high tides. This is in part due to the low flow and in part due to the fact that the head of tide point is usually caused by some physical feature such as a dam or rapids in the river, and therefore, immediately below this point there is a sizable mean tidal change with a considerable tidal prism volume in a short length of channel. For example, on the Hudson River the head of tide is at Troy, New York, where the Hudson is dammed.(6) At that point the flow is all downstream, however, the current records at Albany, New York, less than ten miles downstream show a considerable upstream flow during part of the tidal cycle. Similarly the cubature calculations for the Delaware River show a considerable upstream flow at Fieldsboro, 5.8 miles downstream from the head of tide point.

The short length of Zone 1 and the small effect it has on the tidal pollution assimilation capacity make it unrealistic to try and re-evaluate the velocities in this zone. It is considered better to either assume that the river flows at its arithmetic mean velocity to the end of this zone and to only consider tidal flow downstream from this point or to choose the initial segment from the head of tide long enough to extend over this point. The latter procedure was used in the applications made in this dissertation.

The second zone of the estuary is the portion in which the river fresh water flow is insufficient to fill the tidal prism and therefore, a flow reversal occurs whereby water from downstream flows upstream to fill the prism.

A general plot of the flow velocity across a cross section due to tides and river flow is shown in Illustration 19. The flow velocity upstream (flood flow) is shown below the zero line, and the flow downstream (ebb flow) is shown above the line. T_1 is the total time of the ebb flow, and T_2 the time of the flood flow, and $T_1 + T_2$ the time of the tidal cycle.

V_1 will equal V_2 only if there is no river flow. Therefore, the zero line is not to be considered as a mean value line. V_1 is considered to vary as the cosine within time T_1, and V_2 is considered to vary as the cosine within T_2. Thus the total curve is meerly the joining of different cosine curves and is not a complete cyclic cosine curve around the line $\frac{V_1 + V_2}{2}$.

As in Zone 1 the mean velocities must be adjusted to make provision for the effect of varying velocity on the reaeration coefficient. The velocity term of the reaeration coefficient is described for the two terms as follows:

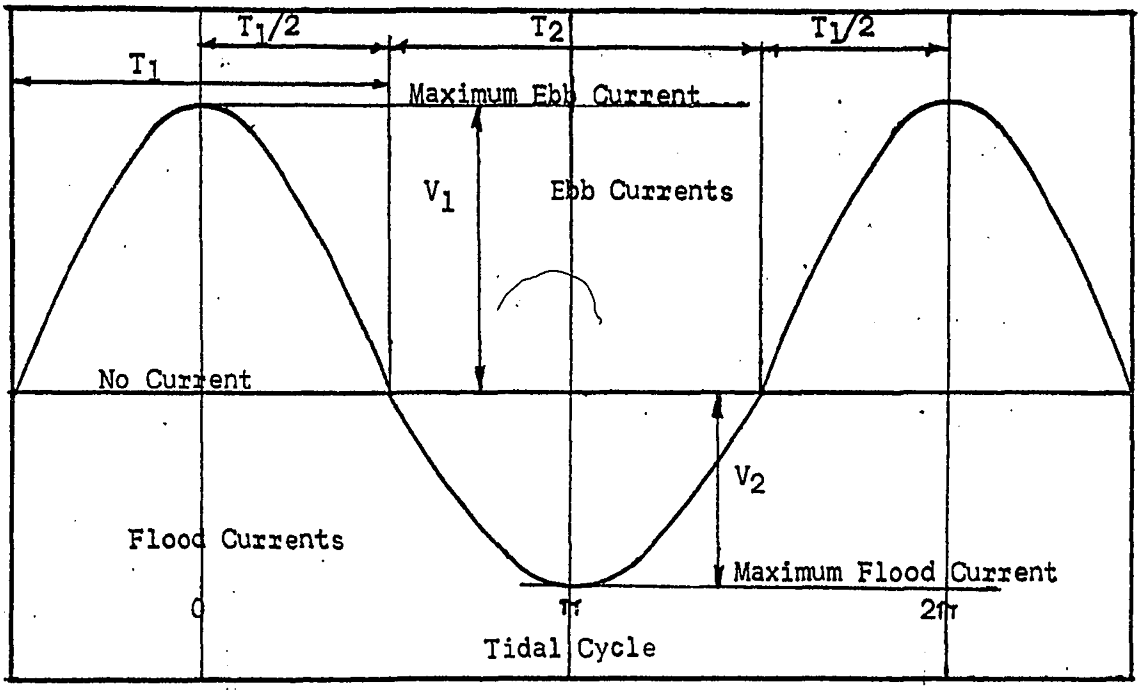


ILLUSTRATION 19

Currents In Tidal Zone 2

$$K_2(\text{Velocity Term})_{V_1} = \int_0^{\pi/2} (V_1 \text{Cos } X)^{\frac{1}{2}} = V_1^{\frac{1}{2}} \int_0^{\pi/2} (\text{Cos } X)^{\frac{1}{2}} \tag{3.9}$$

$$K_2(\text{Velocity Term})_{V_2} = \int_0^{\pi/2} (V_2 \text{Cos } X)^{\frac{1}{2}} = V_2^{\frac{1}{2}} \int_0^{\pi/2} (\text{Cos } X)^{\frac{1}{2}}. \tag{3.10}$$

The effective velocity for the ebb and flow velocities is equal to the squares of the velocity terms of the reaeration coefficient.

Thus
$$V_1 \text{ eff.} = \left[V_1^{\frac{1}{2}} \int_0^{\pi/2} (\text{Cos } X)^{\frac{1}{2}}\right]^2 \tag{3.11}$$

and
$$V_2 \text{ eff.} = \left[V_2^{\frac{1}{2}} \int_0^{\pi/2} (\text{Cos } X)^{\frac{1}{2}}\right]^2. \tag{3.12}$$

The net effective mean velocity for the entire segment is:

$$\text{VMEAN}_{\text{eff.}} = \left[\frac{T_1}{T_1 + T_2}(V_{1\text{eff.}})^{\frac{1}{2}} + \frac{T_2}{T_1 + T_2}(V_{2\text{eff.}})^{\frac{1}{2}}\right]^2. \tag{3.13}$$

The integral $\int_0^{\pi/2} (\text{Cos } X)^{\frac{1}{2}}$ may be evaluated by numerical integration by segments to be equal to 1.198. Thus the mean value over the range 0 to $\pi/2$ is then $1.198/(\pi/2)$ or .763. Thus the equations become:

$$V_1 \text{ eff.} = (.763\, V_1^{\frac{1}{2}})^2 = .583\, V_1 \tag{3.14}$$

$$V_2 \text{ eff.} = .583\, V_2 \tag{3.15}$$

and
$$\text{VMEAN}_{\text{eff.}} = \left[\frac{T_1}{T_1 + T_2}(.583\, V_1)^{\frac{1}{2}} + \frac{T_2}{T_1 + T_2}(.583\, V_2)^{\frac{1}{2}}\right]^2. \tag{3.16}$$

Therefore, if the values V_1, V_2, T_1, and T_2 are furnished, then an effective mean velocity for use in computing the reaeration coefficient may be evaluated. Good estimates of these values can be obtained, and therefore, the effect of tidal action on flow velocities can be evaluated.

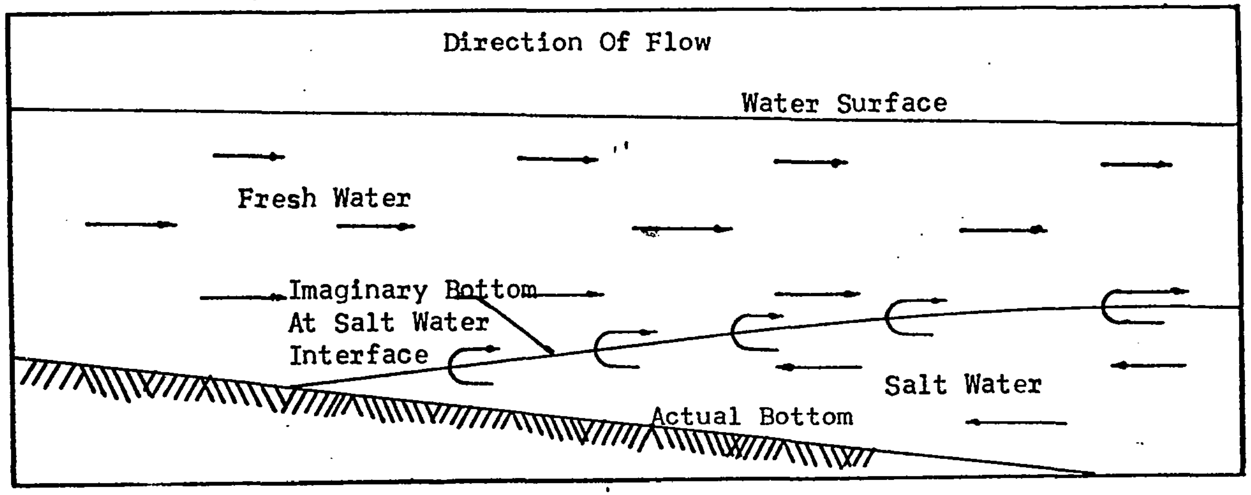
As mentioned previously, the second major problem encountered in the tidal zone is the effect of salinity and salinity density currents.

The salinity zone of the tidal portion of a river may include all or part of the entire tidal zone, although in most cases, it covers only a portion of the Zone 2 considered in the flow evaluation.

The salinity itself affects some of the factors used in determining the waste assimilation capacity of the tidal zone. These factors are the saturated dissolved oxygen content of the water and the K_1 and K_2 coefficients. These relationships will be discussed at length in Chapter VI.

The various salinity structures which range from highly stratified to homogeneous were discussed earlier in this chapter. Although the analysis procedures for the tidal zone will be made on the assumption of uniformly loading each segment to its capacity, the effect of the saline structures can best be understood by visualizing the structure and noting the effect of a single pollutional load. This will be demonstrated for the two structural extremes, the highly stratified and homogeneous estuaries.

The highly stratified estuary can be idealized as shown in Illustration 20. Since there is a point of zero longitudinal velocity between the fresh and salt water layers, the surface of these points for the river defines an imaginary river bottom. It was shown earlier that in this type estuary there is a movement of salt water mass from the bottom layer to the top but not a reciprocal movement.



Longitudinal Salinity Structure

ILLUSTRATION 20

Idealized Highly Stratified Estuary

If a single pollutional load is discharged into the top level, it will stay in the top level, and as it flows toward the sea, it will be diluted by the addition of salt water flowing up through the imaginary bottom. If the pollutional load were discharged into the bottom layer, part of the load would be carried immediately to the upper layer because its carrying water is lighter and warmer than the dense salt water. The rest, however, would remain in the lower layer and would be transported upstream with the salt water and be discharged along with the salt water through the interface into the fresh water upstream from the original point of discharge.

The advantages presented by this type of estuary in considering waste assimilation capacity depends on the type of waste considered. With the decaying wastes such as the oxygen demanding wastes considered in the methods of this dissertation, the advantages may be two-fold.

The first advantage is that the presence of the false bottom makes it plausible to use the depth to the false bottom instead of the total depth in the calculation of the reaeration coefficient. The second advantage is obtained if the salt water entering a particular segment has a better water quality than the upper layer. More specifically, an advantage is obtained if the dissolved oxygen content of the salt water is higher than that of the upper layer when considering oxygen depleting wastes.

An advantage accrues to the non-decaying pollutant because the addition of the mass to the salt water increases its flow velocity and thereby decreases its flushing time from the estuary.

The homogeneous estuary behaves quite differently. There are no zones of density flow, only a continually maintained longitudinal salinity profile. It perhaps can be visualized best by assuming that the salt ions move upstream and downstream with the tide, but that the mean location of each molecule does not change. It may be assumed that density and tidal forces maintain them thus against the downstream flow of fresh water.

Naturally there is probably some actual exchange of the salt ions with those in other places in the estuary and with the ocean, but the overall result is small compared to other forces affecting pollution assimilation capacity.

In the homogeneous estuary it makes little difference at what depth pollutional loads are applied since there is no vertical salinity gradient and since tidal action will dispense the pollutant.

Since it is assumed that there are no actual density currents in a homogeneous estuary, the analysis for waste assimilation capacity in this zone requires only the flow velocity analysis outlined for Zone 2 of the tidal zone and a correction for the salinity. Since there is no imaginary bottom in this type estuary, all calculations are made using the full mean depth.

The computer program written for use for the tidal zone of the river can be used for either the homogeneous or highly stratified estuary.

To analyze the homogeneous estuary the mean salinities, actual mean depth, and river fresh water flow should be furnished to the computer program along with the other data which will be enumerated in the discussion of the program in Chapter V.

For the highly stratified estuary the values furnished for each segment should be the salinity for the upper layer, the mean depth to the imaginary bottom, and the total river flow in the top level which would be a combination of fresh water flow and salt water flow. The salt water flow entering across the salt water interface into a particular segment can be evaluated by the following analysis.

Consider that a segment has inflow from three sources: (1) a known downstream flow F_1 at Salinity A, (2) a known fresh water inflow F_2 at Salinity B, and (3) an unknown salt water inflow at sea Salinity C. Consider also that it has an unknown outflow of flow F_4 and known Salinity D. The flows can be summed as:

$$F_1 + F_2 + F_3 = F_4 \qquad (3.17)$$

and the salt balance as:

$$AF_1 + BF_2 + CF_3 = DF_4. \qquad (3.18)$$

Since A, B, C, D, F_1, and F_2 are known, this leaves two equations with two unknowns. Solving for F_3, the salt water flow across the interface, yields:

$$F_3 = \frac{1}{(C - D)}\left[(D - A)F_1 + (D - B)F_2\right]. \qquad (3.19)$$

A literature search has not yielded any information on the dissolved oxygen content of the saline layer in the highly stratified estuary. Also data has not been found to demonstrate whether any excess oxygen remains with the water as does the salt or whether it transfers out of the lower layer across the salt water interface to equalize the dissolved oxygen content in terms of percent of saturation. To alleviate this problem it is assumed that any salinity inflow has a dissolved oxygen content already equal to, or which has been depressed to, the River Quality Standard.

In the pollutional analysis of this zone it is assumed that no wastes are retained in the lower salinity layer and that no reaeration or reduction of wastes takes place in this zone.

A partially mixed estuary may be evaluated by analyzing it as both a homogeneous and a highly stratified estuary and by computing weighted mean answers.

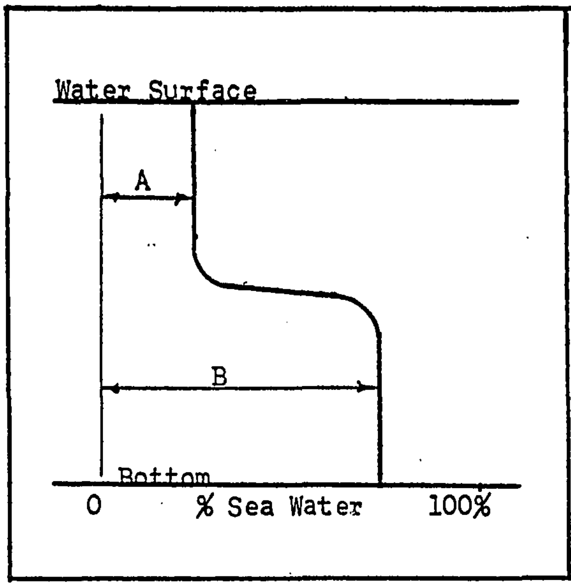
The degree of stratification (DS) is defined as 1 for the highly stratified and as 0 for the homogeneous estuary and may be evaluated as follows. Considering a partially mixed estuary as shown in Illustration 21 A, it is desired to divide it into two estuaries with profiles as shown in 21 B and 21 C. C is the ocean salinity. The degree of salinity may be evaluated as:

$$DS(C) + (1 - DS)(A) = B \,. \qquad (3.20)$$

Since A, B, and C are known,

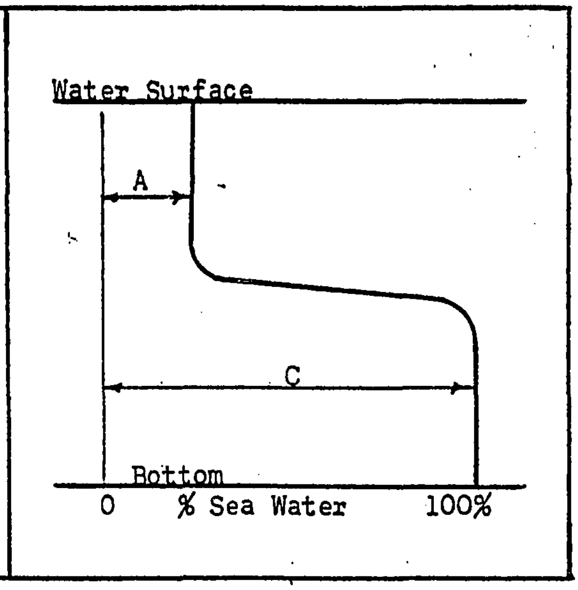
$$DS = \frac{B - A}{C - A} \,. \qquad (3.21)$$

The use of the above methods and assumptions yields a practical method for estimating the waste assimilation capacity of the tidal zone of a river.



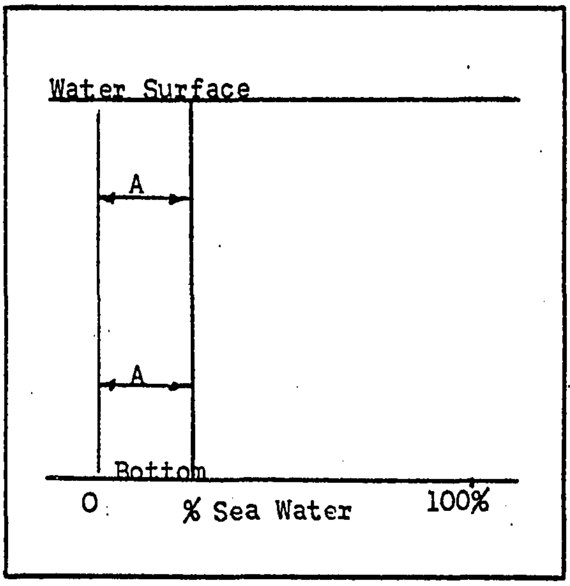
PART A

Partially Mixed Estuary



PART B

Highly Stratified Estuary



PART C

Homogeneous Estuary

ILLUSTRATION 21

Salinity Structures

CHAPTER IV

EVALUATION OF K_1, K_2, AND OXYGEN DEFICIT

As has been discussed in the previous chapters the waste assimilation capacity of a river is a function of several coefficients. These are (1) K_1 the BOD decay coefficient, (2) K_2 the reoxygenation coefficient, and (3) the oxygen deficit in the stream. The coefficients K_1 and K_2 are rather complex factors for which much time and effort has been spent trying to evaluate them. The oxygen deficit is the difference in dissolved oxygen capacity between the saturation value and an actual or allowable stream condition.

The purpose of this dissertation is not to attempt to derive new evaluations for these variables or to particularly endorse any specific theory or formulation. A detailed study has been made, however, of the various published theories and formulas in an attempt to learn and utilize current thinking and also to find those formulations which are adaptable to the methods for estimating waste assimilation which are presented in this dissertation.

Evaluation Of K_1

K_1, the coefficient of deoxygenation or BOD decay coefficient, is usually considered as a constant at a fixed temperature although it has been found to vary with the nature of the pollutant, the relative prior seeding of the stream with decay bacteria, and with the bacteria compatability of the receiving water.(22)

Values for K_1 have been noted between 16% and 70% per day with a mean value of about 40% per day for a seeded river. This reduction in BOD as a function of time is the actual reduction which has been observved in rivers and is thereby distinguished from the reduction rate experienced in bottle sample BOD analysis.

A K_1 value of 0.345 at 20° Centigrade has been used on the examples and applications presented in this dissertation primarily because this is the value used by the United States Public Health Service in their recent study of Ohio River conditions in 1980.(23) This value is entirely arbitrary, and therefore, if any data is in existance which would indicate a more accurate value for a particular river, then it should be used.

The fact that K_1 is not more precisely measured will not cause any great error because K_1 is primarily a coefficient referring to the delaying action or downstream passage of the pollution and for other than a short stretch of river, it becomes rather insignificant.

Several studies have been made to evaluate the temperature effect on the K_1 coefficient.(1, 24, 27) These studies generally agree that in the usual range of summertime natural water temperatures, 15°– 30°C., that the K_1 varies with temperature in accordance with the following formula although the third decimal is considerably in doubt. The formula is:

$$K_1(T) = K_1(20^\circ C.)\ \ 1.047^{(T-20^\circ C.)} \qquad (4.1)$$

The K_1 value has been found to vary considerably when a receiving water has a salinity content. Gotaas has studied this extensively and has determined that in concentrations up to about 25% of the salinity strength of sea water the K_1 value is higher than that for fresh water. (25) If the salt content increases beyond this point, however, then K_1 falls below the value for fresh water. From an examination of Gotaas' results, it was observed that this relationship could be approximately represented by the diagram shown in Illustration 22.

Assuming that between a chloride concentration of 0 and 6 parts per thousand the function is a parobola, it may be evaluated by the equation:

$$K_1(\text{At Salinity S}) = K_1(\text{Salinity}=0)(1-.01233(S-3.0)^2). \qquad (4.2)$$

When the salinity expressed as chloride is greater than 6 parts per thousand, the function is a straight line and may be evaluated as

$$K_1(\text{At Salinity S}) = K_1(\text{Salinity}=0)(1-.021)(S-6.0)). \qquad (4.3)$$

K_1 may be modified to apply to a period of more or less than one day by means of the following formula in which D equals the time in days.

$$(1 - K_1(D)) = (1 - K_1)^D$$

or

$$K_1(D) = 1 - (1 - K_1)^D. \qquad (4.4)$$

<u>Evaluation Of K_2</u>

The K_2 coefficient directly determines the oxygen regenerative capacity of a stream, and therefore, the accuracy of its determination reflects directly on the calculated waste assimilation capacity. The

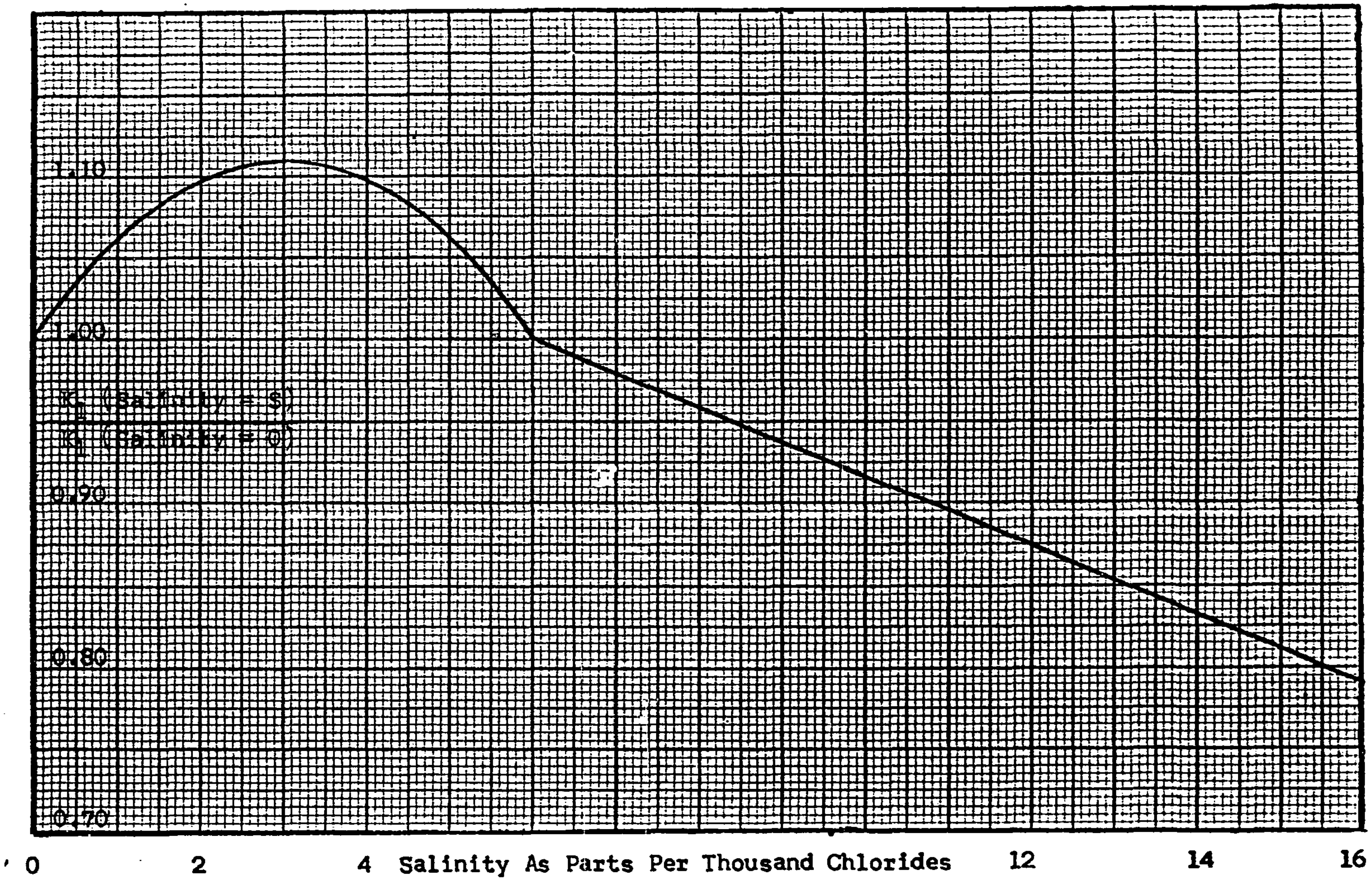


ILLUSTRATION 22

Variation Of K_1 With Salinity

K_2 coefficient, or reaeration coefficient, can be described as the recovery rate of an existing oxygen deficit expressed as fractional recovery per day. The deficit may be stated in parts per million, pounds of oxygen, or other compatable dimensions.

Some authors consider K_2 to relate to the total regenerative capacity of the stream from all sources, while others consider K_2 to apply only to the reoxygenation obtained from the atmosphere and use a K_3 value to represent other sources of oxygen. These other sources are photosynthesis, chemicals, mechanical reaeration, etc.(24, 26)

It is realized that in many instances the factors which make up K_3 can be very significant. However, methods and formulation to evaluate them except by field studies are not available, and these sources in many cases are not dependable. For instance a dark cloudy day would severely decrease photosynthesis. For these reasons and because it is conservative to do so, the K_3 components will be ignored in the considerations of this dissertation. Thus any K_3 values which can in the future be justified may be considered as local advantages over and above the waste assimilation capacities computed by the methods herein.

K_2 has been formulated by a number of authors, the first of whom were Streeter and Phelps.(7, 27) Their formulation has been well accepted, but the variables which they use are not easy to evaluate and in the end depend on the field evaluation of a constant for a particular stretch of river. Their formula is:

$$K_2 = \frac{C\ V^n}{H^2} \qquad (4.5)$$

where K_2 = reaeration coefficient (day^{-1})

V = mean velocity of flow (ft./sec.)

H = mean depth of water above extreme low water (ft.)

and n and C are coefficients which were obtained by substituting field values of V, H, and K2 from field data. It was found that n could be expressed as a function of the "mean velocity per five foot increase in river gage," thus $n = 1/\frac{1}{Y - 1.17}$ where Y equals the increase.

The large amount of field work necessary to evaluate this formula for each stretch of a river has caused several authors to define K_2 in more readily attainable factors.

Probably the first of these was Velz who extended a development made by Black and Phelps based on the laws of diffusion between air and water.(27, 28, 29) This is based on Fick's Law of Hydro-diffusion:

$$\frac{\delta S}{\delta t} = -aq\,\frac{\delta c}{\delta X} \tag{4.6}$$

which states that the rate at which a dissolved substance S is diffusing across any area q and in the direction of the X axis is proportional to q and to the rate at which the concentration is changing along the direction X. $\frac{\delta c}{\delta X}$ is called the concentration gradient, and "a" is known as the coefficient of diffusion. This is analogus to heat flow problems.

Under the water-air conditions diffusion brings about an increase in the concentration of dissolved oxygen at all points below the surface layer and mathematical manipulation has shown

$$D_t = D_a\, 0.811\, \left(e^{-K} + \frac{e^{-9K}}{9} + \frac{e^{-25K}}{25} \;.\;.\;.\right) = D_a(0.811\; S_K) \tag{4.7}$$

where D_a = initial dissolved oxygen deficit (P.P.M.)

D_t = deficit at time t (P.P.M.)

S_K = sum of the continuing series

and $K = \frac{\pi^2 a t}{4L^2}$ where L = depth and "a" the diffusion coefficient.

"a" has been evaluated at a mean value of .00153.

To account for turbulence in this concept, it is considered that there will be a period of quiescent diffusion followed by an instantaneous mixing of the entire depth. The time between these mixes is dependent on the stream turbulence and velocity.

If the decrease in oxygen deficit, or reaeration, in a body of water is denoted as R, then R is equal to the difference between the initial deficit and the deficit at time t. Thus $R = D_a - D_t$. This can be evaluated as:

$$R = D_a - D_t = D_a(1 - 0.811\ S_K). \qquad (4.8)$$

Velz states that it has been found that this closely resembles the empirical relationship $\text{Log } R_o = 1.85 + 0.5 \text{ Log } K$ where R_o equals the reaeration rate per mix at zero dissolved oxygen and where

$$R = R_o \frac{Da}{DO(\text{saturated})}. \qquad (4.9)$$

For example, if:

Depth L = 10 ft.

Temperature = 20 Centigrade

a = .00153

Saturation dissolved oxygen = 9.2

Mixings = 10 per hour

Time per mix = .1 hour

$\frac{Da}{DO(\text{saturated})} = 70\%$

then: $$K = \frac{\pi^2(.00153)(.1)}{4\ (10)^2} = 3.77\ (10^{-6})$$

and $$\text{Log } R_0 = 1.85 + 0.5 \text{ Log } 3.77\ (10^{-6})\ .$$

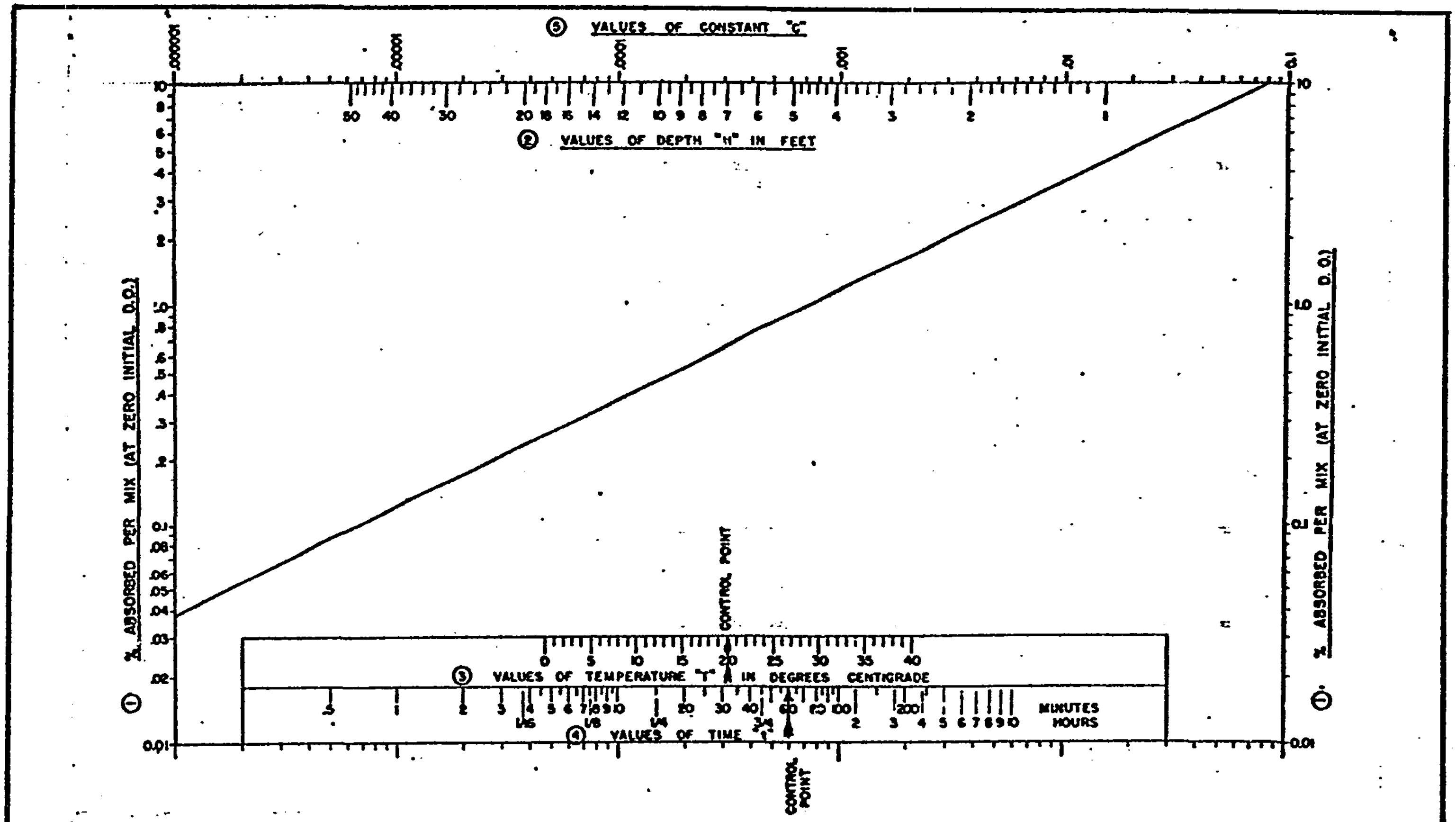
Therefore, R_0 equals .14% of saturation per hour per mix, or 1.4% per hour with ten mixes per hour. This can be converted to other units and thereby equals .51 P.P.M. dissolved oxygen per day, or .0576 pounds per 1000 cubic feet per day.

From this analysis Velz has prepared the sliding scale nomograph which is shown in Illustration 23 along with his instructions for its use.(28) Illustration 24 shows the relationship between mix interval and depth for the Kalamazoo River. Velz has expressed his belief to the United States Public Health Service that these mix interval values are applicable to all fresh water streams except very shallow and swift ones.(30)

The O'Conner-Dobbins formulas were developed by use of diffusion theory and the theory of fluid turbulence. The basic development using the diffusion theory is similar to Velz' work, but to it is added the turbulence theory and through it an evaluation of the rate of surface renewal. A somewhat abbreviated development of their formulas is presented below.(31, 32, 33)

The passage of oxygen from the atmosphere to a body of turbulent fluid may be described in the most complete and the most general manner by the Lewis and Whitman two film concept expressed in differential form similar to Fick's Law of Hydro-diffusion. This leads to the equation

$$\frac{\delta m}{\delta t} = D_G\left[\frac{\delta c}{\delta y}\right]_1 = D_L\left[\frac{\delta c}{\delta y}\right]_2 = D_e\left[\frac{\delta c}{\delta y}\right]_3 \qquad (4.10)$$



Standard Reoxygenation Curve. Example of use of chart: Given: Depth 10 ft.; Temp. 25° C.; mix interval 30 min.; initial D.O. content at 75 per cent saturation. Solution: Slide control of scale (3) to coincide with 10 ft. on scale (2); opposite 25° C. on scale (3) read on scale (5) "C" 0.00024; slide control of scale (4) to 0.00024 on scale (5); opposite 30 minutes on scale (4) read on scale (5) "C" 0.000122; drop vertically from "C" 0.000122 to the Standard Reoxygenation Curve and opposite this intersection read on scale (1) 0.41 per cent saturation; convert to per cent absorbed per day at initial D.O. content of 75 per cent, $0.41 \times (1.00 - 0.75) \times 1440/30 = 4.92$ per cent.

ILLUSTRATION 23

Velz' Nomograph To Determine The Reaeration Coefficient

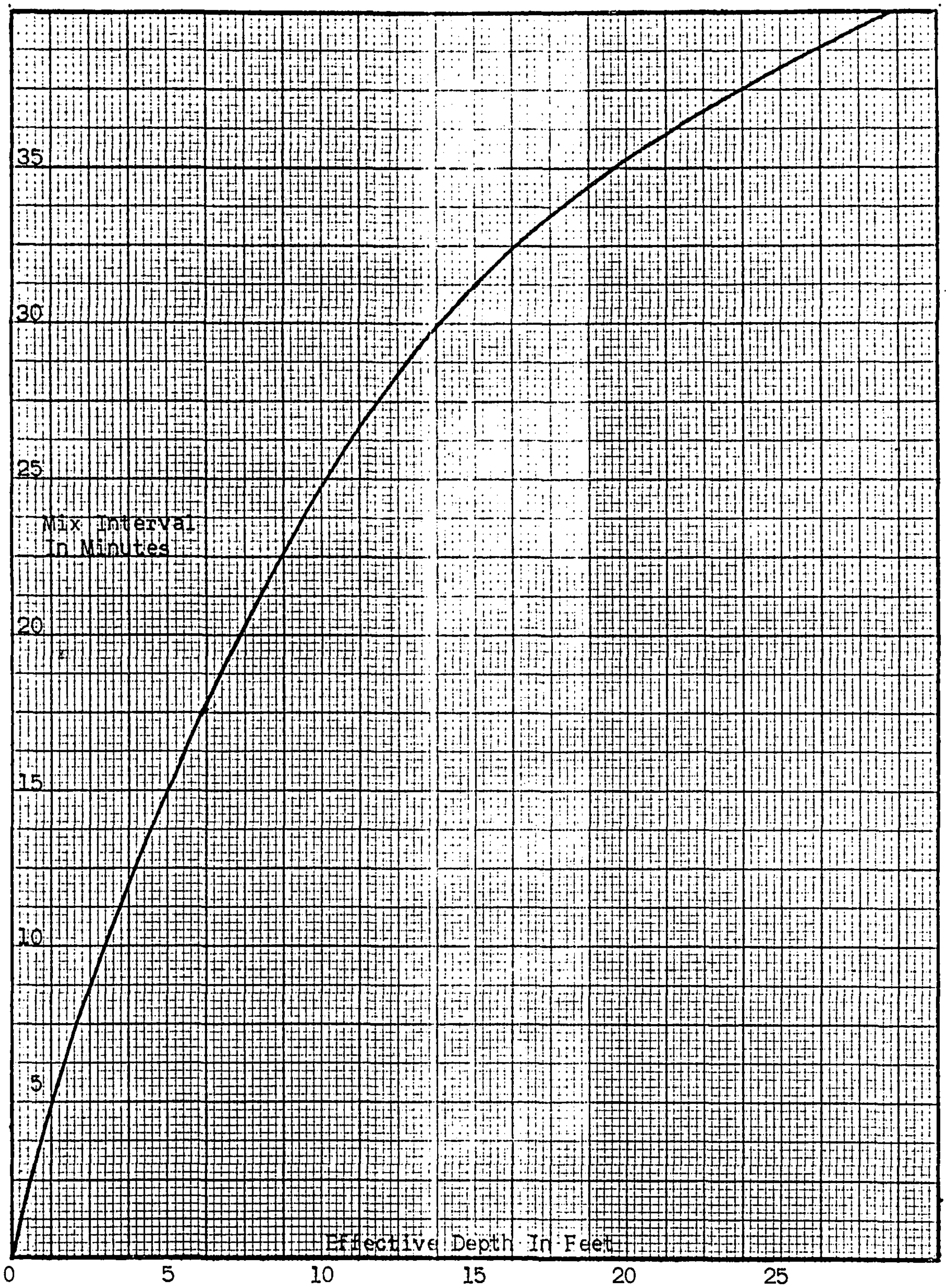


ILLUSTRATION 24

Relationship Of Mix Interval To Depth

in which $\left[\frac{\delta c}{\delta y}\right]_1$ = concentration gradient through the gas film

$\left[\frac{\delta c}{\delta y}\right]_2$ = concentration gradient through the liquid film

$\left[\frac{\delta c}{\delta y}\right]_3$ = concentration gradient in the body of the liquid below the liquid film

D_G = molecular diffusivity of the gas through the gas film

D_L = molecular diffusivity of the gas through the liquid film

and D_e = eddy diffusion coefficient of the gas in the body of the liquid.

This equation can be seen to be true if it is assumed that an equal mass transfers through all three stages in a given period of time.

That the control of the entire process is based on the liquid film may be seen from the following approximate values of the coefficients:

D_G = 0.70 square feet per hour

D_L = 0.00008 square feet per hour

and D_e = 50. square feet per hour.

From the above and from other data and references O'Conner concludes:

(1) The liquid film at the water surface is the controlling factor in the process.

(2) The concentration of the dissolved oxygen may be taken as uniform throughout the entire body.

(3) The effect of turbulence on the liquid film at the surface must be evaluated.

From theory which was originally developed for gas absorption theory by Danckwerts, O'Conner shows that

$$K_L = (D_L\, r\,)^{\frac{1}{2}} \tag{4.11}$$

where K_L is the liquid film coefficient, and r is the rate of surface renewal.

In the equation $\frac{dD}{dT} = -K_2D$ which is the recovery rate with no BOD present, K_2 is represented by

$$K_2 = K_L \frac{A}{V} \tag{4.12}$$

where A = area through which diffusion occurs

and V = volume of the liquid.

This formula may also be represented as $K_2 = \frac{K_L}{H}$ where H equals the mean depth. Therefore, from the above equation for K_L the following is obtained:

$$K_2 = (\frac{D_L\, r}{H})^{\frac{1}{2}} . \tag{4.13}$$

For this equation to be applied the rate of surface renewal must be related to the turbulence of a river and expressed in measurable parameters.

In turbulent flow, momentum, mass, heat, or any inherent characteristic can be transferred from one layer of the fluid to another. Therefore, a method of evaluation may be used which determines the rate at which particles at the surface layer can be replaced by particles arising from the turbulent motion in the body of the fluid. The intensity of the turbulence may be defined by some mean measure of the velocity fluctuations such as $\overline{V}$ and the scale of the turbulence may be defined by the mixing length, ℓ. The mixing length signifies a distance which a particle moves from its point of departure from the mean foreword motion until it mixes again with the main body of the fluid. Therefore, only particles within a zone defined by a mixing length from the surface will affect the renewal of the surface. It may be reasoned that vertical flow exhibiting small length and high velocity will cause a greater rate of surface renewal than flow of great length and low velocity. Therefore, particles at the surface are replaced at a rate directly proportional to the scale of turbulence. Surface renewal may be considered to take place in a period of time defined by:

$$t = \frac{\ell}{\overline{V}} \qquad (4.14)$$

where the values of ℓ and $\overline{V}$ are those prevailing in the vicinity of the surface.

In considering this turbulence O'Conner considers two different cases, namely isotropic and non-isotropic. An isotropic river has a uniform velocity throughout the depth of the stream and is typical of comparatively deep channels. The non-isotropic condition implies a vertical velocity gradient and is typical of shallow streams. O'Conner assumes a five foot mean depth as the dividing line between the two.

For the non-isotropic condition he solves for the surface renewal value theoretically and arrives at the formula:

$$r = \left(\frac{H\, g\, S}{K\, H}\right)^{\frac{1}{2}} \qquad (4.15)$$

where g = the acceleration of gravity

S = the slope of the river channel

and K = the Von Karmon universal constant which is generally taken as .4.

Substituting this value of r into the K_2 equation yields

$$K_2 = \left[\frac{D_L\ H^{\frac{1}{2}}\ g^{\frac{1}{2}}\ S^{\frac{1}{2}}}{K\ H}\right]^{\frac{1}{2}} \left[\frac{1}{H}\right] = \frac{g^{\frac{1}{4}}}{K^{\frac{1}{2}}} \left[\frac{D_L^{\frac{1}{2}}\ S^{\frac{1}{4}}}{H^{\frac{1}{4}}}\right] \left[\frac{1}{H}\right]. \qquad (4.16)$$

If $g = 32.2$ ft./sec.^{2} = 32.2 $(86{,}400)^2$ ft./day^2, then

$$K_2 = \frac{(32.2\ (86{,}400)^2)^{\frac{1}{4}}}{(.4)^{\frac{1}{2}}} \left[\frac{D_L^{\frac{1}{2}}\ S^{\frac{1}{4}}}{H^{5/4}}\right]$$

which equals

$$K_2 = \frac{1110\ D_L^{\frac{1}{2}}\ S^{\frac{1}{4}}}{H^{5/4}}. \qquad (4.17)$$

In this equation

D_L = coefficient of modular diffusion (sq./ft. per day)

S = slope of river channel (ft./ft.)

H = average depth (ft.)

and K_2 = the reaeration coefficient (day^{-1}).

The isotropic case has been evaluated by use of measured field data. It has been found that the mixing length is approximately 10% of the average depth and that the vertical velocity fluctuation is approximately 10% of the mean flow velocity.(32) Thus,

$$r = \frac{0.10\ V}{0.10\ H} = \frac{V}{H} \qquad (4.18)$$

where V is the mean flow velocity.

Substituting in the equation for K_2 yields:

$$K_2 = \frac{D_L^{\frac{1}{2}}\ V^{\frac{1}{2}}}{H^{3/2}} \cdot \qquad (4.19)$$

The coefficient of diffusion, D_L, has been found to be equal to 81 (10^{-6}) at 20 C.. An example of the use of these formulas for an identical condition is shown below.

Data: Velocity = .51 ft./sec. Slope = 0.167 ft./1000 ft.

Mean depth = 5.5 ft. D_L at 19° C. = 79 (10^{-6})

Non Isotropic Condition - Equation 4.7

$$K_2 = \frac{(1110\ ft^{\frac{1}{4}}/day^{\frac{1}{2}})\ (79(10^{-6})ft^2/hr.)(24\ hr/day)^{\frac{1}{2}}\ (.167/1000\ ft/ft)^{\frac{1}{4}}}{(5.5\ ft.)^{5/4}}$$

$$K_2 = \frac{1110\ (.0436)\ (.1133)}{8.4}\ \frac{1}{day}$$

$$K_2 = .651\ day^{-1}$$

Isotropic Condition - Equation 4.19

$$K_2 = \frac{(79\ (10^{-6})\ ft^2/hr)(24\ hr/day)\ (.51\ ft./sec.)(86,400\ sec./day)^{\frac{1}{2}}}{(5.5\ ft.)^{3/2}}$$

$$K_2 = .707\ day^{-1}$$

The K_2 values obtained by the use of the O'Conner formula have been compared with K_2 values determined by field sampling for a number of rivers. These include the Clarion, Little Tennessee, Watauga, Scioto, Holston, French Board, Illinois, and Ohio Rivers and San Diego Bay. The data from the Little Tennessee, Watauga, Holston, and French Board Rivers is of special significance. The water considered was oxygen depleted discharge from reservoirs which contained no oxygen demand pollutants. Since observations were made at night, there was little or no error due to photosynthesis.(33)

In a recent study, the U.S. Public Health Service computed K_2 values for a number of stations on the Ohio River using both the Velz and O'Conner methods. Their results showed a close agreement between the two methods.(23)

The O'Conner formulas have been chosen for use in the application of this dissertation because the data required for them is available and because the formulas are easily programmed for digital computer application. They also can be more easily evaluated in the river tidal zone.

The reaeration coefficient, K_2, is temperature dependent. The K_2 value at temperature T is expressed as:

$$K_2\ (T) = K_2\ (20^\circ\ C)\ \left[\theta^{(T - 20^\circ\ C)}\right]. \qquad (4.20)$$

In the past θ has usually been taken as approximately 1.016, but a recent study by the Tennessee Valley Authority has indicated that 1.0241 is a more accurate value. The latter value will be used in this dissertation.(1, 34)

The reaeration coefficient also changes when the salinity of the water changes. A recent British study has studied this effect and has found that the reduction in the rate of aeration due to salinity can be represented by the formula

$$\text{Reduction} = .059\ (\text{Salinity})^{\frac{1}{2}}$$

or

$$\%\ \text{Reduction} = 5.9\ (\text{Salinity})^{\frac{1}{2}} \qquad (4.21)$$

where the salinity is expressed in parts per thousand.(35)

In the future another factor may become very important in evaluating K_2. This is the effect on reaeration caused by detergents, soaps, and oils. Their importance stems from their altering of the surface tension of the water surface. As the discharge of these products to the stream increases it will probably be necessary to modify K_2 to show their effect.(36)

The Dissolved Oxygen Deficit

In Chapter II it was shown that the oxygen available for waste assimilation could be represented by equation 2.12:

$$AO = \Delta Q(\text{Any DO} - RQS) + K_2\, D_{RQS}\, V$$

where AO = available oxygen (lbs./day)

ΔQ = flow change within the segment (million cubic feet per day)

K_2 = reaeration coefficient (day^{-1})

V = volume of segment (million cubic feet)

Any DO = dissolved oxygen content of Q (pounds per million cubic feet or 62.4 (P.P.M.))

RQS = River Quality Standard (pounds per million cubic feet or 62.4 (P.P.M.))

and D_{RQS} = $DO_{saturated}$ - RQS (pounds per million cubic feet or 62.4 (P.P.M.).

All of these terms have been discussed or evaluated except the saturation dissolved oxygen content, the RQS, and the dissolved oxygen content of the flow change. These will be discussed in turn.

The solubility of oxygen in water, or synonomously, the saturation dissolved oxygen content in water, as a function of temperature and salinity has been discussed considerably in the last few years. Up to this time the values stated in Standard Methods were generally accepted as accurate.(37) However, the work of Truesdale and Vandyke, which was published in 1955, questioned these values.(38, 39)

A recent study made by Elmore and Hayes was published by the Committee of Sanitary Engineering Research of the American Society of Civil Engineers. From an extensive series of tests, they arrived at the formula given below for the solubility of oxygen in fresh water at temperature T. The Elmore and Hayes formula is:

$$\text{Dissolved oxygen (P.P.M.)} = 14.652 - 0.41022T + 0.0079910T^2 - 0.00007777T^3 \qquad (4.22)$$

The Truesdale formula has a similar form, but it also includes terms to adjust for the salinity. The salinity S is expressed in parts per thousand. The Truesdale formula is:

$$\text{Dissolved oxygen (P.P.M.)} = 14.161 - 0.3943T + 0.007714T^2 - 0.0000646T^3 - S(0.0841 - 0.00256T + 0.0000374T^2) \qquad (4.23)$$

In the 25 to 30 degree centigrade range the two formulas have comperable results; however, below that range they differ.

A combination of the two formulas are used arbitrarily in the applications made herein. Where there is no salinity, the Elmore and Hayes formula is used; and where salinity is present, the Elmore and Hayes formula is combined with the Truesdale salinity correction factor.

It is believed that this procedure will yield results well within required accuracy limits.

The River Quality Standard is the lowest dissolved oxygen content which is allowable in a river. This value is usually established by law or by the governmental agencies charged with the regulation of stream quality. The particular value established for a river is based on the desired use of the river and on fish and wildlife considerations. A dissolved oxygen content of four parts per million is generally accepted as being adequate to support fish and other aquatic life, and therefore, this value will be used as the RQS in all the applications in this dissertation.

The dissolved oxygen content of added flow within a river segment may be at any value and hence the labeling as Any DO. Since this dissolved oxygen content can range from zero for a sewage effluent or reservoir discharge to saturated, or even supersaturated, for fresh water inflow, it is necessary to either have actual data about this content or to make rather general assumptions regarding it. It will be assumed that all inflow is at saturation in the dissertation applications.

If the flow change in the segment is negative indicating a loss of flow, this flow loss would have a dissolved oxygen content equal to the River Quality Standard.

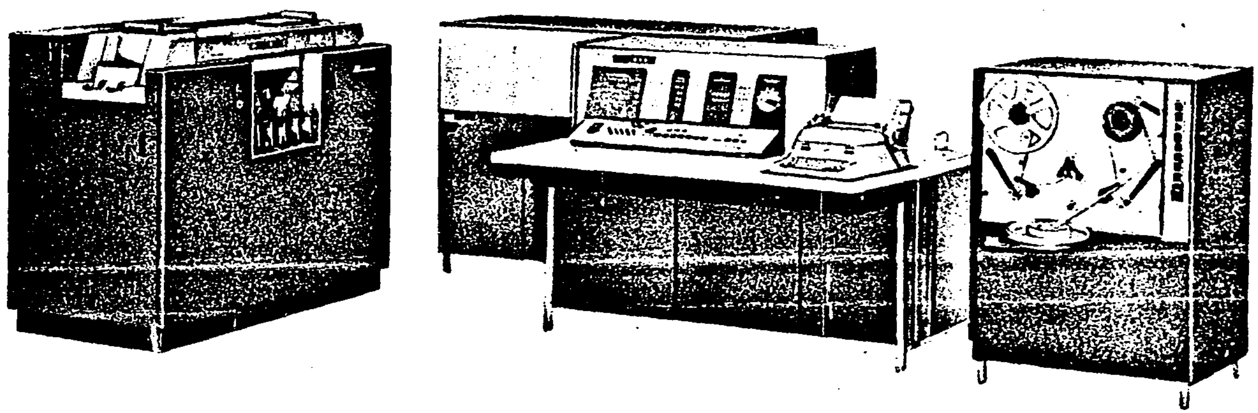
CHAPTER V

COMPUTER EQUIPMENT AND PROGRAMS USED TO DETERMINE WASTE ASSIMILATION CAPACITY

The procedures proposed in this dissertation are primarily designed for use on modern high speed digital computers. It is believed that this is one of the first, if not the first, application of digital computers to organic stream pollution.

Two I.B.M. 1620 computers were used on the project. The majority of the work was done on the unit furnished by the kindness and courtesy of the Biostatistics unit of the University of Oklahoma Medical School. This unit consisted of the 1620 console, 1622 card read punch unit, and a 1623 Core storage unit which expanded the machine storage capacity from 20,000 to 40,000 digits. Illustration 25 shows pictures of a similar system. The 1620 used on the main campus of the University of Oklahoma is similar to the one at the medical school except that it does not have the additional Core storage unit. These units usually rent commercially for $25 to $35 per hour.

All of the programs were written in the language accepted by the Fortran With Format compiler for translation into numerical machine language. As this system of programming is in universal use, it will not be described here. I.B.M. manuals are available which describe the



IBM 1620 Data Processing System

ILLUSTRATION 25

IBM 1620 Computer

process thoroughly.(40, 41, 42)

Three types of input and output format are utilized by the 1620. These are shown below with examples.

Name	Designation	Number	Format	Expressed in Format
Integer	I	+100	I4	+100
Fixed Point	F	+100.0	F6.1	+100.0
Fixed Point	E	+100.0	E10.3	+1.000E+02

An integer number is merely represented by an I plus the number of digits. The F type floating point number designated F6.1 indicates that the number requires six spaces, one for the plus sign, one for the decimal, and four for the number. The number one after the decimal indicates one number is to be to the right of the decimal.

The E type designation is an exponential type expression. E10.3 means that it will take ten spaces to write the number in E format. The three means the base number is to have three digits to the right of the decimal point. The number +1.000E+02 in effect means 1.000×10^2 which would also equal 100.0.

All three types of format were used for program input. Most output numbers for final programs were Integer and F type numbers.

Data Generating Program - Program Number One

As will be discovered later, the computer programs which were used to evaluate the waste assimilation capacity of a river were made as flexible as possible. As a minimum to operate the program, the only data needed for a single river is data at the mouth and any upstream point. If the stream has tributaries, it is further required that no two tributaries shall join the main stream unless data is furnished for a point on the main stream between the mouths of the tributaries. The program will accept data for segments as short as one mile.

The data available for the Ohio River and its tributaries was for locations varying from 10 to 60 or more miles apart. The data in these varying segment lengths could have been furnished to the program, and it would have computed values from this widely scattered data. However, since there is often a great change in flow and channel properties in a long segment, it was deemed advisable to select a shorter segment length and to compute average data for the points between known data points. It was believed that this would give greater accuracy to the program, and it would permit evaluation of allowable loadings for shorter lengths within the segment.

The Data Generating Program was written for the purpose of computing this data at in between points and also for computing stream properties at points where flow was added to a stream at a point between two stations. The Data Generating Program also served the purpose of computing the slope from the elevations of the river channel at the stations.

The complete Data Generating Program and flow chart are shown in the Appendix as Program One. In general the computer is instructed to:

(1) Read two data sets of two cards each,

(2) Compute the channel slope between the two data sets,

(3) Punch two cards containing data set one plus the slope,

(4) Test to see if the distance between the two stations is greater than the desired segment length,

(5) If not, read next data set and repeat the process,

(6) If length is longer, then compute and punch the data for each station one segment length larger than the previous station until the distance between stations is smaller than the segment length, and then read the next data set and repeat the process.

The switching and storage used to test for and compute the data for incomplete stations may be observed in the flow chart.

This program is designed to handle only a single river without any tributaries. Once the data for all rivers in a basin have been processed in this manner, they are reassembled in proper tributary order for use in the Waste Assimilation Program.

As mentioned above, the input and output format from the Data Generating Program is the same. In both cases it consists of two cards of 72 digits. In processing the program the data cards are preceded by a single card which gives the desired segment length in Format I4, E12.5. For example, for a ten mile segment length this card would be punched +010+1.00000E+01 in the first 16 spaces of a card.

The format of the input data and example values are shown in Table 4. This data is for the Pricetown, Ohio station on the Mahoning River which is the same station which will be used to show data analysis methods in Chapter VI. Illustration 26 shows I.B.M. cards punched with this data.

The flag in digit ten is of use only to the Data Generating Program, and it merely indicates that the data is complete. It is zero for complete cards and one for those which are incomplete. Only the total

TABLE 4

DATA FORMAT - WASTE ASSIMILATION PROGRAM

Space On Card	Format	Information	Example
CARD 1			
1-3	I3	River Number	006
4-8	I5	River Mile	00068
9	I1	Zero	0
10	I1	Flag	0
11-12	I2	River Designation	01
13-24	E12.5	River Mile	+6.80000E+01
25-36	E12.5	Temperature	+2.50000E+01
37-48	E12.5	River Quality Standard	+4.00000E+00
49-60	E12.5	Flow	+1.57000E+02
61-72	E12.5	Flow Added At Station	+0.00000E+00
CARD 2			
1-12	E12.5	Width	+1.93000E+00
13-24	E12.5	Area	+2.07000E+02
25-36	E12.5	Elevation	+9.05000E+02
37-48	E12.5	Slope	+0.00000E+00
49-60	E12.5	Initial Deficit	+0.00000E+00
61-72	E12.5	Added BOD	+0.00000E+00

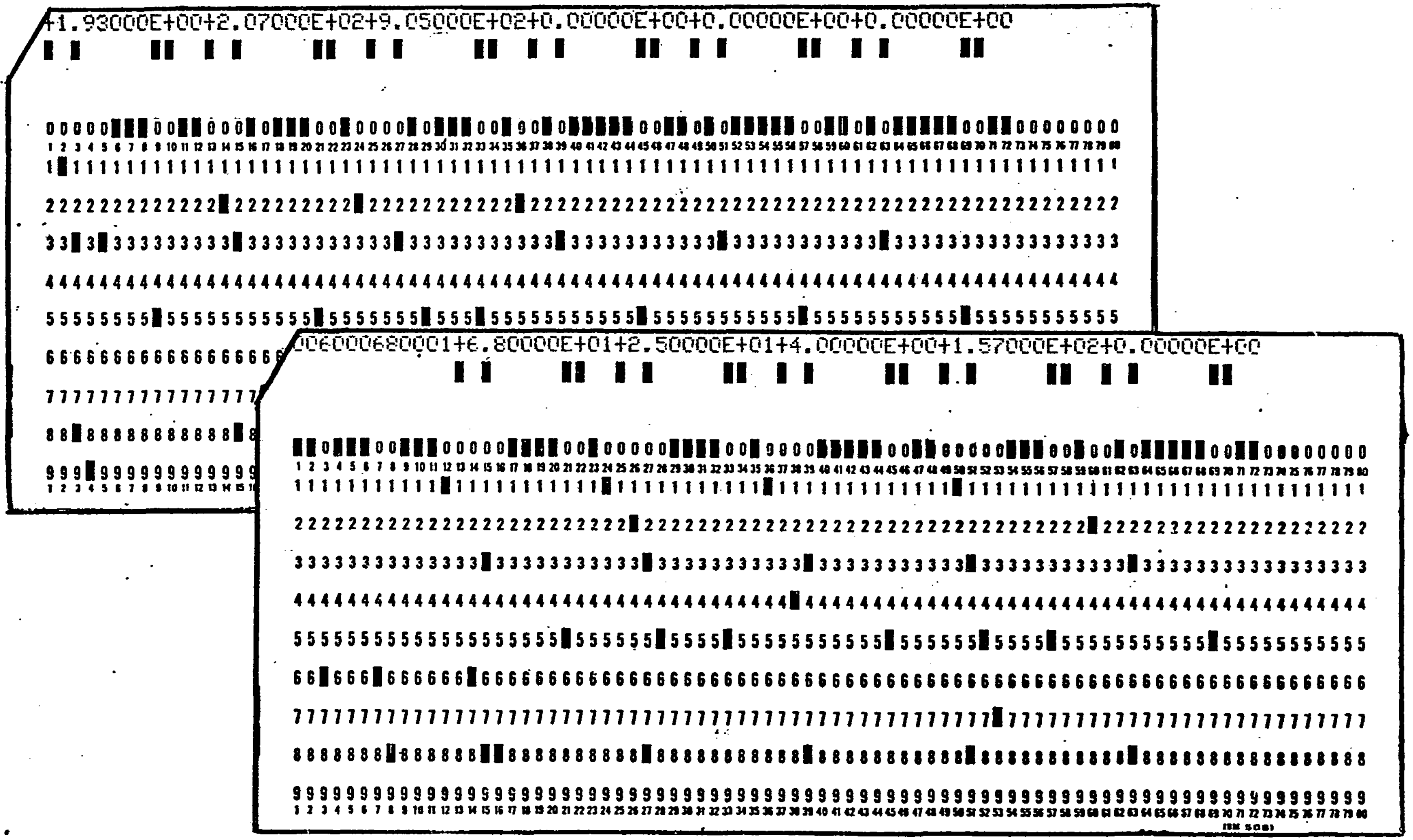


ILLUSTRATION 26

Input Data Cards

flow and channel property data can be incomplete. Digits 11 and 12 indicate whether the river is a main stream (00), primary tributary (01), or a secondary tributary (02). River number 6 is the Mahoning River. Mile 68 is at Pricetown, Ohio. The output card for this station will be the same as the input card except the slope will be computed and punched.

The Data Generating Program requires a time of only one second per data station to read the cards, compute the information, and punch the output cards. Computer time of only forty minutes was needed to process the data for the entire Ohio River basin. This time could have been shortened, but it was desired to punch the main stream, primary tributaries, and secondary tributaries' data on different colored cards to facilitate easier assembly for use in the Waste Assimilation Program. This required loading the machine language program and Fortran Subroutines into the computer three separate times.

Waste Assimilation Program - Program Two

The Waste Assimilation Program is designed to compute the total allowable BOD which a river system without tidal action can assimilate if it is continually loaded to maximum capacity throughout its length.

The calculations made for each segment of the river and the summation for the entire river are those outlined by the theory for the non-tidal river presented in Chapter II and by the analysis of the various coefficients presented in Chapter IV. The program further includes rather complicated switching and storage routines which make it possible for the program to evaluate and integrate together both primary and secondary tributaries as well as the main stream.

The program is designed so that it can handle a river basin of up to 200 streams, each of a length up to 9999 miles. The only limitations are that no tributary can be of a higher order than a secondary tributary and that two consecutive primary tributaries shall not join the main stream unless data is furnished for a data point on the main stream located between the mouths of the two tributaries. This restriction likewise applies to the flow of secondary tributaries into the primary tributaries.

The mouth of every river and tributary is numbered as mile number 0000, and complete data must be furnished to the computer for this station. Other stations upstream are numbered in terms of river miles above the mouth of the river.

The computer program is designed to begin processing at the upstream end of the main stream and to proceed downstream towards the mouth. Tributaries and sub-tributaries are calculated as they are reached. In other words, calculation will begin at the head water of the main stream and will continue downstream until the first tributary is reached. The program will then calculate the values for the tributary and then return to the main stream.

For the purpose of discussing the program mechanics in the text, it will be assumed that we are considering only a single river without tributaries. The switching and storage procedures are shown in detail in the program flow chart in the Appendix should they be of interest.

The procedure followed by the computer in carrying out the commands of the Waste Assimilation Program is as follows:

(1) Punch column headings and set answer values equal to zero,

(2) Read the first data set and type initial conditions,

(3) Read second data set,

(4) Compute average values for the flow, area, width, velocity, temperatures, etc. for the segment,

(5) Compute the saturated dissolved oxygen value and the deficit between it and the RQS,

(6) Compute the detention time of the river in the segment,

(7) Set K_1 and modify it according to temperature and according to the time spent in the segment,

(8) Compute K_2 from the channel properties using the O'Conner-Dobbins formulas and modify it according to the temperature,

(9) Compute the total available oxygen which enters the segment per day by reaeration and by the addition of flow which has dissolved oxygen in excess of the RQS,

(10) Compute BOD load left from previous segment,

(11) Compute total BOD which can be applied to this segment,

(12) Compute the allowable BOD as being equal to the total BOD minus the previous BOD,

(13) Compute sum of passage time, allowable BOD for river, and allowable BOD for basin,

(14) Punch river number, river mile, passage time, RQS, allowable load in segment, sum of loads for river, total load for basin, mean flow, and K_2,

(15) Store values needed to compute next segment; i.e. total BOD of this segment, K_1 of this segment, etc., and

(16) Read the next data set and repeat the process until the river

mouth has been reached.

1980 Ohio River Study

The United States Public Health Service has recently completed a study to estimate the stream conditions of the Ohio River for the year 1980. Since their data for channel conditions and flows, as well as their results, were available, it was decided to apply this data to the Waste Assimilation Program to determine the waste assimilation capacity under these conditions. By so doing a test was provided for the mechanics of the program, and a specific check was available for one of the components of the program, namely the K_2 coefficient.

The data was processed through the Data Generating Program to obtain the data for each ten mile segment length, and this completed data was then furnished to the Waste Assimilation Program.

It was found that the K_2 values computed by the Waste Assimilation Program agreed closely with those of the P.H.S. study. The program results indicated that at these flows the Ohio River will be able to assimilate a BOD load of 3,865,364 pounds in the 974 mile stretch considered in this program. Assuming 30% sewage treatment this would be the waste load contributed by 22,000,000 people.

The use of the Waste Assimilation Program to process this 1980 data furnished by the U.S. Public Health Service will be referred to as the 1980 Ohio River Study.

The river flow time was checked by comparing it with a recent study entitled Time of Travel of Water in the Ohio River, Pittsburg to Cincinnati, which was published by the U.S. Geological Survey.(43)

It was found that the values were in good agreement with the ones obtained in this study.

Additional testing of the computer program was made by means of a trace routine which printed out the answers of every numerical calculation made by the computer. This made it possible to quickly and easily check the results of each program step.

The processing speed of the Waste Assimilation Program has proved to be very rapid. It is able to completely process a segment of the river in eleven seconds. It required only twenty minutes to load the program into the computer and to process the entire data for the 1980 Ohio River Study.

Tidal Zone Waste Assimilation Program - Program Three

The program used to determine the waste assimilation capacity in the tidal zone is very similar to the one used for the non-tidal zone. It has been simplified considerably, however, by the fact that it is only to consider a single river and by the elimination of the non isotropic K_2 formula.

Since it will be shown in Chapter VI that data can usually be obtained or approximated for any point desired in the tidal zone and since the tidal zone is shorter than most rivers without tides, no Data Generating Program is provided.

Normally a study of a river with a tidal zone will include the use of the Waste Assimilation Program for the stretch of river above the head of tide point and the Tidal Zone Waste Assimilation Program for the tidal zone. It is desirable to designate the river miles on the upstream portion so that the head of tide is river mile zero.

If this is done, the computer will punch out the total segment BOD load and the K_1 value for the segment above the tidewater point. This information and the river and river basin allowable BOD sums are then fed as the first data card to the Tidal Zone Waste Assimilation Program in order to provide continuous analysis for the entire river, including the tidal zone.

The input data for the Tidal Zone Waste Assimilation Program consists of an initial card which gives the K_1 value and BOD sums from the upstream segment, and two cards for each segment to be considered. The two data cards for each segment are different in content and format from those for the upstream river. The format and example values for the initial card and segment data cards are shown in Table 5.

The ebb and flood currents correspond to the V_1 and V_2 values which were used in the development of the mean effective velocity as outlined in Chapter III. Similarly, ebb time and flow time refer to T_1 and T_2 respectively. The salinity value is furnished as parts per thousand.

As with the other programs a detailed flow chart and program listing are shown in the Appendix. The general steps which the program commands the computer to do are as follows:

(1) Read upstream values,

(2) Punch column headings and set initial answer values,

(3) Read first data set and type initial conditions,

(4) Read second data set,

(5) Compute average value for flow, area, width, salinity, etc.,

TABLE 5

DATA FORMAT - TIDAL ZONE WASTE ASSIMILATION PROGRAM

Space in Card	Format	Information	Example
INITIAL CARD			
1-10	E10.3	TABOD	+3.657E+04
11-20	E10.3	FKIT	+0.375E+00
21-30	E10.3	SLOAD	+2.215E+05
31-40	E10.3	TLOAD	+2.215E+05
DATA CARD 1			
1-3	I3	River Number	001
4-7	I4	River Mile	0100
8	I1	Zero	0
9-18	E10.3	River Mile	+1.000E+02
19-28	E10.3	Temperature	+2.700E+01
29-38	E10.3	River Quality Standard	+4.000E+00
39-48	E10.3	Flow	+2.851E+03
49-58	E10.3	Flow Added At Station	+0.000E+00
59-68	E10.3	Initial D.O. Deficit	+3.780E+00
DATA CARD 2			
1-10	E10.3	Width	+1.990E+01
11-20	E10.3	Area	+6.770E+04
21-30	E10.3	Ebb Current	+1.950E+00
31-40	E10.3	Time of Ebb	+7.200E+00
41-50	E10.3	Flood Current	+2.060E+00
51-60	E10.3	Time of Flood	+5.300E+00
61-70	E10.3	Salinity	+0.000E+00

(6) Compute effective mean velocity from current and time data,

(7) Compute saturated dissolved oxygen value and the deficit between it and the RQS,

(8) Compute the detention time of the river in the segment,

(9) Set K_1 and modify it according to temperature, salinity, and time,

(10) Compute K_2 and modify it for temperature and salinity,

(11) Compute the total available oxygen which enters the segment per day by reaeration and by the addition of flow which has dissolved oxygen in excess of the RQS,

(12) Compute BOD left from previous segment,

(13) Compute total BOD which can be applied to this segment,

(14) Compute the allowable BOD as being equal to the total BOD minus the previous segment's left over BOD,

(15) Compute sum of passage time, allowable BOD for river, and allowable BOD for river basin,

(16) Punch river number, river mile, passage time, RQS, allowable load in segment, sum of allowable load for river, sum of allowable load for basin, and K_2,

(17) Store values from this segment needed to compute values in the next one, and

(18) Read the next data set and repeat the process until the mouth of the river has been reached.

The Tidal Zone Waste Assimilation Program was checked for arithmetic accuracy by the use of the trace routines mentioned in connection with the Waste Assimilation Program. Comparable studies which would permit further checking were not found.

The program execution speed was again found to be rapid. A data set is processed in approximately ten seconds. It took less than ten minutes to load the program and process the entire tidal zone of the Delaware River.

CHAPTER VI

COLLECTION AND ANALYSIS OF DATA

It is expected that the greatest immediate use for the methods proposed in this dissertation will be in the field of water resources planning. In other words these methods will be used to determine the ultimate BOD loading which a stream can assimilate without violating the River Quality Standard. This will permit the planning board to evaluate waste treatment standards and flow augmentation requirements for future years. In planning studies of this type it would be too expensive to actually make river studies to obtain the necessary data, and indeed one of the major reasons for the development of these methods is to provide planning estimates without actual river studies.

Fortunately, much data is already in existence and is available for use in such a study. This data is found in the publications and files of several government agencies, namely the United States Geological Survey (U.S.G.S.), the United States Coast and Geodetic Survey (U.S.C.G.S.), the United States Army Corps of Engineers (U.S.A.C.E.), the United States Public Health Service (U.S.P.H.S.), and the Weather Bureau.

The information necessary to analyze a particular segment of a river without tidal action is as follows:

(1) The selection of the segment length and the location of the ends of the segment in terms of river miles above the river mouth,

(2) The river flow to be considered,

(3) The mean depth of the water,

(4) The cross section area,

(5) The mean velocity,

(6) The channel slope,

(7) The water temperature, and

(8) The initial dissolved oxygen deficit of any added flow.

In the tidal zone of the river the following additional data must be furnished assuming that the salinity portion exhibits homogeneous type characteristics:

(1) Mean maximum ebb flow through section,

(2) Duration time of ebb flow,

(3) Mean maximum flood flow through section,

(4) Duration of flood flow, and

(5) Salinity.

The above data must be available at enough points throughout a river basin to provide sufficient accuracy by using average values between stations. Each of these data requirements will be discussed in turn.

Segment Length And River Mile Designation

The length of the segment to be used in the calculation of the waste assimilation capacity of a river is arbitrary, but its selection should depend on three factors: (1) the abundance and uniformity of the available data, (2) the desired accuracy of results, and (3) the cost of computation.

For example, if fairly uniform data is available for sections ranging from 10 to 30 miles apart on a river, a segment length of 5 or 10 miles would probably provide sufficiently accurate results at a reasonable cost. For this same data a 1 mile segment length would not give much better accuracy, and the cost of computation would be increased 5 fold or 10 fold. If, on the other hand, a short river or tidal zone with extensive data were being considered, a segment length of 1 or 2 miles might be preferable.

Segment lengths of 10 miles for rivers without tides and 5 miles for the river tidal zones were used in the Ohio River Basin and Delaware River studies which will be mentioned in Chapter VII.

For use in the methods of this dissertation every data station on a river, tributary, and sub-tributary must be designated in terms of river miles above the mouth of its respective river. For navigable rivers the U.S. Army Corps of Engineers has already assigned such a designation, and the river mile number for any point in question can be read or scaled from their river navigation charts.(44)

For some rivers the U.S. Geological Survey has determined the river mile numbers for their gaging stations, and these values may be found in their Water Supply Papers in the paragraphs describing the location of each station. For upstream points for which neither the Corps of Engineers maps or the Water Supply Papers provide river mile designation, it is necessary to measure or estimate these values from U.S. Geological Survey topographic maps or detailed highway maps.(45)

Often special reports which have been published concerning particular rivers yield data or river mile location.(2, 14, 46, 47, 48, 49, 50)

In tidal rivers the river mile designation can be read or scaled from either U.S. Army Corps of Engineers navigation charts or from the nautical charts prepared by the U.S. Coast and Geodetic Survey.

River Flow

The most extensive data available for stream flow in non-tidal streams is that of the U.S. Geological Survey. This agency currently maintains more than 7000 gaging stations on the rivers in the United States. A summary of the mean daily flows, mean monthly flows, and mean yearly flows are published yearly for each of these stations.

For publications and collection purposes the continental United States has been broken up into 18 regions, each one of which is administered separately and for which the data is published in a separate Water Supply Paper.(51, 51) The various areas are:

1. North Atlantic slope basins, in two volumes:
    A. North Atlantic slope basins, Maine to Connecticut.
    B. North Atlantic slope basins, New York to York River.
2. South Atlantic slope and eastern Gulf of Mexico basins, in two volumes:
    A. South Atlantic slope basins, James River to Savannah River.
    B. South Atlantic slope and eastern Gulf of Mexico basins, Ogeechee River to Pearl River.
3. Ohio River basin, in two volumes:
    A. Ohio River basin except Cumberland and Tennessee River

basins.

B. Cumberland and Tennessee River basins.

4. St. Lawrence River basin.

5. Hudson Bay and upper Mississippi River basins.

6. Missouri River basin, in two volumes:

   A. Missouri River basin above Sioux City, Iowa.

   B. Missouri River basin below Sioux City, Iowa.

7. Lower Mississippi River basin.

8. Western Gulf of Mexico basins.

9. Colorado River basin.

10. The Great Basin.

11. Pacific slope basins in California.

12. Pacific slope basins in Washington and upper Columbia River basin.

13. Snake River basin.

14. Pacific slope basins in Oregon and lower Columbia River basin.

An example of the data that is published yearly for each U.S. Geological Survey gaging station is shown in Illustration 27.

Unfortunately the U.S. Geological Survey data is available for only that portion of the river which is unaffected by tides. For the tidal zone fresh water flow estimates must usually be obtained by determining runoff coefficients for the area of the basin for which the runoff is known, and then apply this coefficient to the area of the basin which contributes runoff to the point considered.

Several other governmental agencies maintain gaging stations throughout the United States. The location and jurisdiction of these

BEAVER RIVER BASIN

915. Mahoning River at Pricetown, Ohio

Location.--Lat 41°07'50", long 80°58'24", in T.2 N., R.5 W., on left bank a quarter of a mile south of Mahoning-Trumbull County line, 0.3 mile downstream from Milton Dam, half a mile southwest of Pricetown, Mahoning County, and 3 miles upstream from Kale Creek.

Drainage area.--275 sq mi.

Records available.--July 1929 to September 1960.

Gage.--Water-stage recorder. Datum of gage is 905.00 ft above mean sea level, adjustment of 1912. Prior to Aug. 14, 1929, staff gage at same site and datum.

Average discharge.--31 years, 255 cfs (unadjusted).

Extremes.--Maximum discharge during year, 2,540 cfs Apr. 7 (gage height, 7.98 ft); minimum, 33 cfs Mar. 15-27 (gage height, 1.50 ft).

1929-60: Maximum discharge, 6,770 cfs Jan. 25, 1937 (gage height, 15.01 ft), from rating curve extended above 4,200 cfs on basis of velocity-area studies; minimum, 0.4 cfs Nov. 9, 10, 1941, Feb. 19-21, Oct. 10-21, 1945.

Remarks.--Records good. Flow regulated by Berlin Reservoir beginning 1942 and Milton Reservoir throughout the period of record (see p. 129).

Revisions (water years).--WSP 728: 1930(M).

Rating tables, water year 1959-60 (gage height, in feet, and discharge, in cubic feet per second)

Oct. 1-10		Oct. 11 to Feb. 13		Feb. 14 to Sept. 30			
2.4	172	3.1	330	1.5	33	3.0	325
2.7	245	3.5	450	1.7	59	4.5	850
3.1	340	4.0	625	1.9	89	6.0	1,500
		5.0	995	2.1	122	7.7	2,380
				2.5	202		

Discharge, in cubic feet per second, water year October 1959 to September 1960

Day	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
1	195	357	444	468	496	126	201	80	62	212	236	212
2	193	357	444	468	411	94	524	80	62	212	236	212
3	193	357	441	468	363	94	524	80	80	212	236	212
4	190	357	441	468	363	94	528	80	99	212	236	212
5	190	357	441	468	363	94	930	89	100	212	236	212
6	242	354	438	468	363	94	1,400	100	136	212	236	212
7	264	354	447	468	363	94	*2,380	100	*176	212	236	212
8	261	351	454	468	363	88	1,960	102	176	224	*236	*212
9	264	351	460	468	*363	246	739	*60	187	228	221	212
10	335	354	471	471	366	626	503	42	169	231	219	212
11	366	354	*465	471	366	291	500	42	202	*231	216	212
12	366	354	488	432	366	36	264	42	209	231	216	212
13	366	354	485	414	682	36	91	42	154	231	216	212
14	363	351	685	414	870	34	68	42	105	231	216	214
15	363	348	955	*411	866	33	*68	40	103	231	214	214
16	366	*348	955	411	870	*33	70	40	216	231	214	195
17	366	414	955	414	608	33	70	40	456	231	214	169
18	366	450	955	414	463	33	70	40	528	233	214	191
19	363	450	738	414	463	33	70	40	528	233	214	191
20	360	450	478	460	457	33	70	50	313	233	214	191
21	*360	447	474	496	457	33	70	56	207	233	214	191
22	360	444	474	496	457	33	70	56	191	233	214	191
23	360	444	474	496	457	33	70	56	191	233	214	191
24	360	444	471	496	287	33	70	56	193	233	214	191
25	360	444	471	496	193	33	70	56	193	233	214	191
26	360	444	471	496	193	33	70	56	191	233	212	191
27	360	444	471	496	193	34	70	94	202	236	212	180
28	360	444	468	496	193	37	70	135	207	236	212	176
29	360	444	468	496	191	38	74	135	209	236	212	176
30	360	444	468	496	--------	37	80	135	200	236	212	176
31	357	--------	468	496	--------	36	--------	102	--------	236	212	--------
Total	9,929	11,865	16,838	14,394	12,448	2,795	11,744	2,166	6,073	7,061	6,818	5,995
Mean	320	396	543	464	429	90.2	391	69.9	202	228	220	200
Cfsm	-	-	-	-	-	-	-	-	-	-	-	-
In.	-	-	-	-	-	-	-	-	-	-	-	-

Calendar year 1959: Max 2,220 Min 4.9 Mean 391 Cfsm - In. -
Water year 1959-60: Max 2,380 Min 33 Mean 295 Cfsm - In. -

* Discharge measurement made on this day.

ILLUSTRATION 27

Daily Stream Flow Data From A Water Supply Paper stations may be determined by examining the publication River Basin Maps Showing Hydrologic Stations.(53)

The flow to be used in a pollution study is usually arbitrary. Pollution problems usually occur at times of low flow and high temperature, but as in any project dealing with nature, a degree of conservativeness must be fixed.

For the applications to be made in this study it was decided that the lowest mean river flow experienced in the last ten years during a calendar month from June to October would make a good basis of design. Since the flows were to be considered at the mean August temperature, it was further decided that if the low monthly flow was in June or October, it would be modified upward to give it credit for its lower temperature.

The low flow values for the Ohio River basin were obtained in the following manner. First the available U.S. Geological Survey gaging stations on the Ohio River and its primary and secondary tributaries were determined. Then for each year during the last ten, the monthly mean flows were examined and the lowest monthly flow and the September flow was listed. From these the lowest monthly flow in the last ten years and the lowest September flow in the last ten years was determined.

If the lowest flow found was for either July, August, or September, it was accepted as the low flow. If the lowest flow was for June or October, it was modified upwards by means of the temperature correction which will be evaluated later in this chapter. Then the modified June or October value was compared with the lowest July to September flow, and the smallest of these was accepted as the low flow.

RIVER FLOW ANALYSIS

River:..Mahoning-Beaver River Basin

Station:..U.S.G.S. 915

Nearest Town:...Pricetown, Ohio

Drainage Area in Square Miles:....................................276

Elevation in Feet (gage datum):...................................905.00

Station Location is River Mile 68

of the ....Beaver River

Flow Analysis

Water Year	Mean Flow in September	Lowest Monthly Mean Flow	Month of Lowest Flow
1951	238	195.0	October
1952	244	95.5	June
1953	226	69.0	March
1954	157	16.0	March
1955	248	96.1	April
1956	320	76.8	January
1957	241	90.6	May
1958	278	69.4	April
1959	241	102	April
1960	200	69.9	May
10 year Low Flow	157		September

Equivalent Low September Flow in
Cubic Feet per Second: 157

Channel Properties at Above Flow

Mean Velocity in Feet per Second: 0.76

Width in Feet: 107

Mean Depth in Feet: 1.93

Cross Sectional Area in Square Feet: 207

ILLUSTRATION 28.

Typical River Flow Analysis Form

An example of the flow analysis sheet used to summarize the ten year flow records is shown in Illustration 28. This analysis for station 915, Mahoning River at Pricetown, Ohio, shows a low September flow of 157 cubic feet per second which also is the low July to October flow.

Area, Depth, and Velocity

The determination of the channel and flow properties cannot be arrived at as simply as the flow. This is because the U.S. Geological Survey is primarily interested in the quantity of water which flows in a stream and is not necessarily interested in the channel geometry and velocity characteristics. They are interested in these channel properties only as a means of determining and evaluating the flow as a function of the height of the gage. In order to properly evaluate this relationship the U.S. Geological Survey makes periodic measurements of the channel cross section and corresponding velocities to determine the flow at the station. For each station the data from these investigations is tabulated on the U.S. Geological Survey form No. 9-207 which is kept in the district offices. This data is not published, but the district offices will provide copies on request. Sometimes it is necessary to pay reproduction costs. The 9-207 forms do not directly yield the information which is needed for the stations, but they do furnish data from which a graph can be plotted which will yield the desired information. One difficulty encountered in using this data is the fact that the U.S. Geological Survey does not use the same cross section for all measurements. In other words, the person making these measurements is free to choose the easiest place for the measurement although it may be upstream or downstream from the gage location. Although this was known, it was found that for the great majority of stations the measurements were either made at the same or similar sections. Sometimes measurements were noted which were completely uncompatable with the others for a station, and these were usually discarded. Illustration 29 is a copy of a 9-207 form furnished for an Ohio station.

It was found that if the logarithm of the width and the mean velocity from the 9-207 forms were plotted against the logarithm of the flow, smooth curves or straight lines could be plotted through the points. This permitted the evaluation of the width and mean velocity for the ten year low month, and from these the cross section area could be calculated. The graph for the station at Pricetown, Ohio, is shown in Illustration 30. On this graph it may be noted that almost all the points cluster closely around the lines drawn.

As noted in the river flow analysis for this station, the equivalent low September flow for this station is 157 cubic feet per second. Projecting vertically from a flow of 157 cubic feet per second to the velocity curve, a velocity of 0.76 feet per second can be read on the left scale. Similarly a channel width of 107 feet is read on the right scale. Since area equals the flow divided by the velocity, the area equals 157/0.76 or 207 square feet. The mean depth equals the area divided by the width, hence 207/107 equals 1.93 feet. By similar manner these values were determined for over a hundred stations on major rivers in the Ohio River basin.

9—207
(Sept. 1952)

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)

File No. Washington ________ Field ________

Discharge measurements of Mahoning River at Pricetown, Ohio, during the year ending Sept. 30, 1958

Date	Made by—	Width	Area	Mean velocity	Gage height	Discharge	Rating No. 1 Shift adj.	Percent diff.	Method	Number meas. sections	Gage height change	Time	Meas. rated		REMARKS
1957		Feet	Sq ft	Fps	Feet	Cfs	Feet				Feet	Hr			
Sept. 25	Francis	108	245	1.00	2.59	245	+.05	0	W .6/.2.8	28	0	11/12	G	68	
Oct. 23	Graff	107	220	.82	2.37	181	-	0	W .6/.2.8	25	0	5/6	G	57	
Nov. 21	Graff	107	183	.61	2.03	111	-	-1	W .6/.2.8	24	-.005	3/4	G	44	
Dec. 18	Francis	106	204	.75	2.24	152	-	-1	W .6/.2.8	27	0	5/6	G	35	
1958															
Jan. 22	Francis	108	247	.95	2.60	235	-	0	W .6/.2.8	28	0	11/12	G	37	
Feb. 25	Graff	107	178	.60	1.97	107	-	+6	W .6/.2.8	23	0	3/4	G		
Mar. 21	Graff	107	179	.57	1.98	102	-	0	W .6/.2.8	24	0	2/3	G	39	
Apr. 24	Francis	107	154	.44	1.78	67.8	-	0	W .6/.2.8	28	0	2/3	G	49	
May 9	Francis	130	522	2.32	5.09	1,210	-	+6	B 50#	30	-.005	1	G	55	
21	Graff	107	221	.88	2.38	195	+.05	0	W .6/.2.8	25	0	11/12	G	64	
June 24	Graff	107	235	.91	2.53	213	-	-2	W .6/.2.8	25	0	5/6	G	68	
July 29	Francis	126	336	1.95	3.83	654	-	+5	B 50#	28	0	5/6	F	71	

ILLUSTRATION 29

U. S. Geological Survey 9-207 Form

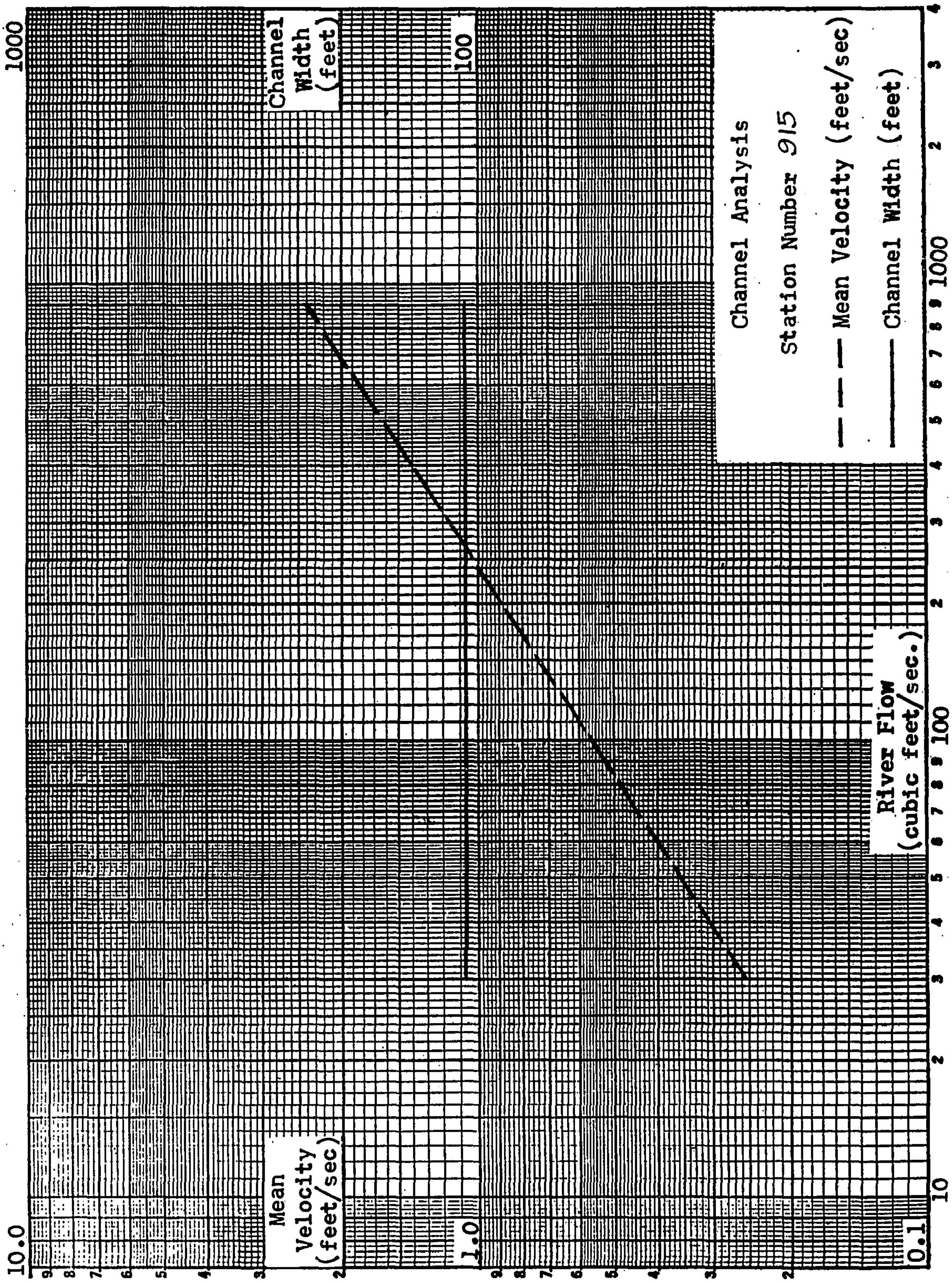


ILLUSTRATION 30

Typical Channel Analysis Graph

It should be realized that the selection of these measuring sites as being representative of the entire river may not be completely valid, but they do provide an indication of the size and character of the channel and flow which is better and more easily obtained than any other available information.

The U.S. Army Corps of Engineers publishes detailed navigational maps for some of the major rivers. These maps include bottom soundings and contours below the pool elevation. Where these maps are available cross-section profiles may be plotted from the data given.

In the Ohio River basin these maps are available for all of the Ohio River (scale: 1" = 600 ft.), 79 miles of the Allegheny River (scale: 1" = 400 ft.), 129 miles of the Monongahela River (scale: 1" = 400 ft.), 93 miles of the Muskingum River (scale: 1" = 416.7 ft.), 80 miles of the Little Kanawha River (scale: 1" = 200 ft.), 95.4 miles of the Kanawha River (scale: 1" = 200 ft.), 100 miles of the Big Sandy-Levisa Fork River (scale: 1" = 200 ft.), 206 miles of the Green River (scale: 1" = 1000 ft.), several miles of the Barren River (scale: 1" = 1000 ft.), 409 miles of the Wabash River (scale: 1" = 1000 ft. and/or 1" = 2000 ft.), and for 236 miles of the White River (scale: 1" = 900 ft.).(44)

The use of these maps have three disadvantages when compared to available U.S. Geological Survey data. First they usually require more effort and are not as accurate. Secondly, the maps are almost all 30 years old; and thirdly, a complete set of these maps for the basin would cost several hundred dollars.

In the tidal zone of the river, however, the Corps of Engineers navigation charts and those published by the U.S. Coast and Geodetic Survey are very valuable. This is for four reasons. The first is that they are about the only information generally available, the second is that except for tidal action, the water surface elevation is nearly constant; third, the flow characteristics are determined from other sources; and fourth, the shortness of the tidal zone makes such extensive work feasible.

Illustrations 31 and 32 show sections taken from a U.S. Army Corps of Engineers Ohio River navigation chart and a U.S. Coast and Geodetic Survey Delaware River chart.

Channel Slope

The channel slope can easily be approximated by computing the slope between data stations. It is probably easiest to use the gage datum elevation for the U.S. Geological Survey gaging stations although the stream line elevation can be estimated from navigation and topographical maps. Since the slope is only used in the calculation of the K_2 coefficient in water of less than five feet mean depth, it can be ignored for streams known to have a mean depth greater than five feet. In such a case the slope and elevation should be listed as zero.

Water Temperature

It was shown in Chapter IV that the saturation dissolved oxygen capacity, K_1, and K_2 are all functions of temperature. Generally, colder water temperatures make possible greater waste assimilation capacity.

Three studies were made regarding temperature in connection with

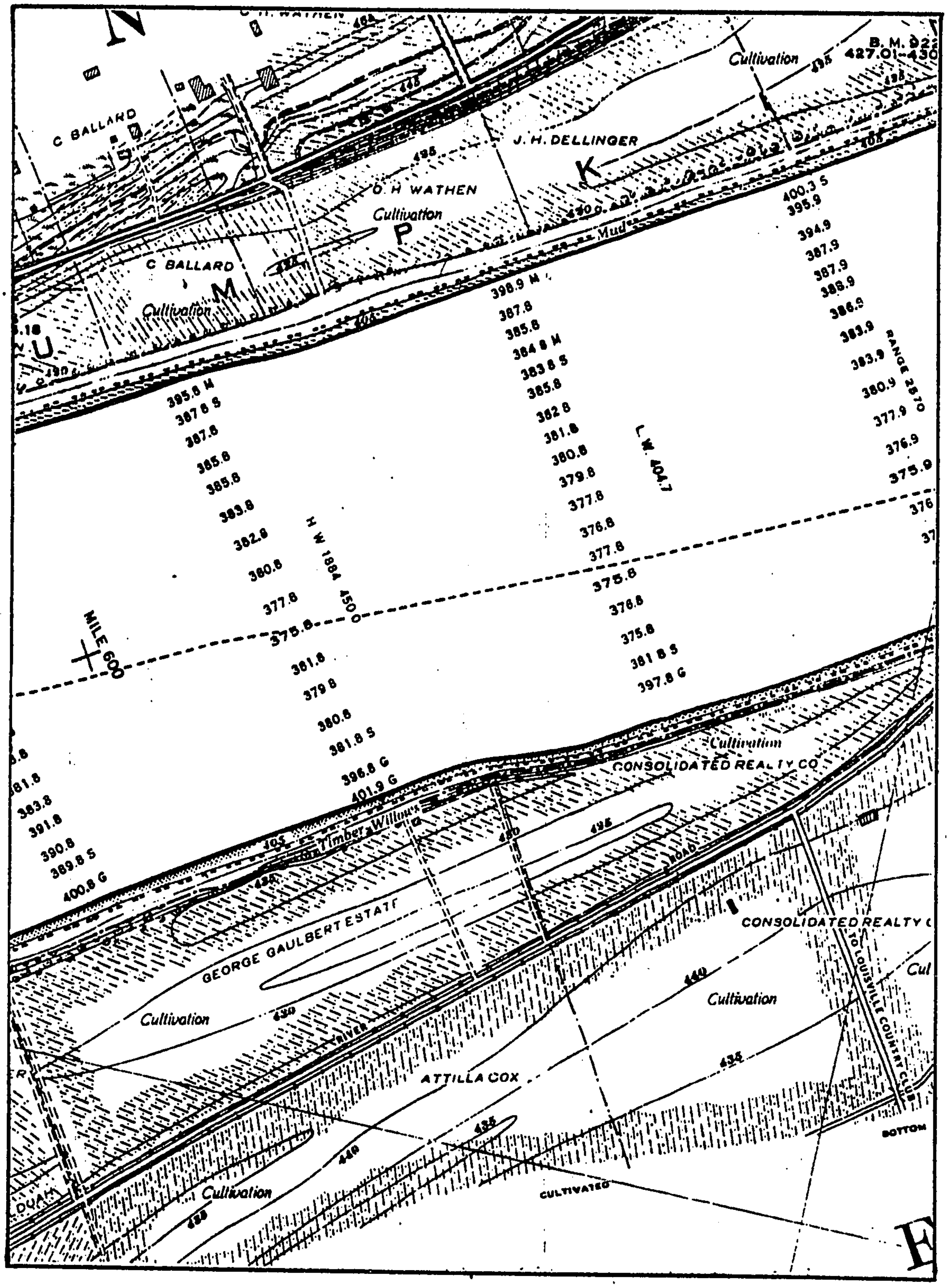


ILLUSTRATION 31

Section Of An Ohio River Chart

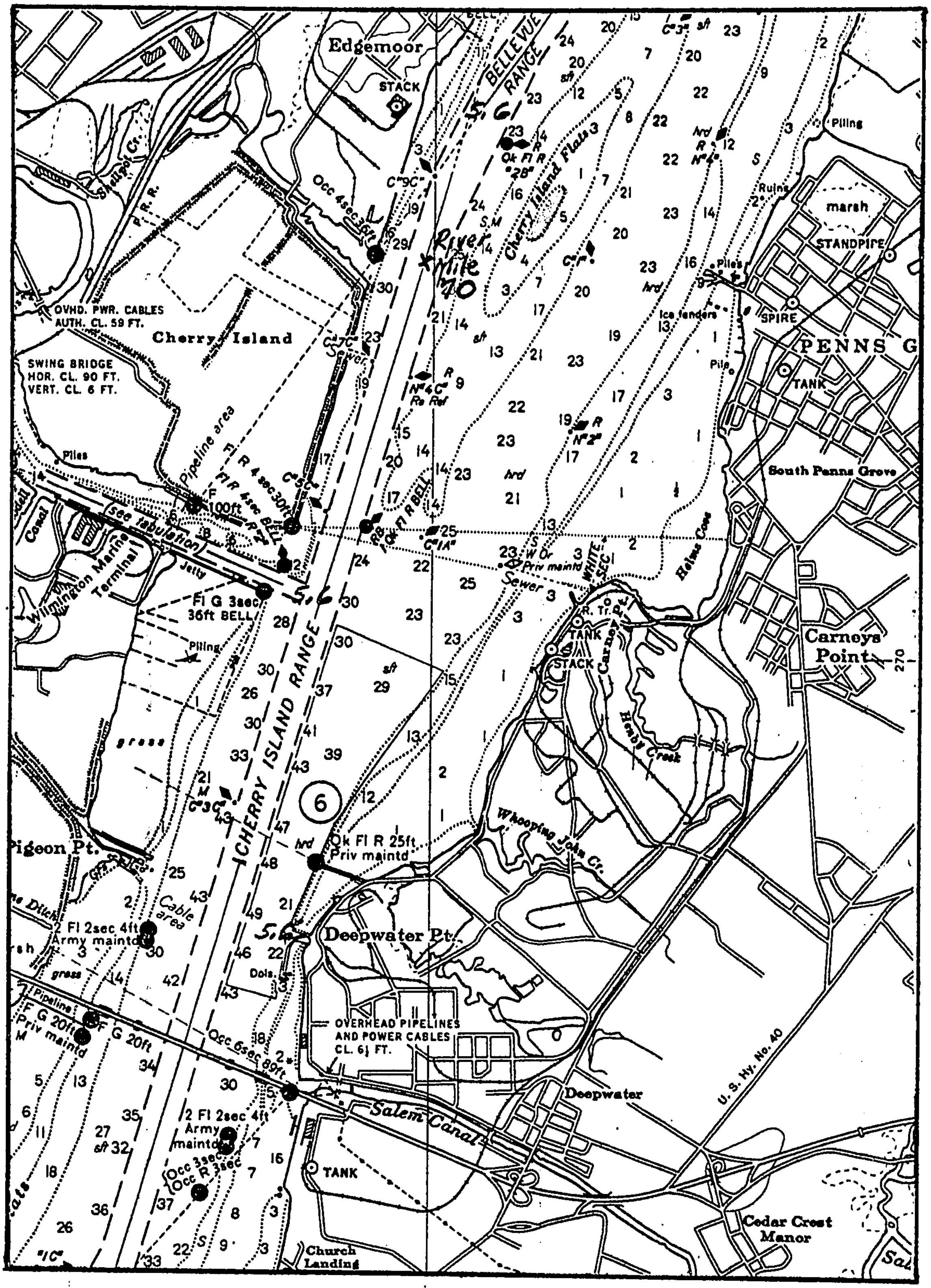


ILLUSTRATION 32

Section Of A Delaware River Chart

this dissertation. The first was an analysis of summer month water temperatures in Ohio, Indiana, and Pennsylvania; the second was an analysis using the computer to evaluate the effect of lower water temperature compared to increased water flow; and the third was the effect of increased water temperature on the waste assimilation capacity.

The first study was made possible by virtue of a recent practice followed by the U.S. Geological Survey in the states mentioned of noting the water temperature on the 9-207 form whenever a channel measurement was made.

The study consisted of determining the average temperature for each summer month for Pennsylvania and for Indiana and Ohio combined. Over 1000 water temperature reading were used in determining these average values. Table 6 is a summary of the results obtained.

TABLE 6

MEAN SUMMER WATER TEMPERATURES

	Indiana - Ohio		Pennsylvania	
	Temperature in Degrees		Temperature in Degrees	
	Farenheit	Centigrade	Farenheit	Centigrade
July	75.75	24.30	73.94	23.3
August	76.64	24.80	74.66	23.7
September	71.50	21.95	69.35	20.7
October	60.28	15.70	56.66	13.7
November	48.97	9.42	47.03	8.45

From this study it was noted that the water temperatures in Pennsylvania were one to two degrees centigrade cooler than Indiana or Ohio.

The most important information, however, was noting the difference of at least 6.25 degrees centigrade between the September and October temperatures. This information formed the basis for the second study.

The purpose of the second study was to determine what river flow increase would give the same increase in waste assimilation capacity as the 6.25 degree lower temperature mentioned above. The 1980 Ohio River Study which was used to test the Waste Assimilation Program provided an excellent means to evaluate this problem.

The procedure used is as follows: first a set of data cards were prepared with a temperature 6.25 degrees centigrade below the 25 degrees centigrade used in the 1980 Ohio River Study; second, another set of data cards were prepared in which the flow had been increased by an arbitrary amount, namely 17.2%; third, the total waste assimilation capacity was determined by the Waste Assimilation Program; and fourth, the results were adjusted accordingly.

It was found that the lower temperature produced an increase in the allowable BOD loading of 363,228 pounds per day. It was also found that the 17.2% increase in flow caused an increase of 317,827 pounds of BOD per day. Assuming that the increase caused by flow is approximately linear, the increase in flow required to equal the decrease in temperature is equal to $\frac{363,228}{317,827}(17.2\%) = 19.7\%$.

From this information it was decided to increase the low October river flows by 20% to compare them with the low September flows. The application of this procedure was discussed earlier in this chapter.

The third temperature study is part of the Wabash Study which will be discussed in Chapter VII.

Although the 9-207 forms did not contain sufficient years of record to evaluate the maximum summer water temperatures, the records did show a variance of two to three degrees centigrade above the average values. It is assumed that these temperatures were measured in the daytime. Fry, Churchill, and Elder in the report of a Tennessee Valley Authority investigation show a chart of temperature variation for the entire day for a several day period. This chart shows a net change from high to low of about two degrees centigrade.(54)

In view of the above it was arbitrarily decided to use a water temperature of 25 degrees centigrade for the Ohio River and tributaries above the mouth of the Scioto River, to use 26 degrees centigrade between the Scioto River mouth and Cincinnati including the Scioto River, and to use 27 degrees centigrade below Cincinnati. The only exception to this was in the vicinity of Youngstown, Ohio, on the Mahoning River where U.S. Geological Survey measurements showed that pollution in the form of hot water from industrial plants had raised the water temperature considerably. The higher values were used in this case. These assumptions are in agreement with the temperature values experienced by Streeter and Phelps in their study of the Ohio River conducted in 1914. (1)

In most cases sufficiently accurate water temperature information can be obtained from the U.S. Geological Survey for use in pollution analysis work. This data can be obtained from either the district engineer or the district chemist.

In the tidal zone regions much temperature data can be obtained from the U.S. Coast and Geodetic Survey publications which give surface water temperature and salinity.(55) For the Delaware River the temperature was listed for Philadelphia and for the river mouth at Breakwater Harbor. Since the two differed, the temperature difference was distributed in proportion to the salinity. In other words, if the salinity was equal to one half of the sea water salinity, it was assumed that one half fresh water and one half sea water had mixed; and therefore, their mean temperature would prevail.

Initial Dissolved Oxygen

Water which flows into a river seldom has the same dissolved oxygen content as the river itself. This inflow may be rainwater, fresh stream flow, groundwater flow, or even possibly a highly aerated sewage or industrial waste which has a higher dissolved oxygen content, or it may be the discharge from a storage reservoir or a sewage or industrial waste with a lower dissolved oxygen content. Information for this deficit on a basin wide basis has not been found to be available.

For the Ohio River studies it was assumed that inflow water was at saturation dissolved oxygen. For the Delaware River the same was assumed except that the flow added to the Delaware at Philadelphia was assumed to be at the River Quality Standard.

Tidal Zone Flow Currents

Tides and tidal currents were discussed extensively in Chapter III. It was pointed out there that in order to properly evaluate the reaeration coefficient, it was necessary to determine the mean maximum ebb and flood currents and the time periods for which the river is in ebb flow and flood flow. Mean maximum flow may be explained as being the mean flow acting across a river cross section at maximum ebb or flood flow.

It was further mentioned in Chapter III that this information could be obtained in two ways. The first estimate is obtained by means of cubature calculation from tide height data throughout the river length. The second estimate is obtained from main channel current predictions published by the U.S. Coast and Geodetic Survey. To these two estimates could be added the third alternative of an extensive field study, although such an expenditure is out of the question for a planning level report.

It was originally believed that calculation of cubature for the tidal zone would be impractical because of the extensive calculations and detailed tidal data required. However, after continued study it was found that good tidal data could be approximated from tidal prediction tables and that the calculations could be arranged for digital computer application. Thus the use of cubature calculations became a powerful tool for use in tidal zone analysis.

The tidal data which is needed for this method is the tidal height at each hour for each station considered for a time period exceeding an entire tidal cycle. Since the U.S. Coast and Geodetic Survey publishes the predicted time and elevation of the tides at its high and low points, the data for the times between the extreme values was determined for each station as follows. To begin with, a day was selected which had a tidal change approximately equal to the mean tide change. The time picked for the Delaware River was from midnight to 5:00 P.M. on June 26, 1962. The second step was to plot the tidal height for the extreme values against time on rectangular coordinant paper. Then a line approximating the cosine curve was plotted between the respective high and low values. This was done by a short cut method recommended by the U.S. Coast and Geodetic Survey which consists of drawing a line between the two extremes and dividing it into fouths. The center point is considered a point on the curve and likewise are points located one-tenth of the total height difference above and below the other respective quarter points. From a smooth curve drawn through these points the tidal height at each hour, or fractions thereof, can be determined. The values were obtained for stations located every five miles along the Delaware River between Trenton, New Jersey, and Miah Maul Shoal. Illustration 33 shows the graph prepared as outlined above for a Delaware River station.

Once these tidal elevations have been determined the calculation is carried out as illustrated in the sample calculations in Chapter III. The computer program which is used is designed to follow this example procedure step by step and to punch out the results in the same format. The flow chart and program includes an initial card to define the head of tide point and three cards for each station. Table 7 shows the input items and their format. The widths and cross sectional area were determined from profiles plotted from U.S. Coast and Geodetic Survey navigation charts. The river flow used is the ten year low flow for the Delaware River as determined by the methods outlined earlier in this chapter.

The output from the computer will give the velocity at the different times. To determine the maximum ebb and flood currents and the

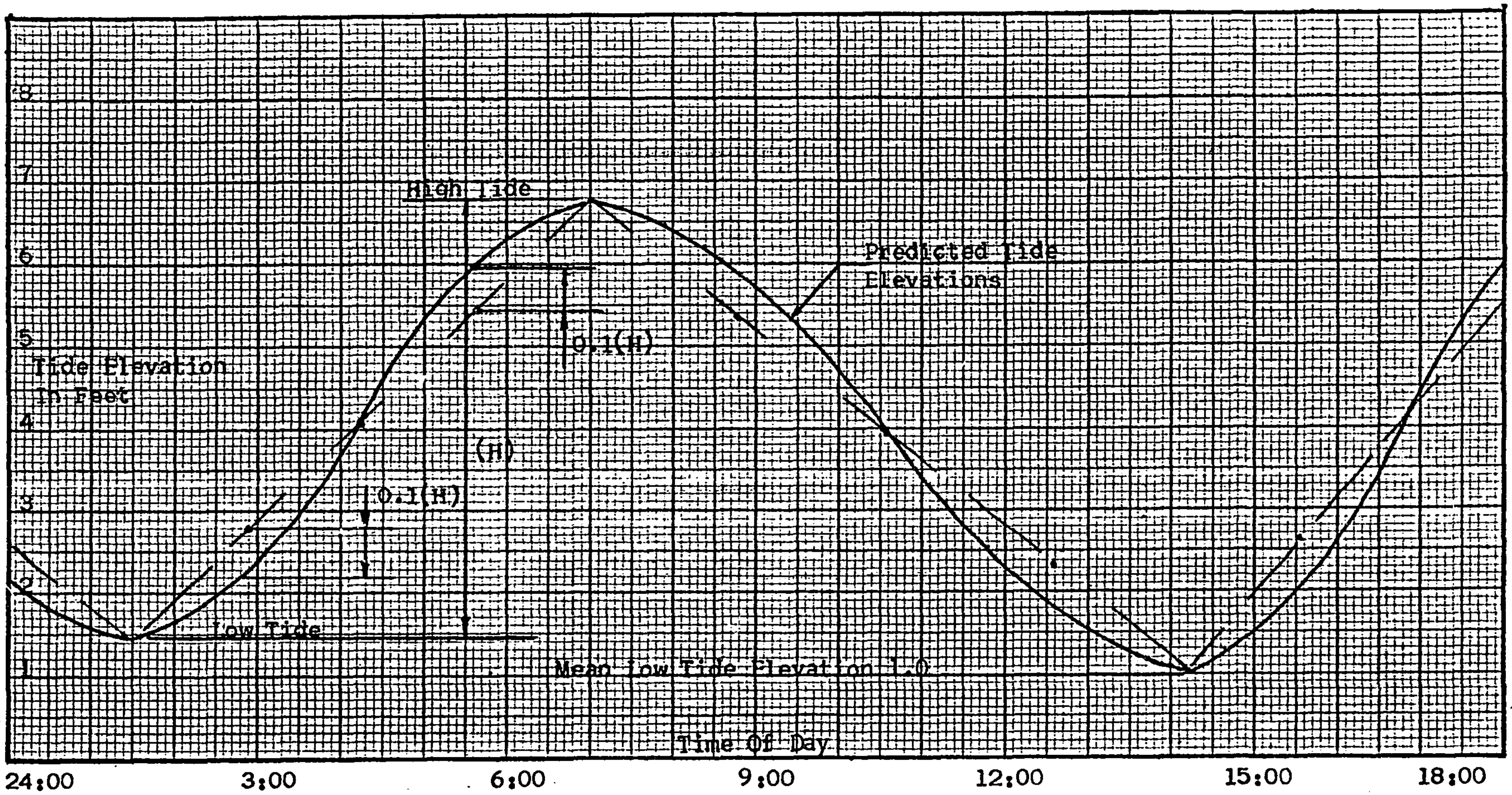


ILLUSTRATION 33

Predicted Tide Elevations

At Edgemoor Delaware For June 26, 1962

TABLE 7

DATA FORMAT - CUBATURE PROGRAM

Space	Format	Information	Example
INITIAL CARD			
1-10	E10.3	River Mile at Head of Tide	+1.305E+02
FIRST CARD			
1-10	E10.3	River Mile	+1.300E+02
11-20	E10.3	Minimum Tide Elevation	+1.000E+00
21-30	E10.3	Maximum Tide Elevation	+7.900E+00
31-40	E10.3	Width at Low Tide	+7.000E+02
41-50	E10.3	Width at High Tide	+8.000E+02
51-60	E10.3	River Flow	+1.828E+03
61-70	E10.3	Cross Section Area at Low Tide	+4.470E+03
SECOND CARD			
1-6	F6.1	Tide Elevation at 12:00	+07.00
7-12	F6.1	Tide Elevation at 1:00	+05.80
13-18	F6.1	Tide Elevation at 2:00	+04.40
etc.	etc.	etc. through 8:00	
THIRD CARD			
1-6	F6.1	Tide Elevation at 9:00	+06.50
7-12	F6.1	Tide Elevation at 10:00	+07.60
13-18	F6.1	Tide Elevation at 11:00	+07.80
etc.	etc.	etc. through 17:00	

time of ebb and flood it is necessary to plot these answers against time on rectangular coordinant paper and to plot a smooth curve through these points. The maximum currents are measured vertically from the line of zero current, and the times are the distances between where the curve cuts this zero line. Illustration 34 shows the curve plotted from output data for the stations at Philadelphia, Pa., and Edgemore, Delaware.

The U.S. Army Corps of Engineers has calculated the cubature for the Delaware River for mean tide conditions and a flow increasing from 12,000 cubic feet per second at the head of tide point. The results of their calculations for Philadelphia are also shown in Illustration 34.

The Corps of Engineers results were used in the study of the waste assimilation capacity of the Delaware River primarily because at the time the data was needed, the Cubature Program had not been completed.

The maximum flows obtained from both studies are shown for several stations in Table 8. The results agree quite closely down to Fort Miffin at mile 90, although as can be seen in the graph for Philadelphia, the area under the ebb curve differs. Below Fort Miffin the maximum value computed by the cubature program greatly exceeds the maximum flow given by the Corps of Engineers calculations; however, as shown in the graph for Edgemore, Delaware, the areas under curves are almost the same.

It is believed in view of the close comparison in area under the curves and because the computed values more nearly approach the cosine shape than those of the Corps of Engineers that it would be more accurate to use the values computed by the Cubature Program for waste

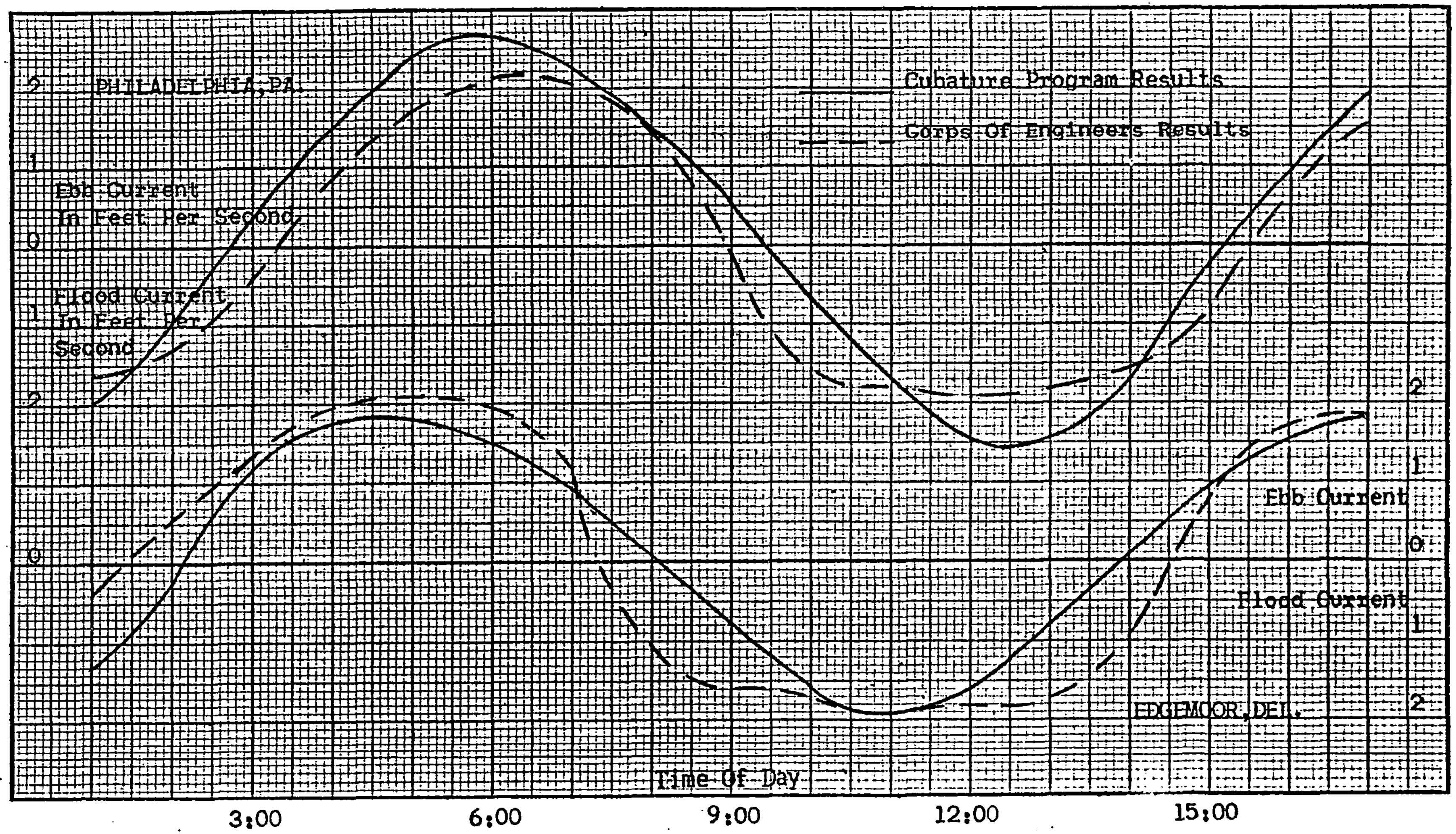


ILLUSTRATION 34

Cubature Study Results For The Delaware River

TABLE 8

PART A
COMPARISON OF EBB CURRENTS

	River Mile	USCGE Current Tables	U.S. Army Channel Currents	U.S. Army Cubature Calculations	Cubature Program Results	Ratio Col 4 / Col 2	Ratio Col 5 / Col 2
Trenton	130	. .	. .	.80	.47	. .	. .
Feldsboro	125	2.36	. .	1.11	1.33	.47	.57
Burlington	115	2.70	. .	1.40	1.95	.52	.72
Torresdale	108	2.70	. .	1.86	2.05	.69	.76
Philadelphia	100	2.70	2.70	1.96	1.93	.72	.72
Fort Miffin	90	3.72	2.50	1.95	1.92	.52	.52
Eddystone	82	3.72	3.50	1.86	2.90	.50	.78
Marcus Hook	77	2.70	2.50	2.00	2.77	.74	1.00
Edgemore	71	2.54	2.45	1.93	2.50	.76	1.00
New Castle	64	4.05	2.60	2.05	2.90	.51	.72
Reedy Point	56	3.71	2.80	2.03	2.81	.54	.76
Artificial Is.	50	3.21	3.00	2.00	3.01	.62	.93
Woodland Beach	40	3.54	. .	1.82	2.74	.51	.77
Ship John	35	. .	. .	1.90	2.75	. .	. .
Elbow of Cross Ledge	25	3.72	. .	. .	. .	. .	. .
14 Foot Bank Light	15	2.54	. .	. .	. .	. .	. .
Entrance	0	3.21	. .	. .	. .	. .	. .
Mean Value						.60	.77

TABLE 8

PART B
COMPARISON OF FLOOD CURRENTS

	River Mile	USCGS Current Tables	U.S. Army Channel Currents	U.S. Army Cubature Calculations	Cubature Program Results	Ratio Col 4 / Col 2	Ratio Col 5 / Col 2
Trenton	130	. .	. .	. .	. .	. .	. .
Feldsboro	125	. .	. .	1.60	.90	. .	. .
Burlington	115	2.20	. .	1.46	1.48	.66	.67
Torresdale	108	. .	. .	1.97	1.77	. .	. .
Philadelphia	100	2.70	3.00	2.08	1.80	.77	.68
Fort Miffin	90	3.21	2.40	1.95	1.80	.61	.55
Eddystone	82	2.87	2.70	1.89	2.63	.66	.92
Marcus Hook	77	2.87	2.70	2.08	2.70	.72	.94
Edgemore	71	2.70	2.80	2.05	2.56	.76	.95
New Castle	64	3.21	2.80	1.98	3.00	.71	.94
Reedy Point	56	3.38	3.20	1.98	3.01	.59	.89
Artificial Is.	50	2.54	2.50	2.05	3.02	.81	. .
Woodland Beach	40	3.38	. .	1.80	3.00	.53	.89
Ship John	35	. .	. .	1.80	3.29	. .	. .
Elbow of Cross Ledge	25	2.20	. .	. .	. .	. .	. .
14 Foot Bank Light	15	2.20	. .	. .	. .	. .	. .
Entrance	0	3.04	. .	. .	. .	. .	. .
Mean Value						.682	.83

assimilation studies.

The predicted current velocities which are obtained from U.S. Coast and Geodetic Survey tables and charts are usually for the deep channels of the river.(10, 12) Therefore, in order to be able to determine the mean current for the cross section, a ratio of channel velocity to mean velocity must be determined. To do this the channel velocities determined from the current tables and from Corps of Engineers' studies are compared in Table 8 with the mean values obtained by the cubature calculations for several stations. It can be noted that there is often considerable difference between the channel velocities shown by the two agencies.

By comparing the averages of the Coast and Geodetic Survey channel values with the U.S. Corps of Engineers' mean values, it was found that the mean ebb flow equaled 60% of the maximum ebb current and that the mean flood flow equaled 68% of the maximum flood flow. The same ratios for the cubature computed from predicted tides are 77% and 83%. In view of the deviation of the Corps of Engineers' currents from the cosine form, it is recommended that the higher ratio values of 77% and 83% be used in waste assimilation capacity studies.

It is believed that either of the two methods for obtaining the currents and flow time are satisfactory and that whichever one is easiest should be used in a given application.

Salinity

The distribution of salinity in the tidal zone of a river was discussed extensively in Chapter III. The only accurate way to determine the salinity and its structure is by measuring it.

Since the salinity density currents materially affect shoaling of channels, numerous salinity measurements have been made by agencies interested in navigation. Much salinity data can be obtained from various reports and from the district offices of the U.S. Corps of Engineers.(7, 8, 15)

Salinity at major points and river mouths can also often be obtained from U.S. Coast and Geodetic Survey publications.(55) If the upstream point of salinity intrusion is known in a well mixed estuary, the Arons and Stommel equation may be applied.(21)

The salinity data which was used for the Delaware River analysis was that published by the U.S. Corps of Engineers in connection with their model study.(7, 8)

CHAPTER VII

APPLICATIONS

1960 Ohio River Basin Study

After the Waste Assimilation Program was completed and tested, it was decided to use it to analyze the waste assimilation capacity of a complete river basin. The Ohio River basin was chosen for several reasons. First, it is a major river basin which is unaffected by tides. Second, there has been much interest paid to water quality in this basin; and third, most of the major rivers still have stream flow in dry years. This analysis is called the 1960 Ohio River Basin Study because the channel data and flow properties are those recorded in the decade ending with 1960.

The greatest effort in making the study was the collection and reduction of data. This was done in accordance with the procedures outlined in the previous chapter. Flow data was analyzed for over 150 U.S. Geological Survey gaging stations, and finally 110 stations were selected to be used. U.S. Geological Survey 9-207 forms were then obtained for the last five years for each station. River flow analysis sheets and graphs of velocity and width were then prepared for each station from the "Water Supply Papers" and 9-207 forms. Additional river flow analysis sheets were also prepared for the mouths of the rivers. After the data was collected and analyzed, it was punched on I.B.M. cards for use in the computer program.

Needless to say, considerable time and effort were needed to assemble and reduce this data. It is estimated that after procedures were standardized, data for a basin this size can be collected and processed in about three man months assuming a 40 hour work week.

Several minor details of the study bear mentioning. Only rivers which had a ten year low monthly flow of over ten cubic feet per second were used. The head of the river was chosen as either the point where the flow was only one cubic foot per second or at the base of an upstream dam which regulated flow. If there was still a large flow at the upstream station, the head of the river was often considered to be at a distance upstream roughly equal to the square root of the drainage area. Two procedures were used in computing the flow at the mouth of the river: where the flow increase in the larger stream was large enough to allow it, the flow of the last station upstream was increased in proportion to the larger drainage area at the mouth; where the flow increase of the larger stream was not so large, the unmodified flow of the upstream station was used.

The Tennessee and Cumberland Rivers were not considered in this study. Their effect on the overall program was eliminated by considering them together as a single river one mile long with a River Quality Standard equal to the dissolved oxygen saturation value.

Illustration 35 is a map of the Ohio River basin.

It was decided to use a segment length of ten miles for this study, and therefore, the above data was processed through the Data Generating

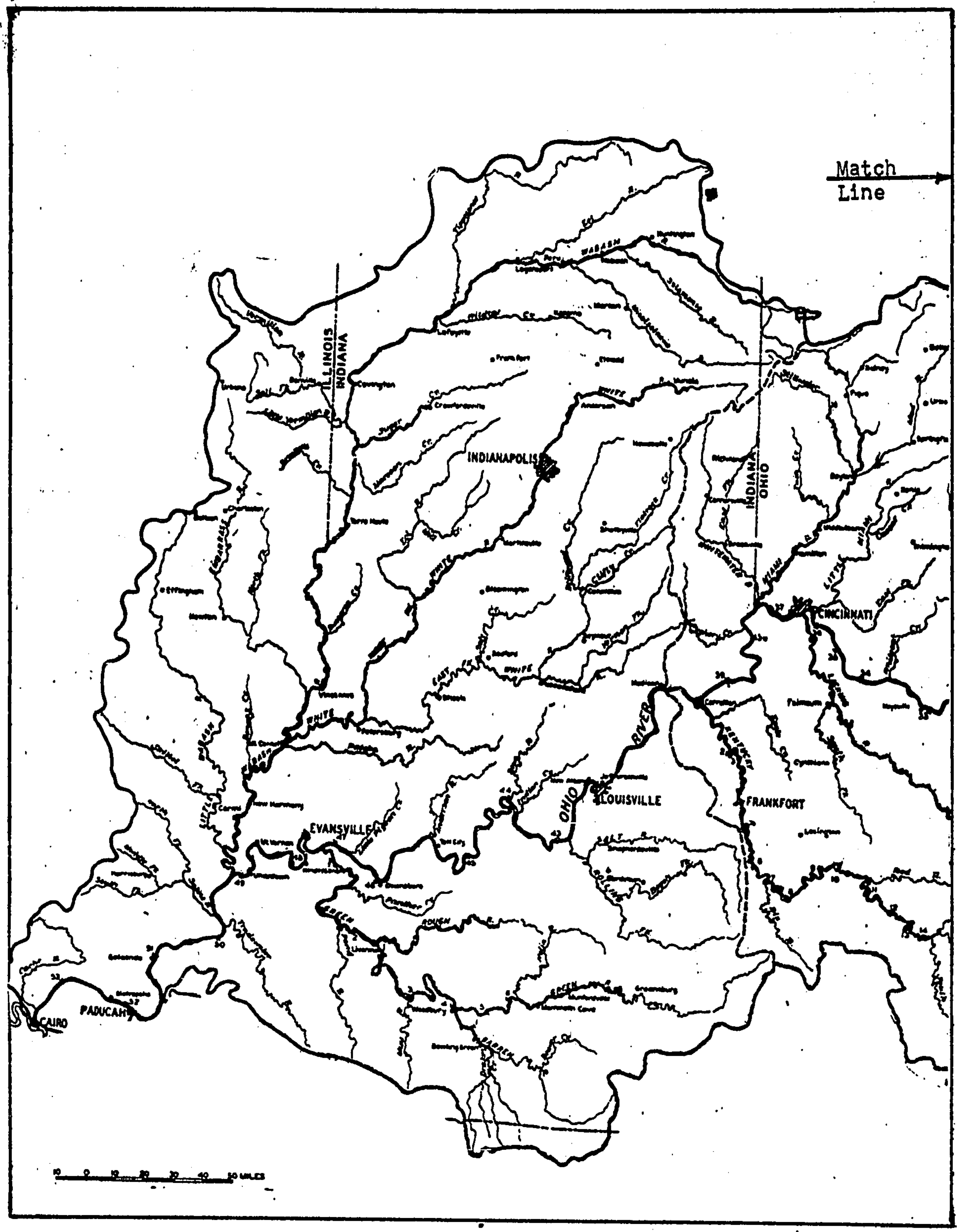


ILLUSTRATION 35 PART A

The Ohio River Basin

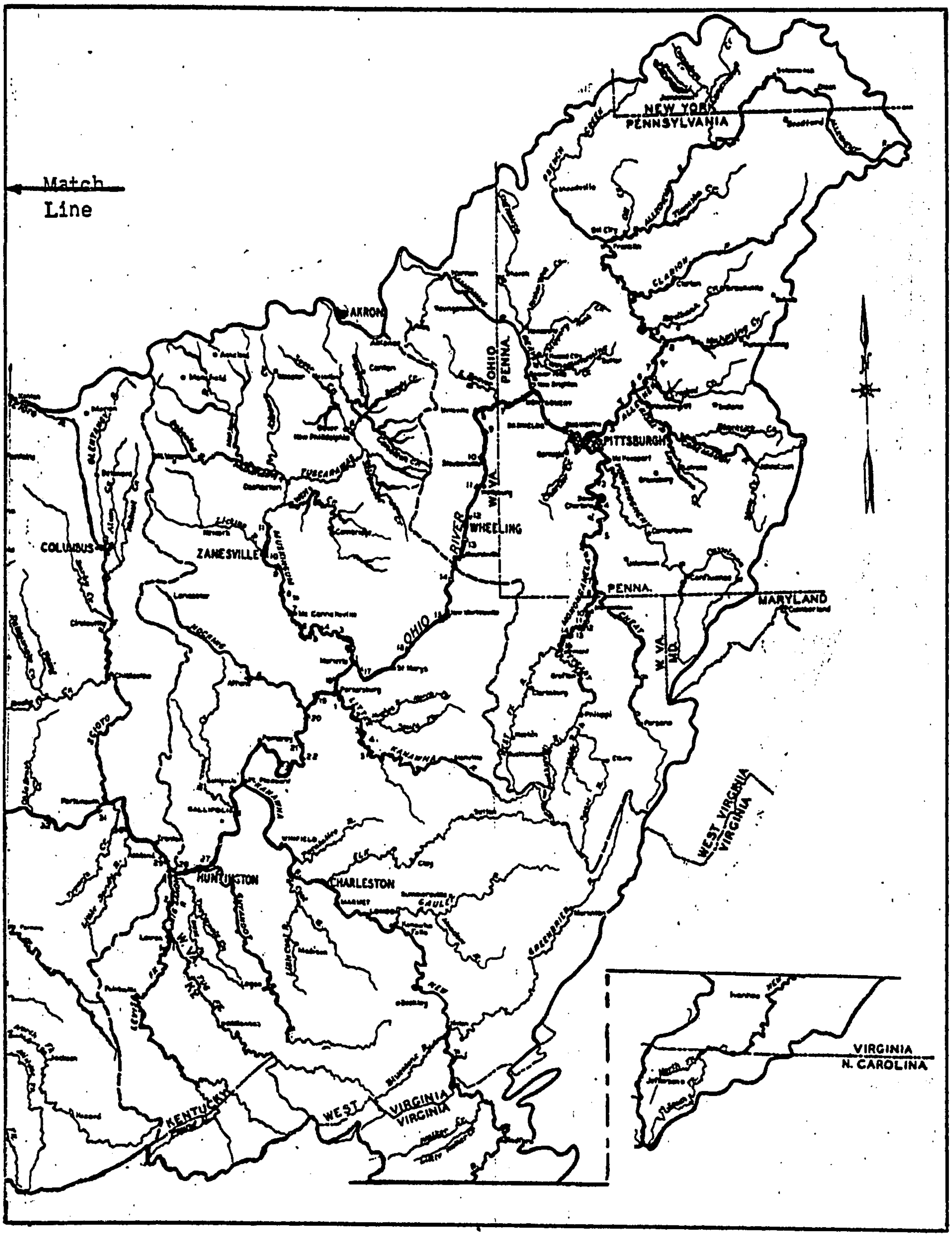


ILLUSTRATION 35 PART B

The Ohio River Basin

Program to obtain data cards to each ten mile segment. These new cards comprised the input to the Waste Assimilation Program.

The processing of the data for the entire Ohio River basin in the Waste Assimilation Program required roughly an hour and 15 minutes.

A summary of the waste assimilation capacities and the corresponding population equivalents for the individual rivers in the basin is shown in Table 9. The allowable BOD values are in units of pounds per day. These numbers may be multiplied by four to give the load in population equivalents.(23)

If the computer output were examined, it would be noted that there is an occasional negative value shown for the allowable BOD for a small segment. This occurs because the residual BOD from upstream is greater than the available oxygen in the segment. This condition is brought about when an upstream segment has an unusually large amount of available BOD in that segment. A large drop in the K_2 values between two segments will also cause a negative value in the lower segment. If it is desired to determine the actual BOD value for a section which includes a negative value, the negative value should be averaged algebraically with the few segments above the negative value.

One of the major uses of a study of this type would be to determine sewage and industrial waste treatment criteria. For example, if the population were spread over the basin in proportion to the waste assimilation capacities shown, then the river could purify the wastes of four times 6,376,000 or 25,504,000 population equivalents. If the pollution equivalent of population and industry were double this value, then a minimum treatment of 50% would be required. This will be

TABLE 9

SUMMARY OF ALLOWABLE BOD VALUES FOR THE OHIO RIVER BASIN

River Number	River Name	Pounds of BOD per Day	Population Equivalents
1	Ohio	3,959,266	15,837,064
2	Clarion	48,355	193,420
3	Kiskimentas - Conemaugh	53,243	212,972
4	Monongahela	239,398	957,592
5	Cheat	32,846	131,384
6	Mahoning - Beaver	52,828	211,312
7	Muskingum	78,314	313,256
8	Little Kanawah	5,793	23,172
9	Hocking	19,238	76,952
10	New - Kanawha	508,293	2,033,172
11	Guyandot	20,116	80,464
12	Levisa - Big Sandy	33,392	133,568
13	Scioto	87,570	350,280
14	Miami	80,192	320,768
15	Kentucky	106,419	425,676
16	Green	114,653	458,612
17	Nolin	11,759	47,036
18	Barren	11,160	44,640
19	Wabash	737,836	2,951,344
20	Mississinewa	1,703	6,812
21	White	173,612	694,448
22	Tennessee - Cumberland	Assumed 0	
Rounded Total for Entire Basin		6,376,000	25,504,000

discussed further for the Ohio River basin in Chapter IX.

Wabash River Study

A second study was made on the Wabash River to show the possible application of the Waste Assimilation Program. The purpose of this study was to evaluate the effect of increased river flow on the waste assimilation capacity and to evaluate the effect of increased water temperature on the waste assimilation capacity.

The first of these studies is important because much consideration has been given lately to the desire and feasibility of low flow augmentation. This study was carried out in two major steps. First the low ten year flow was increased by 50% and 100%, and the channel properties for all three flows were determined from the channel analysis graphs used in the 1960 Ohio River Basin Study. Secondly data cards were prepared for all three conditions and the three sets processed in the Waste Assimilation Program.

The results of this study are listed in Table 10, Part A. Only the total allowable BOD loads from above river mile 184 are considered. From the results it can be noted that the increase in waste assimilation capacity is equal to about 55% of the increase in flow. This relationship appears to be almost constant in the flow range shown.

The temperature study is important because of the increased use of river water as cooling water. Since this use raises the temperature level in the stream, it can materially affect waste assimilation capacity. This is often called thermal pollution. The study of temperature consisted of preparing three data card sets with identical flow characteristics but different temperatures of 27, 29, and 31 degrees centigrade respectively. These three decks of cards were then processed through the Waste Assimilation Program. The results of the temperature study are listed in Table 10, Part B. Both the Wabash River and its main tributary, the White River, were considered in this study.

TABLE 10

WABASH RIVER STUDY

Part A
Flow Analysis

	River Flow As % of 10 Year Low		
	100%	150%	200%
Total Allowable BOD (10^{-3})	287.8	365.8	447.4
Mile 466 to 184 Increase over 100% value		27%	55.5%

Part B
Temperature Analysis

		Temperature in Degrees Centigrade		
		27	29	31
Total Allowable BOD (10^{-3})	Wabash	737.8	709.0	679.3
	White	173.6	165.5	157.3
	Total	911.4	874.5	836.6
Percent of 27 Degree Value		100.0%	95.95%	91.78%

In the temperature analysis it can be seen that an increase of two degrees centigrade caused a 4.05% loss in waste assimilation capacity, and a four degree centigrade increase caused an 8.22% loss. For small temperature changes this can be considered as a 2% loss in waste assimilation capacity per degree increase in temperature.

Delaware River Study

The Delaware River Study was carried out to show an application of the Tidal Zone Waste Assimilation Program and also to evaluate the necessity of considering tidal action in determining the waste assimilation capacity of the tidal zone.

The first part of the analysis was the determination of the waste assimilation capacity of the Delaware River from Barrysville, New York, to the mouth of the river assuming that there was no tidal effect. The data upstream from the head of tide at Trenton, New Jersey, was collected and analyzed in the same manner as for the Ohio River. Downstream from this point, mean depth and cross sectional area were determined from cross sectional profiles plotted from U.S. Coast and Geodetic Survey navigation charts.

The second stage of the study was the determination of the waste assimilation capacity assuming there are tidal effects. The data for this application was collected and analyzed as discussed in Chapter VI.

A summary comparing the results of the analysis considering tidal action and the analysis which does not consider it is shown in Table 11.

In the zone between Philadelphia and Ship John Shoal the allowable BOD load assuming no tides is 560,000 pounds of BOD per day, whereby if tidal action is assumed, the total allowable load is 3,438,000 pounds of BOD per day. This is an increase of over 500%. From this one can readily see the necessity of considering tidal action in computing the waste assimilation capacity of tidal rivers.

TABLE 11

DELAWARE RIVER STUDY - WASTE ASSIMILATION CAPACITY

	Assuming No Tides (lbs./day (10^{-3})	Assuming Tides (lbs./day (10^{-3})
Above Trenton, N.J.	221	221
Trenton to Philadelphia	68	193
Philadelphia to Ship John Shoal	560	3438
Ship John Shoal to Mouth	496	5486
Total	1345	9338

A map of the tidal portion of the Delaware River is shown in Illustration 36.

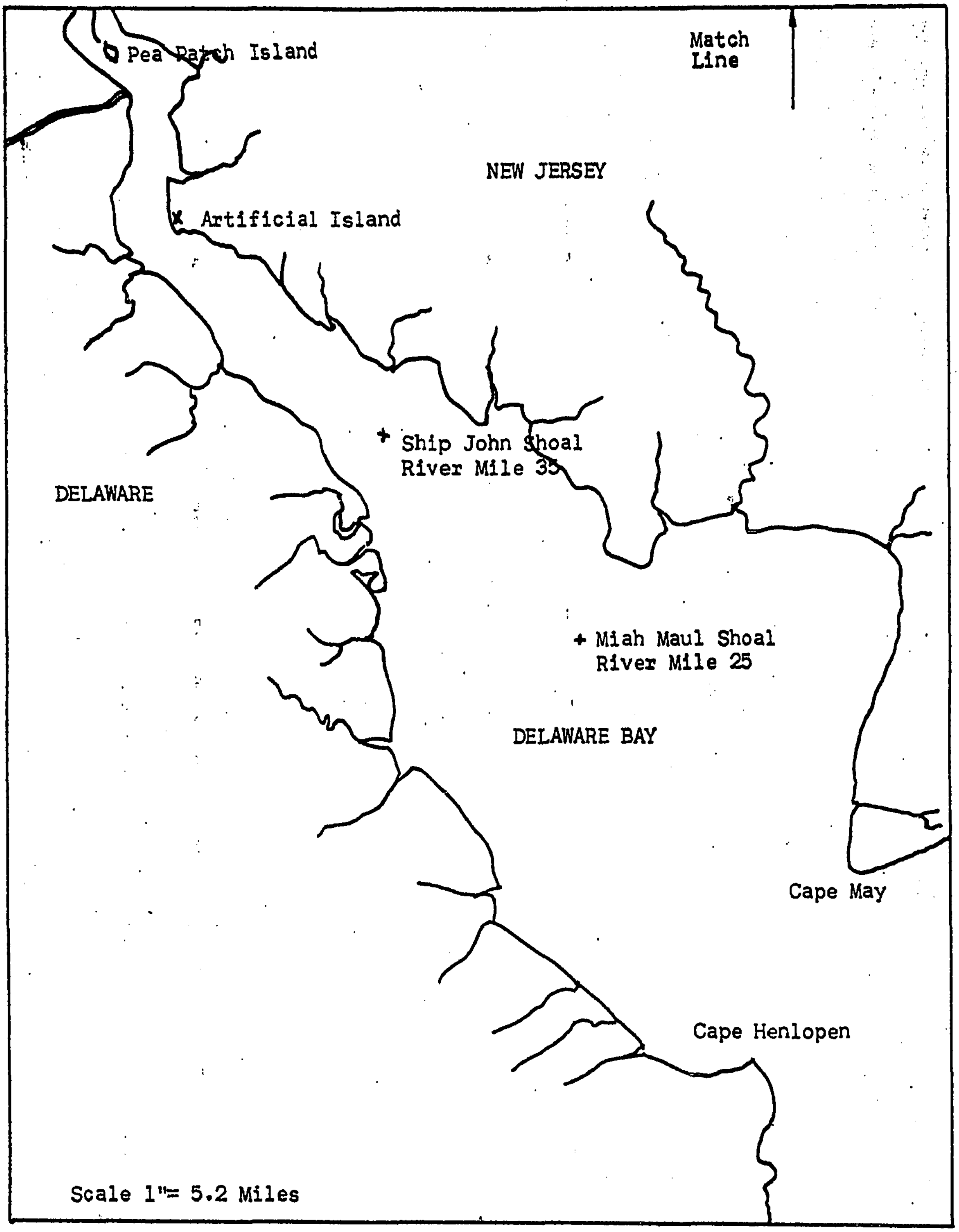


ILLUSTRATION 36 PART A

The Delaware River

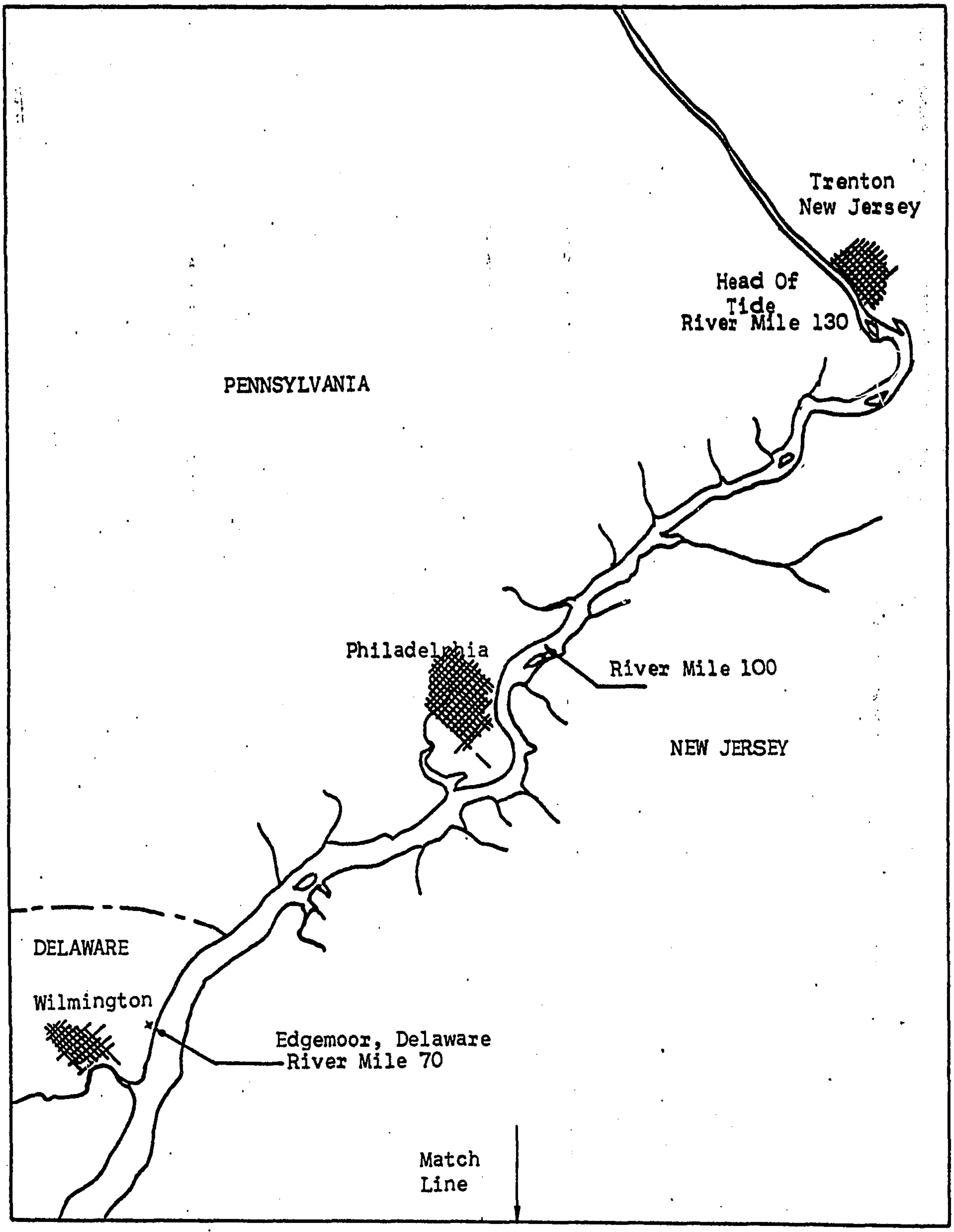


ILLUSTRATION 36 PART B

The Delaware River

CHAPTER VIII

POINT LOAD COMPARISON

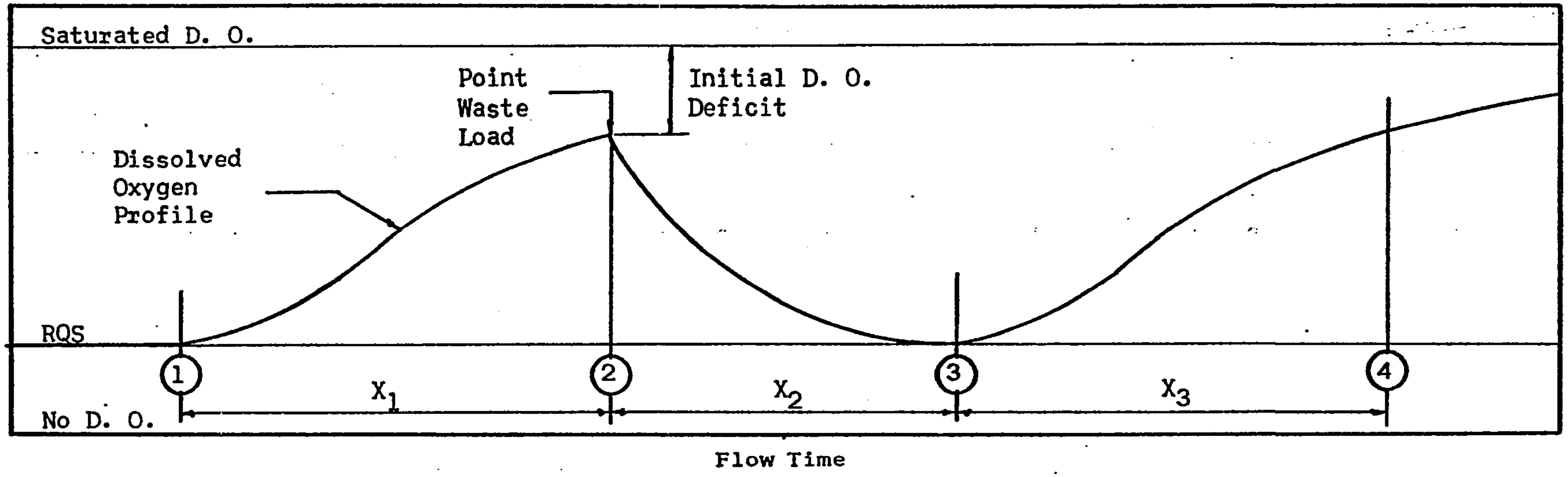
The waste assimilation capacity computed for a river by the methods used in this dissertation is the theoretical maximum for the river and is based on the assumption that each and every segment is utilized to the limit of its assimilation capacity. The value thus obtained for each segment and for the river as a whole is actually equal to the daily BOD loading which would have to be continuously applied along the river to keep the dissolved oxygen content depressed to the River Quality Standard.

Since in practice it is not probable that actual BOD loadings would be spread along the river in the manner necessary to attain the maximum assimilation efficiency of the river, it is necessary to evaluate the effect of non-continuous loading. This evaluation has been called the Point Load Comparison, because in general it may be considered as the comparison of the point loadings of cities and the continuous assimilation capacity of the river.

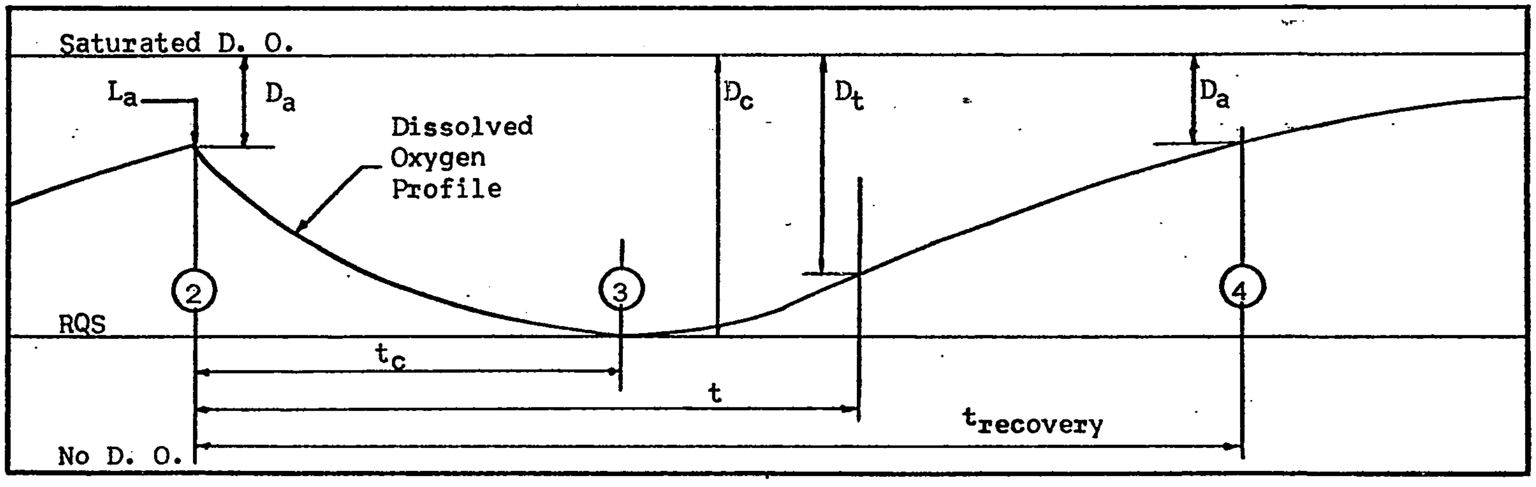
To begin, a single infinitely small segment of a river is considered. If the flow entering this segment has a dissolved oxygen content at the River Quality Standard, then the largest load which can be applied to the segment without violating the RQS downstream is the waste assimilation capacity for the segment. Thus the theoretical maximum point load would be equal to the waste assimilation capacity. Therefore, the only way in which a point load larger than the individual segment waste assimilation capacity can be permitted is for the flow coming into the segment to have a dissolved oxygen content greater than the River Quality Standard. In other words, if a city is going to place a point load on a river for example, by means of a single outfall, then the water flowing downstream to the point of discharge must have a quality substantially above the minimum allowable stream condition.

Illustration 37, Part A shows the longitudinal profile of the dissolved oxygen content of a river upstream and downstream from a point load application. Upstream from the point labeled 1 on the diagram the river is loaded with BOD to the extent of its assimilation capacity, and therefore, the oxygen level remains at the RQS. It should be remembered that under these conditions dD/dt, the change of deficit with respect to time, is equal to zero. At Point 1 it is assumed that BOD loading ceases, and therefore, as the available oxygen exceeds the residual BOD from upstream, dD/dt becomes negative and the oxygen content of the water begins increasing. By the time Point 2 is reached the oxygen level has built up and the deficit correspondingly decreased to the level defined on the chart as the "Initial Deficit." At this Point 2 a BOD load is assumed applied to the river which is of such magnitude that it cannot be assimilated by the reaeration capacity of the zone immediately downstream from its application, and thus dD/dt becomes positive and the dissolved oxygen content begins dropping.

The maximum allowable point load that can be applied at Point 2



PART A



PART B

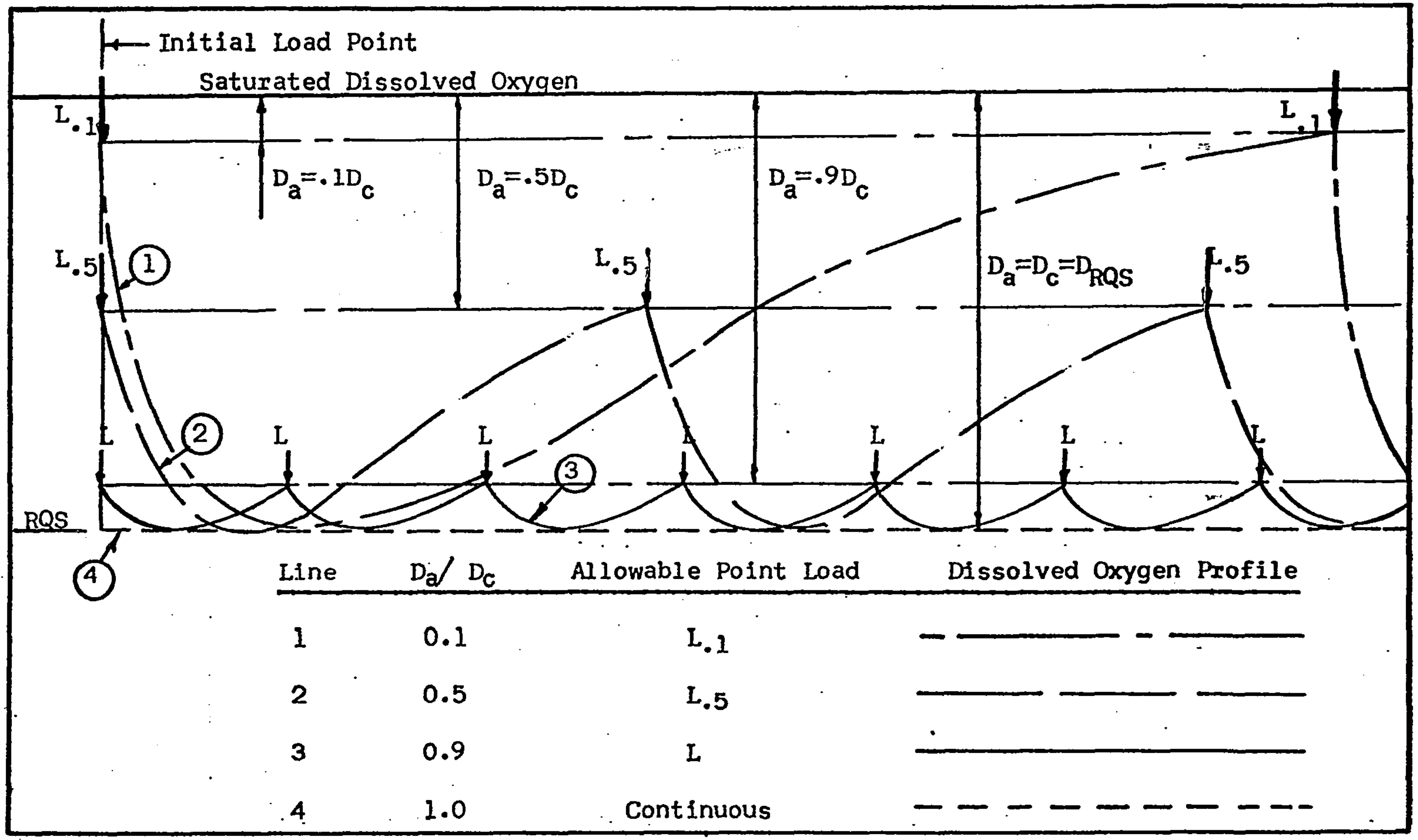
ILLUSTRATION 37

Nomenclature For The Dissolved Oxygen Profile is the one which causes the dissolved oxygen content downstream to just equal, but not go below, the RQS. The point where the dissolved oxygen content line touches the RQS line is labeled Point 3. At this point dD/dt again equals zero, and assuming the flow is the same, the river is again at exactly the same condition that it was at Point 1. If no BOD is applied after Point 3, the curve between Point 3 and Point 4 will be the same as the curve between Point 1 and Point 2; and distance X_1 will equal distance X_3. The fact that these two curves are equal is important because equations exist for the curve between Points 2 and 4.

Illustration 37, Part B is the portion of Illustration 37, Part A between Points 2 and 4 which has been relabeled with the notations used by Streeter and Phelps in their oxygen profile equations.(1)

The answer which is desired in the point load comparison study is the ratio of the maximum allowable point load to the total waste assimilation capacity for the distance between Points 1 and 3. This is not a constant but instead is a function of several variables. Among them are the initial oxygen deficit and the reaeration coefficient.

Another way of discussing the relationship of the continuous and point loads is demonstrated by means of Illustrations 38 and 39. Illustration 38 shows four oxygen sag curves for a particular stretch of river. Line A represents the curve for a condition where the initial deficit, D_a, is only a small fraction, .1 D_c, of the total deficit allowed when a point load is added. Line B represents the case where $D_a = \frac{1}{2} D_{RQS} = \frac{1}{2} D_c$; line C represents the case where $D_a = .9\ D_c$; and line D is the infinite case or continual loading case in which D_a is only an infinite amount smaller than D_c. In all cases it is assumed



Flow Time

ILLUSTRATION 38

Dissolved Oxygen Profiles For Various Loading Conditions

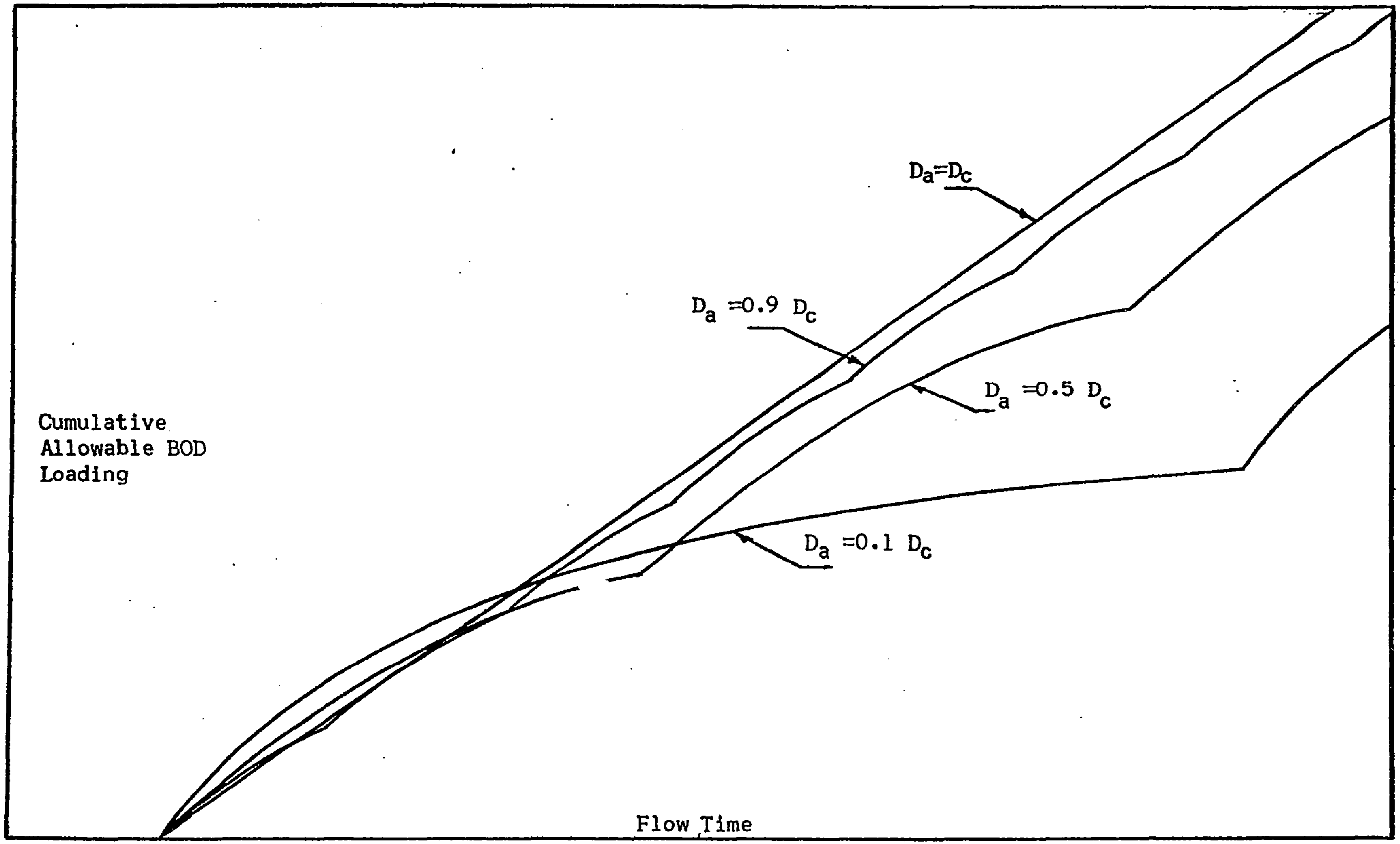


ILLUSTRATION 39

Cumulative Allowable BOD For Various Loading Conditions that another load equal to the original allowable load is applied when the river recovers to its original dissolved oxygen condition.

Illustration 39 shows the cumulative totals of the allowable BOD which is assimilated downstream from the start of these loadings. It can be noted from these lines that where the initial deficit is smaller, a larger single load can be applied but that the total load for the stretch of river is greater with more smaller loads and reaches a maximum as D_a approaches D_c.

The object of the Point Load Comparison Study may be restated as the evaluation of the area over the curve of a non-continuous loading as a fraction of the limiting continuous loading. The equations necessary for the evaluation of the Point Load Comparison are developed below.

As shown in Chapter II the basic equation for the dissolved oxygen profile is:

$$\frac{dD}{dt} = K_1 L - K_2 D \; . \tag{8.1}$$

The general solution of this equation to provide the dissolved oxygen deficit at any particular distance downstream from a pollutional load is given by Streeter and Phelps as:

$$D(t) = \frac{K_1 \, L_a}{K_2 - K_1} \left(e^{-K_1 t} - e^{-K_2 t} \right) + D_a \left(e^{-K_2 t} \right) \tag{8.2}$$

where $D(t)$ = dissolved oxygen deficit at time (t) (P.P.M. or lbs. per day),

t = distance downstream expressed as time of flow (days),

K_1 = BOD decay rate,

K_2 = reaeration coefficient,

L_a = BOD load (P.P.M. or lbs. per day),

and D_a = initial deficit (P.P.M. or lbs. per day).

At the point where the oxygen profile touches the RQS line, $dD/dt = 0$, $D = D_c$ where D_c equals the deficit to the RQS line, and $L = L_c$ where L_c is the maximum allowable BOD point load. Thus equation 8.1 becomes

$$0 = K_1 L_c - K_2 D_c$$

or

$$K_1 L_c = K_2 D_c. \tag{8.3}$$

The change in BOD with respect to time can be defined as follows. Since the BOD reduction is a first order reaction,

$$L_{(t)} = L_a e^{-K_1 t} .$$

In this case K_1 is the instantaneous rate of BOD reduction.

For the critical time L_c,

$$L_c = L_a e^{-K_1 t_c}. \tag{8.4}$$

Substituting equation 8.4 into 8.3 yields:

$$K_1 L_a e^{-K_1 t_c} = K_2 D_c$$

or

$$L_a = \frac{K_2}{K_1} \frac{D_c}{e^{-K_1 t_c}}. \tag{8.5}$$

T_c has been evaluated by several authors.(1) Its equation is

$$T_c = \frac{1}{K_1(f - 1)} \ln f \; 1 - (f - 1) \frac{D_a}{L_a} \tag{8.6}$$

where $f = K_2/K_1$. Thus substituting 8.6 into 8.5 for T_c, the following expression for L_a is obtained:

$$L_a = f\, D_c\, e^{K_1 \frac{1}{K_1(f-1)} \ln f\, 1 - (f-1)\frac{D_a}{L_a}} \qquad (8.7)$$

It should be noted that L_a appears on both sides of the equation.

For use in determining the point load comparison coefficient it is necessary to determine two values from the above equations. They are the maximum allowable BOD, L_a, and the distance expressed as time to the point where deficit has recovered to the initial deficit. This time is called the recovery time t_r.

The L_a value determined by use of the equation shown above is the total BOD load which can be applied at the considered point of application. This includes both the residual in the stream at that point and the actual applied load. If a river is considered to have no initial BOD, the total L_a value could be applied to the most upstream segment; but each additional point load made at distance t_r downstream must be adjusted for the residual BOD from the upstream loading.

The residual from the upstream load may be evaluated by either of the two following formulas.

$$L_R = L_a\, e^{-K_1 t_r} \qquad (8.8)$$

where L_R = residual BOD (P.P.M. or lbs. per day),

L_a = initial BOD load (P.P.M. or lbs. per day),

K_1 = instantaneous BOD decay rate,

and t_r = recovery time.

Or $$L_R = L_a \; 1 - (1 - K_1')^{t_r} \tag{8.9}$$

where K_1' = BOD decay rate per day.

Thus the allowable BOD point load in a segment of length t_r equals:

$$\begin{aligned} L_P &= L_a - L_R \\ &= L_a - L_a\, e^{-K1 t_r} \\ &= L_a\,(1 - e^{-K1 t_r}). \end{aligned} \tag{8.10}$$

The allowable total continuous loading can be evaluated by the methods of Chapter II for this segment of length t_r as:

$$ALOAD = Q\ t_r\ K_2\ D_c \tag{8.11}$$

where Q = river flow (lbs. per day),

K_2 = reaeration coefficient (day^{-1}),

$D_c = D_{RQS}$ = deficit to the RQS value (lbs. per day),

and Qt_r = water in the segment (lbs.).

The K_1 term shown in equation 2.5 need not be considered since for a uniform flow, the residual BOD in and out of the segment are equal.

It is now known that either a point load, L_P, or a continuous loading, ALOAD, could be assimilated in the segment of length t_r. The point load comparison coefficient, called E for efficiency, is the ratio of this point loading to the continuous loading. Thus,

$$E = \frac{L_P}{ALOAD} = \frac{L_a\,(1 - e^{-K1 t_r})}{Q\ t_r\ K_2\ D_c} \tag{8.12}$$

where L_P and ALOAD are both expressed as pounds per day.

The following steps were used in carrying out the computation of the point load comparison coefficient, E, for a particular set of K_1, K_2, D_a, D_c, and flow values. The first step was the calculation of L_a by a method of successive approximations. This was accomplished by solving for L_a assuming that there was no initial deficit, D_a. Then using this value in the right hand side of equation 8.7 a new L_a value was obtained. If these two L_a values did not agree within .1%, a new value was calculated using the most recent approximation in the right hand side. This procedure continued until the desired accuracy was obtained, at which time the final answer on the left hand side of the equation was accepted as the true L_a value.

This L_a value could have been obtained from the chart prepared by Fair and Guyer which is shown in Illustration 40.(56) In this chart a ratio of the L_a to D_c is obtained if the K_2/K_1 ratio and the D_a/D_c ratios are known. This method of obtaining L_a was not used because the successive approximation technique proved to be easily adapted for computer use, and thus the need for chart reading and for the data cards required to furnish the L_a values to the computer were eliminated.

The second step in the analysis of the point load comparison coefficient was the calculation of T_c by equation 8.6 which was by direct substitution since L_a had been determined.

The third step was the calculation of the recovery time t_r. This was accomplished by increasing T_c several times and solving for $D_{(t)}$ each time by means of equation 8.2. One time value was obtained which gave a larger deficit than D_a; another time value was obtained which gave a smaller deficit than D_a; the correct answer was known to lie

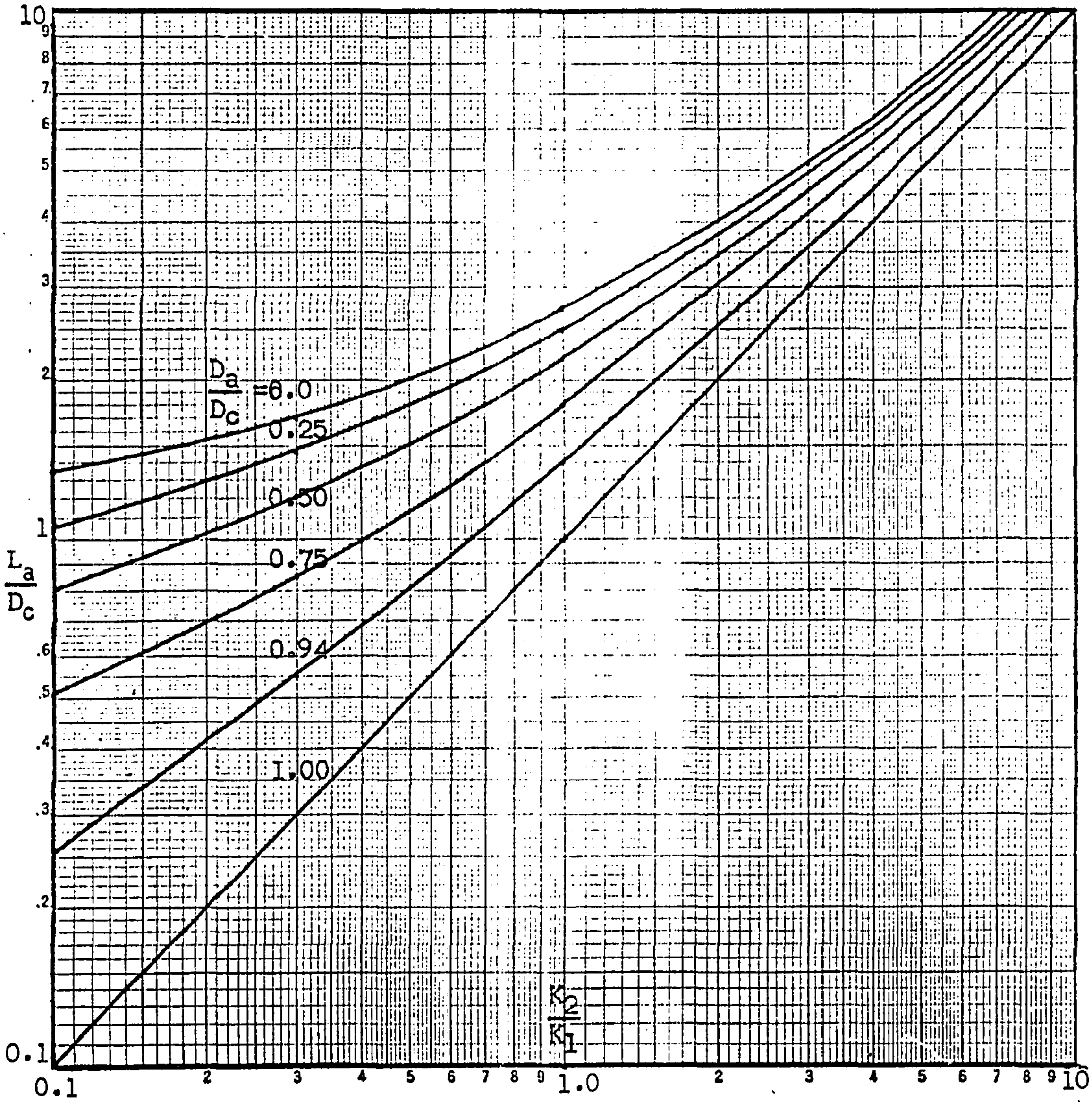


ILLUSTRATION 40

Chart To Determine Allowable Point Loadings somewhere between the two. It was then obtained to a .1% accuracy by using a half interval method of substitution.

The last step was the determination of efficiency, or synonomously the point load comparison coefficient, by means of equation 8.12.

Two computer programs were developed to compute the point load comparison coefficient. The first program was designed to compute the coefficient for 36 different combinations of K_2, D_a, and D_c with a fixed flow and K_1. The second program was designed to compute the coefficient for ten different flows with K_1, K_2, D_a, and D_c held fixed. The computational parts of both programs are identical.

These two programs which are designated Programs 5a and 5b and their corresponding flow charts are shown in the Appendix.

When the results of Program 5b were examined, it was noted that the point load comparison ratio was the same for all ten flows. From this it is concluded that this ratio is independent of the flow.

Since the initial deficit D_a is dependent on the time in which the oxygen is allowed to build up between points, it was decided to plot the efficiency against the passage time t_r for the results obtained from Program 5b. Since the answers for the different D_c values fell on the same lines, it is concluded that with K_1 fixed the efficiency is a function of only the passage time t_r and the reaeration coefficient K_2.

Illustration 41 is the graph mentioned above. It can be seen that if the passage time between loadings t_r and K_2 are known, the point load comparison ratio can be easily determined. The passage time may be determined from the distance in miles between loadings and the mean flow

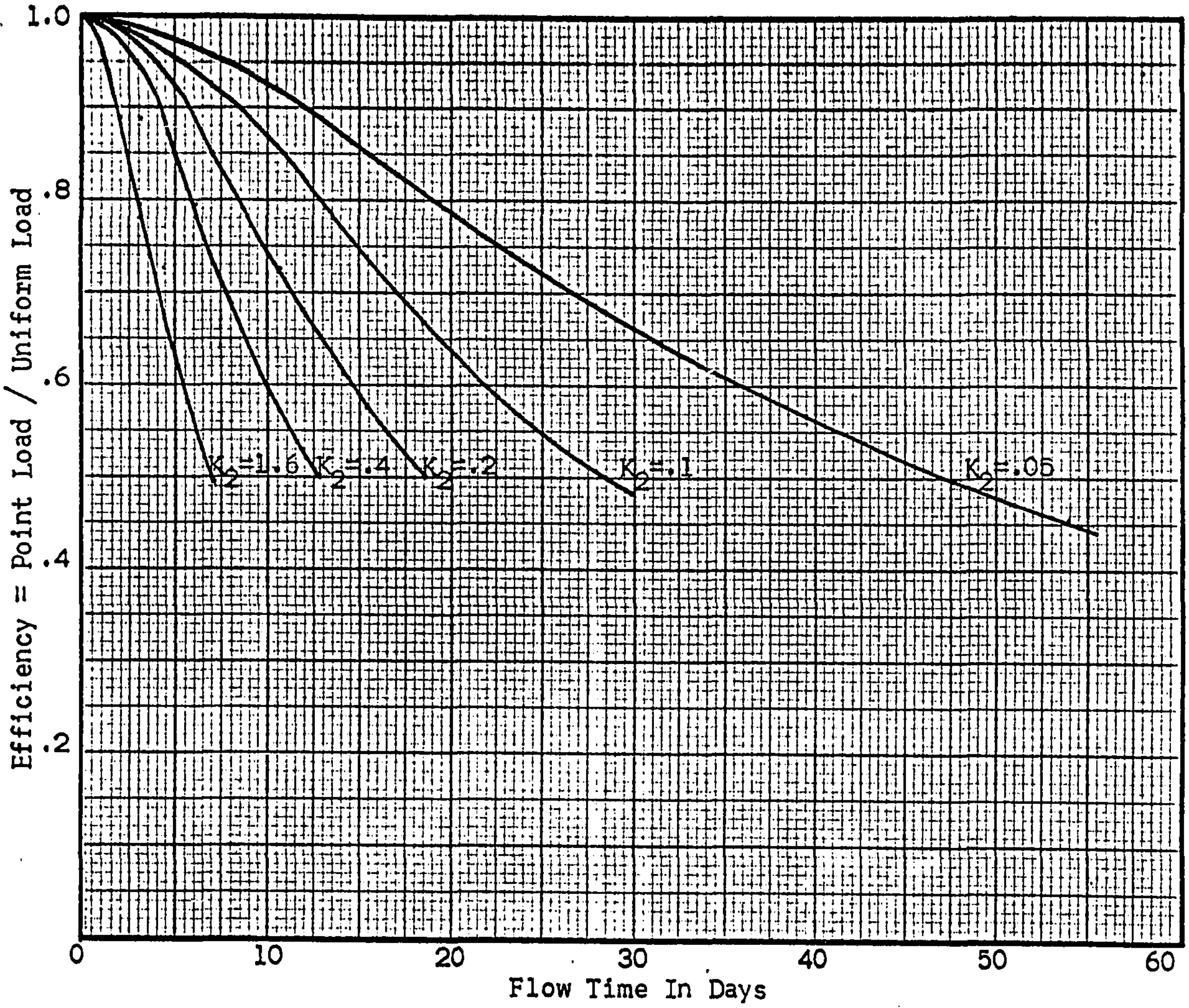


ILLUSTRATION 41

Point Load Efficiency As A Function Of K_2 And Flow Time velocity of the river by the formula:

$$\text{Flow Time in Days} = \frac{\text{Distance in Miles}}{16.36\ (\text{Mean Velocity})} \qquad (8.13)$$

For an example of the use of Illustration 41 it is assumed that loadings 100 miles apart are to be applied to the Ohio River and that the mean velocity of the river flow is .25 feet per second. Thus,

$$\text{Flow Time in Days} = \frac{100}{16.36\ (.25)} = 24.2\ \text{Days}.$$

Assuming a K_2 of .1, a point load comparison coefficient of .58 is read from the chart. Therefore, the waste assimilation capacity determined from the Waste Assimilation Program must be reduced by 42% to correct for the point loadings.

Illustration 42 is a plot of the efficiency against the ratio of D_a to D_c. From this graph it may be noted that the efficiency approaches the maximum 1.0 as D_a approaches D_c. This agrees with the discussion presented earlier in the chapter.

The discussion of the point load comparison has been confined to the river without tidal action. It is considered that the large scale mixing accomplished by the tidal currents in the tidal zone of the river make a point load comparison analysis unnecessary in that zone.

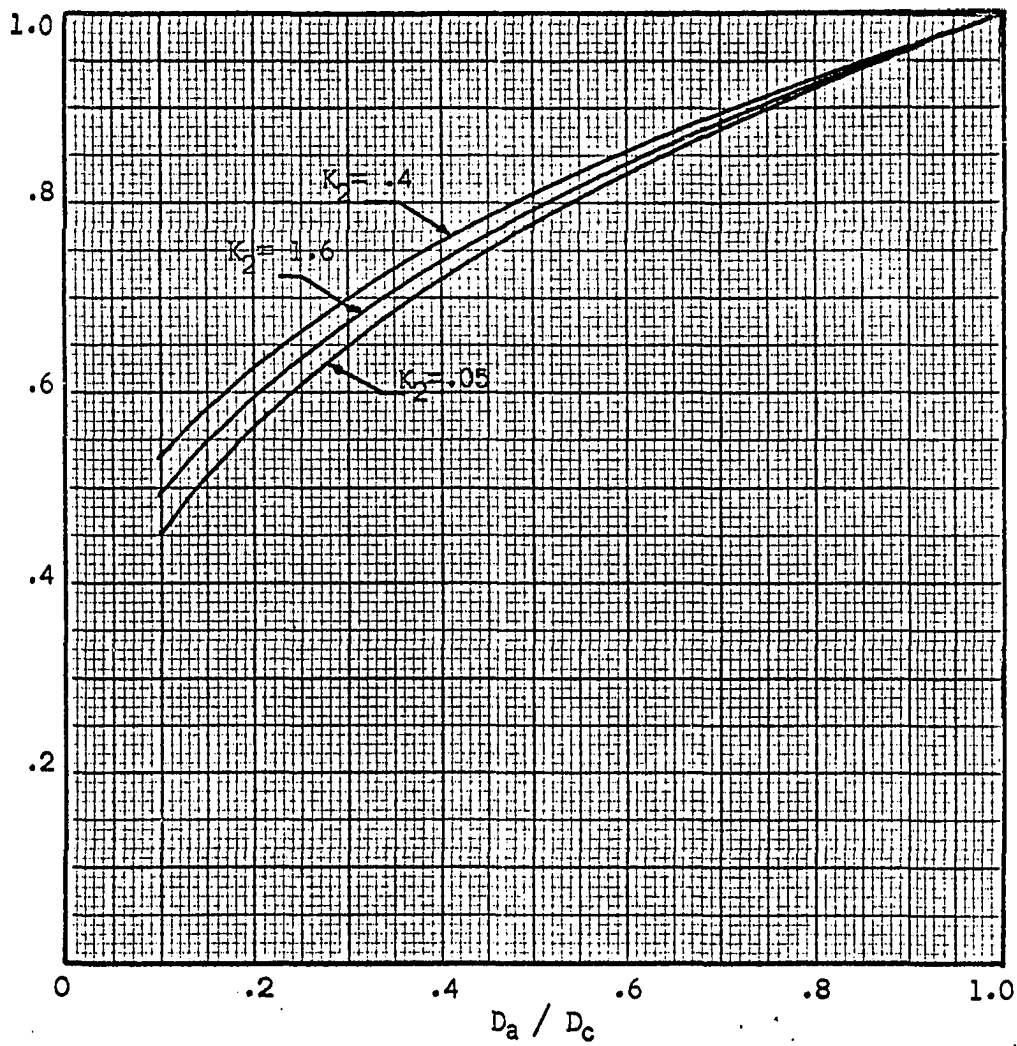


ILLUSTRATION 42

Point Load Efficiency As A Function Of K_2 And D_a/D_c

CHAPTER IX

SUMMARY AND CONCLUSIONS

The purpose of this dissertation has been the development with regard to oxidizable wastes of a new procedure for the estimation of the waste assimilation capacity of a river system. It was desired that this procedure should provide an accurate estimate of the waste assimilation capacity for the tidal and non-tidal portions of a river and was furthermore desired that the procedure should be dependent only on data which is readily attainable without actual field measurement.

The method chosen for this procedure was a numerical integration utilizing small segments of river length. This method had several advantages which were: (1) the method could be developed from existing stream pollution formulas, (2) the waste assimilation capacity for any group of segments could be summed up, whether they were for a small portion of a river, an entire river, or an entire system of rivers, (3) digital computer techniques could be easily applied to the method, and (4) the results given are the ultimate waste loadings which a segment or segments can assimilate and therefore, are independent of the location of population and industrial centers.

The theory of the procedure of analyzing the river by segments was developed in Chapter II. The corresponding theory to apply the numerical integration technique to the calculation of the traditional dissolved oxygen sag curve was also developed in Chapter II. The latter was developed primarily as a tool for testing computer techniques against existing studies.

Chapter III included an extensive discussion of the tidal phenomena and its effect on tidal rivers, particularly with regard to waste assimilation. From this discussion on tides, the methods presented in Chapter II were modified so that they would apply to the tidal zone of a river exhibiting a homogeneous salinity structure. In addition, a method was outlined which would also permit the analysis of river tidal zones exhibiting highly stratified or mixed salinity structures.

In Chapter IV various coefficients necessary for the evaluation of the formulas presented in Chapters II and III were developed and discussed, and correction factors were introduced to account for temperature and salinity changes.

In order to test the feasibility of the procedures mentioned above and to evaluate their use in actual stream pollution analysis, it was decided to program the procedures for both the tidal and non-tidal rivers for use on an I.B.M. 1620 computer. This resulted in the development of two major computer programs: the Waste Assimilation Program, and the Tidal Zone Waste Assimilation Program. In brief, the process of these programs was to read the data describing the channel and flow characteristics at two ends of a river segment and to compute from this data the waste assimilation capacity of the segment. This process was then repeated over and over again utilizing the next downstream segment until the entire river had been analyzed. The Waste Assimilation Program was designed to process an entire river system. The tidal zone program, however, was limited to a single river.

Four major applications were made with these programs. These were the 1980 Ohio River Study, the 1960 Ohio River Basin Study, the Wabash River Study, and the Delaware River Study.

The 1980 Ohio River Study consisted of the computation of the waste assimilation capacity for the Ohio River using the same stream data used in a recent U.S. Public Health Service Study. This data is that anticipated for the year 1980. The results obtained from the computer indicated a close correlation between the two studies.

The 1960 Ohio River Basin Study was the analysis of the waste assimilation capacity of 21 rivers in the Ohio River basin at a condition of low river flow. Extensive data on stream flow was collected from U.S. Geological Survey files and publications and from other minor sources. From this data the ten year low flow and the corresponding stream channel characteristics were determined. The Waste Assimilation Program was then utilized to determine the waste assimilation for each ten mile segment of these rivers and the corresponding sums developed for each river and the entire basin. The results indicate that at the ideal loading distribution the basin can assimilate wastes which have an equivalent 6,376,000 pounds of biochemical oxygen demand (BOD) per day. This is the average waste loading of a population of 25,504,000 persons.

The Wabash River Study was an extension of a portion of the 1960 Ohio River Basin Study to determine the effect of increased river flow and increased temperature on the waste assimilation capacity. To do this the stream conditions were determined for river flows equal to 100%, 150%, and 200% of the low flows determined in the Ohio River Basin Study, and all three conditions were processed by the Waste Assimilation Program. The results indicated that the waste assimilation capacity is increased .55% by every 1% increase in river flow. Similarly, a single river flow condition was processed at three different temperatures. The results from this study indicated that a one degree centigrade increase in water temperature lowered the waste assimilation capacity by 2%.

The Delaware River Study included the use of both waste assimilation capacity programs. Data for both the non-tidal and tidal portions of this river was collected from the U.S. Geological Survey, the U.S. Coast and Geodetic Survey, and the U.S. Army Corps of Engineers. After being processed this data was utilized by the Waste Assimilation Program for the entire river length and by the Tidal Zone Waste Assimilation Program for the tidal zone only. In the river zone affected by tides it was found that the analysis considering tides indicated a waste assimilation capacity roughly six times as large as the analysis ignoring tidal action. This result emphasized the need of considering the tidal effect.

The waste assimilation capacity concept is based on the assumption that waste loadings will be made in the individual segment which has assimilation capacity. Since in actual practice the condition does not usually happen, Chapter VIII consisted of an analysis to evaluate non-ideal waste loadings.

A number of conclusions may be drawn from the procedures and applications of this dissertation.

It has been concluded that adequate data is presently available from various sources which can be used to provide accurate and fairly detailed estimates of the waste assimilation capacity of the rivers of the United States. This data which is available for both the tidal and non-tidal zones of rivers can be obtained and reduced to usable form with a nominal amount of effort and expense.

The analysis procedures developed in this dissertation for analyzing the oxidizable waste assimilation capacities of rivers with and without tidal action are based on existing accepted stream analysis theory. They have a great advantage over the procedures used in current practice, because whereas the presently used procedures require the idealization of the stream into relatively long stretches with constant channel flow and reaeration characteristics, the procedures of this dissertation permit the use of the actual values to the maximum extent to which they can be determined. The new procedures are economical in comparison to previous methods primarily because of their adaptation to digital computer techniques.

The new techniques are very flexible and are accurate within the limits imposed by present knowledge of the various coefficients used in the formulas, namely the BOD decay coefficient, the reaeration coefficient, the saturation dissolved oxygen content, and the minimum allowable stream dissolved oxygen content.

The basic technique can be modified to consider other wastes as time and needs require. In view of the above it is concluded that these procedures can become a valuable tool in the field of water resources analysis and planning. This major conclusion is supported by the results of the various applications which have been made of these procedures and from the additional conclusions which can be drawn from the applications themselves.

The waste assimilation capacity values determined by the 1960 Ohio River Basin Study and the Delaware River Study, which are discussed in Chapter VII, may be used as an example to demonstrate how these results may be used in determining future waste treatment requirements. Table 12 consists of predicted populations for the various sub-basins in the Ohio River Valley for the years 1980 and 2000. These values were determined by distributing the total basin urban population predictions published by the Senate Select Committee on Water Resources in proportion to the urban population values for 1940 which were given in a 1943 study of the Ohio River.(46, 57)

The Public Health Service study of future Ohio River conditions estimates that the total pollution load in population equivalents for 1980 is 2.12 times the urban population, and for the year 2000 it is 2.36 times the urban population.(30)

The pollution loads computed using the above factors for 1980 and 2000 are listed in the first two columns of Table 13. Column 3 consists of the waste assimilation capacity for the individual sub-basins as determined by the Waste Assimilation Program, and Column 4 consists of the waste assimilation capacity values obtained by assuming that non-continuous loading requires a 40% reduction in the values of Column 3.

The required degree of treatment may be obtained for a particular river basin by dividing the waste loading minus the waste assimilation

TABLE 12

URBAN POPULATION - OHIO RIVER BASIN

	Population In Thousands			
	1940	1960	1980	2000
Ohio	3292	5815	8474	11,855
Monongahela	585	1033	1506	2106
Beaver	477	843	1228	1718
Muskingum	398	703	1025	1433
Little Kanawha	0	0	0	0
Hocking	48	85	124	173
Kanawha	176	311	453	634
Guyandot	8	14	21	29
Big Sandy	31	55	80	112
Scioto	448	791	1153	1613
Miami	502	887	1293	1808
Kentucky	96	170	247	346
Green	44	78	113	158
Wabash	1198	2116	3083	4314
Total	7303	12,900	18,800	26,300

TABLE 13

OHIO RIVER BASIN WASTE TREATMENT REQUIREMENTS

	Assimilation Capacity P.E. (10^{-3})		Waste Loading P.E. (10^{-3})		Degree of Treatment in Percent	
	Actual	Modified	1980	2000	1980	2000
Ohio	15,837	9,502	17,965	27,978	47.1	66.0
Monongahela	1,088	653	3,192	4,970	79.5	86.9
Beaver	211	127	2,603	4,054	95.1	96.9
Muskingum	313	188	2,173	3,382	91.3	94.4
Hocking	77	46	263	408	82.5	88.7
Kanawha	2,033	1,220	960	1,496	00.0	18.4
Guyandot	80	48	45	68	00.0	29.4
Big Sandy	134	80	170	264	52.9	69.7
Scioto	350	210	2,444	3,807	91.4	94.5
Miami	320	192	2,741	4,266	93.0	95.7
Kentucky	425	255	524	817	51.3	68.8
Green	550	330	240	373	00.0	71.5
Wabash	3,653	2,192	6,536	10,181	66.5	78.5
Total Basin	35,071	15,043	39,856	62,064	62.2	75.8

capacity by the waste loading. The treatment requirements for the rivers in the Ohio basin expressed in percent are shown in Columns 5 and 6 of Table 13 for the years 1980 and 2000. It may be noted that the requirements range from 0.0% to 95.1% for the year 1980 and from 11.5% to 96.9% for the year 2000 with an average value for the entire basin of 62.2% in 1980 and 75.8% in 2000.

Table 14 is a similar analysis for the Delaware River. In this example the predicted waste loading for the Philadelphia standard metropolitan area is considered applied to the stretch of river between Trenton, N. J., and Delaware City, Delaware, a river distance of 70 miles. The oxidizable waste loading in population equivalents is considered to be 2.0 times the population. It is assumed that the combination of spread out waste discharge and tidal mixing makes it unnecessary to reduce the waste assimilation capacity to allow for non-continuous loading. From the results shown in Table 14 it may be seen that the degree of treatment in percent required to maintain a four parts per million RQS is 45.7% at present, 59.3% by 1980, and 68.4% by 2000.

TABLE 14

DELAWARE RIVER WASTE TREATMENT REQUIREMENTS

	1960	1980	2000
Population (10^{-3})	4,310	5,750	7,410
Population Equivalents (10^{-3})	8,620	11,500	14,820
Waste Assimilation Capacity Population Equivalents (10^{-3})	4,683	4,683	4,683
Treatment Required %	45.7	59.3	68.4

The results determined by the use of the Waste Assimilation Program in the Wabash River Study have led to conclusions regarding flow augmentation and thermal pollution. It was shown that for the Wabash River an increase of 100% in stream flow increased the waste assimilation capacity by 55%. Assuming that this value is valid for the other sub-basins in Indiana and Ohio, the low flow requirements as an alternative to increased waste treatment may be evaluated.

For example, if the maximum economical waste treatment is estimated to be 90% in 1980, the cost of additional high cost treatment above 90% must be balanced against the cost of low flow augmentation. In Table 15 the flow increase over the 10 year low flow which is necessary to raise the waste assimilation capacity to 10% of the total waste loading has been computed for each of the rivers which required over 90% treatment in Table 13.

TABLE 15

FLOW AUGMENTATION ANALYSIS - 1980

	Waste in Loading P.E. (10^{-3})	Waste Assimilation at Low Flow P.E. (10^{-3})	Waste Assimilation Required at 90% Treatment	Flow Augmentation Required
Beaver	2603	127	260	189.0%
Muskingum	2173	188	217	29.3%
Scioto	2444	210	244	29.4%
Miami	2741	192	274	77.7%

The thermal pollution which exists in the Mahoning River of the Beaver River basin has raised the water temperature to about six degrees centigrade over the normal water temperature. From the results of the Wabash River Temperature Study it may be concluded that the waste assimilation capacity for oxidizable wastes for thermally polluted portions of this river would be about 12% higher if this pollution were eliminated.

On a basin wide basis the results from the 1960 Ohio River Basin Study can be compared to the results obtained by Reid in his report to the United States Senate Select Committee on National Water Resources. (2) Reid has estimated that a river flow of 14,000 million gallons per day, or 21,658 cubic feet per second, can assimilate 13.17 million population equivalents of waste loading per day. Not considering the flows of the Tennessee and Cumberland Rivers the results of the 1960 Ohio River Basin Study assuming a 60% efficiency factor for non-continuous loading indicate that a flow of 12,540 cubic feet per second can assimilate a waste loading of 15.043 million population equivalents.

Since according to the Wabash River Study the percent increase in waste assimilation capacity equals .55 times the increase in flow, the flow and waste assimilation capacity from the 1960 Ohio River Basin Study can be revised upward so they can be compared with the values of Reid. The flow increase from 12,450 to 21,658 cubic feet per second is 73.16%, and therefore, the waste assimilation capacity is increased 40.24% to equal 21.095 million population equivalents per day. Thus, the results of this report indicate a waste assimilation capacity roughly 60% greater than those obtained by Reid. This seems reasonable in view of his conservative assumptions.

Reid further estimated that a flow of 16,400 million gallons per day, or 25,370 cubic feet per second, could assimilate 15.8 million population equivalents per day. The values given for the Ohio River basin may be modified to give the corresponding flow for a 15.8 million population equivalent load. The 5.03% increase in waste assimilation capacity is equal to a 9.14% increase in flow, and therefore, the necessary flow becomes 13,686 cubic feet per second. Thus for an equal waste loading the results of this report require only 54% of the flow estimated by Reid.

The procedures presented in this dissertation have utilized existing data and have provided realistic estimates of the oxidizable waste assimilation capacity of rivers which are independent of conditions other than temperature, stream flow and velocity, and river channel configuration. The applications utilizing the procedures have shown them to be accurate, fast, and relatively inexpensive and have shown that much useful information is gained for use in water resources planning. It is believed that much useful information could be obtained by applying these procedures to all the major river basins in the United States.

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APPENDIX

COMPUTER PROGRAMS AND FLOW CHARTS

Program One

Data Generating Program

The purpose of this program is to compute input data for segments between points of known data. It will also compute flow, area, depth, and elevation for a point of known change in flow located between two known data points. See text for a complete description.

Program statements beginning with "C" are comment statements which do not affect the program processes.

Program Notation Key

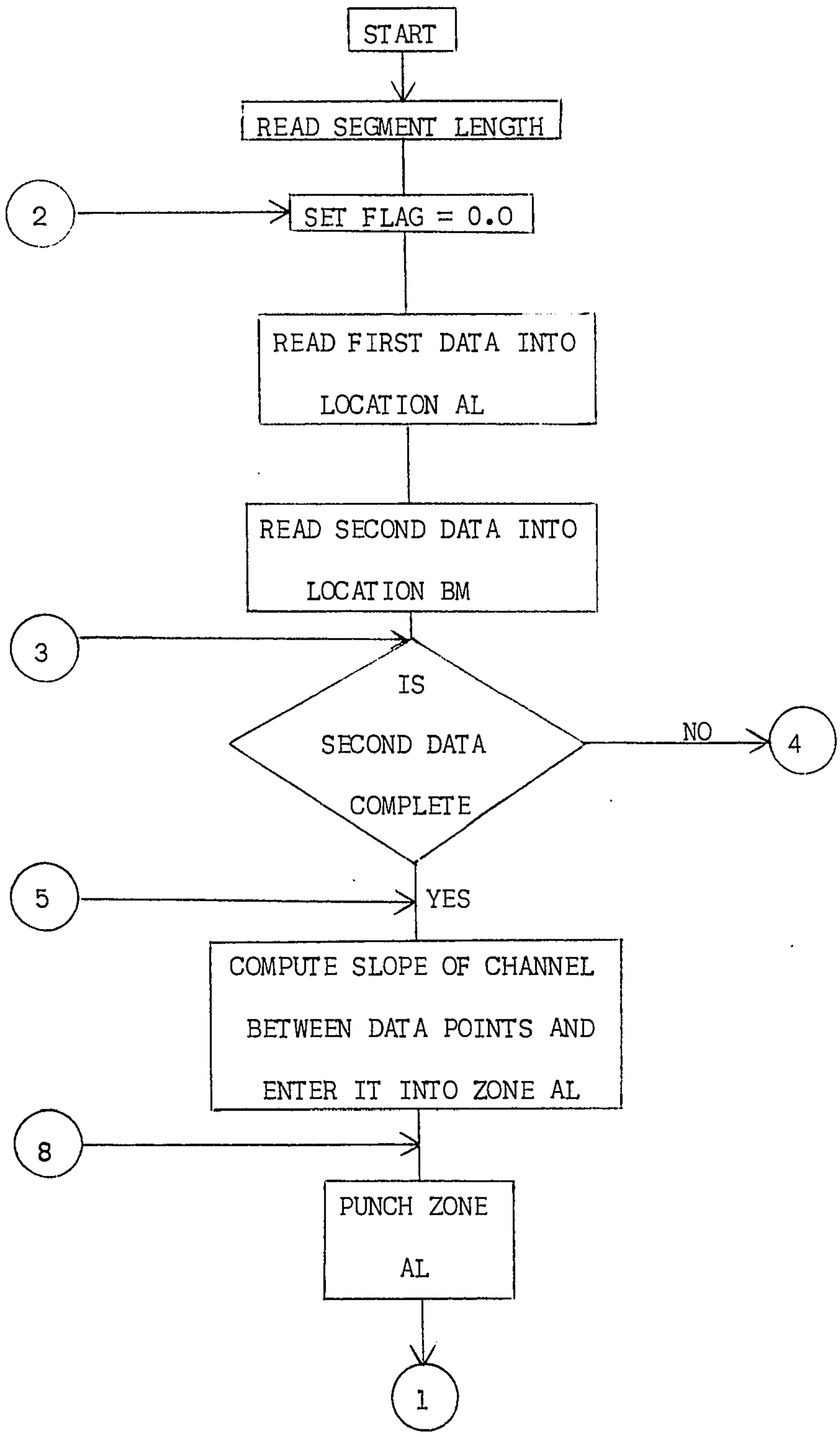
ISL	Segment length - fixed point number
FSL	Segment length - floating point number
L(1) = M(1) = N(1)	River Number
L(2) = M(2) = N(2)	River Mile
L(3) = M(3) = N(3)	Always Zero
L(4) = M(4) = N(4)	Complete Data Flag
L(5) = M(5) = N(5)	Order of River
A(1) = B(1) = C(1)	River Mile
A(2) = B(2) = C(2)	Temperature
A(3) = B(3) = C(3)	River Quality Standard
A(4) = B(4) = C(4)	Flow
A(5) = B(5) = C(5)	Flow Added in Segment
A(6) = B(6) = C(6)	Mean Depth of River
A(7) = B(7) = C(7)	Area of Cross Section
A(8) = B(8) = C(8)	Elevation

A(9) = B(9) = C(9) Slope

A(10) = B(10) = C(10) Initial D.O. Deficit

A(11) = B(11) = C(11) Added BOD

Flow Chart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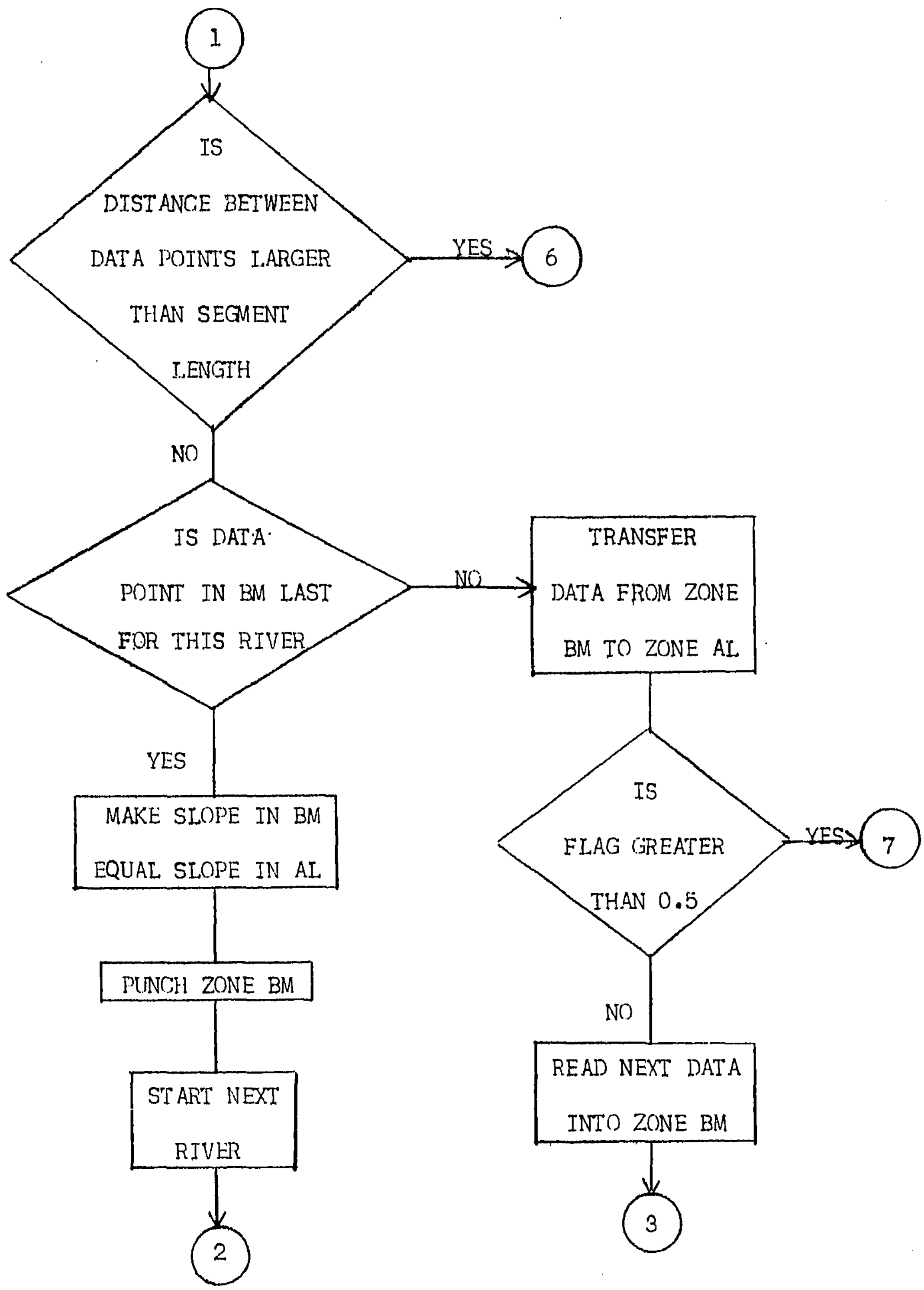
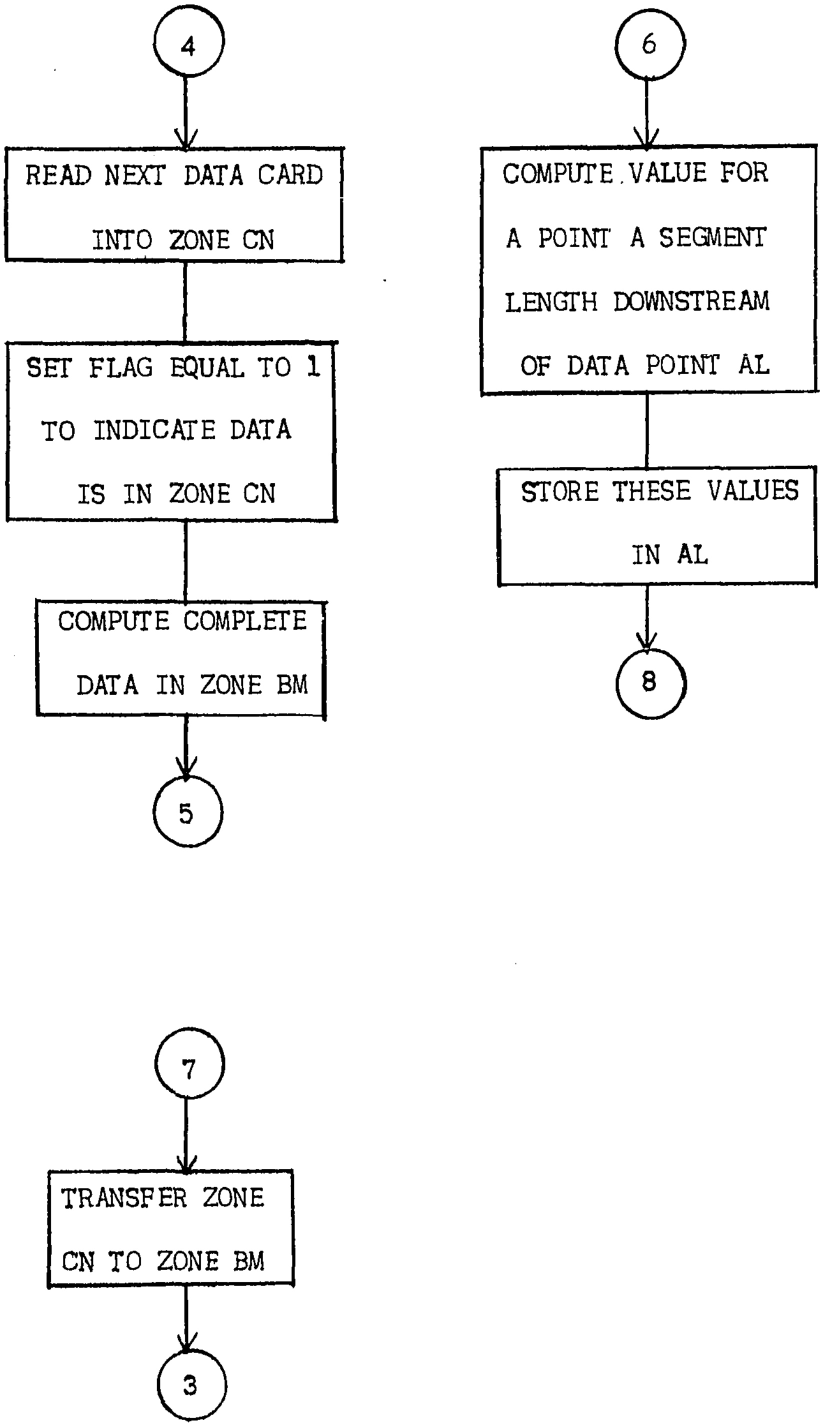
READ SEGMENT LENGTH
2
SET FLAG = 0.0
READ FIRST DATA INTO LOCATION AL
READ SECOND DATA INTO LOCATION BM
3
IS SECOND DATA COMPLETE
NO
4
5
YES
COMPUTE SLOPE OF CHANNEL BETWEEN DATA POINTS AND ENTER IT INTO ZONE AL
8
PUNCH ZONE AL
1


1
IS
DISTANCE BETWEEN
DATA POINTS LARGER
THAN SEGMENT
LENGTH
YES
6
NO
IS DATA
POINT IN BM LAST
FOR THIS RIVER
NO
TRANSFER
DATA FROM ZONE
BM TO ZONE AL
YES
MAKE SLOPE IN BM
EQUAL SLOPE IN AL
PUNCH ZONE BM
START NEXT
RIVER
2
IS
FLAG GREATER
THAN 0.5
YES
7
NO
READ NEXT DATA
INTO ZONE BM
3


4
READ NEXT DATA CARD INTO ZONE CN
SET FLAG EQUAL TO 1 TO INDICATE DATA IS IN ZONE CN
COMPUTE COMPLETE DATA IN ZONE BM
5
6
COMPUTE VALUE FOR A POINT A SEGMENT LENGTH DOWNSTREAM OF DATA POINT AL
STORE THESE VALUES IN AL
8
7
TRANSFER ZONE CN TO ZONE BM
3

Program Listing

```
C      DATA GENERATING PROGRAM
C      PREPARED BY ROY W. HANN, JR.
C      READ SEGMENT LENGTH
  110 Read 2,ISL,FSL
    2 FORMAT(14,E12.5)
  115 DIMENSION A(11),B(11),C(11),F(2),L(5),M(5),N(5)
C      SET FLAG = 0.0
  120 F(1)=0.0
C      READ FIRST DATA
  130 READ 3,L(1),L(2),L(3),L(4),L(5),A(1),A(2),A(3),A(4),A(5)
    3 FORMAT (13,15,11,11,12,E12.5,E12.5,E12.5,E12.5,E12.5)
  131 READ 4,A(6),A(7),A(8),A(9),A(10),A(11)
    4 FORMAT(E12.5,E12.5,E12.5,E12.5,E12.5,E12.5)
  140 IF(F(1)-0.5)141,141,225
C      READ SECOND DATA
  141 READ3,M(1),M(2),M(3),M(4),M(5),B(1),B(2),B(3),B(4),B(5)
  142 READ4,B(6),B(7),B(8),B(9),B(10),B(11)
C      IF M(4)=0 DATA IS COMPLETE
C      IF M(4) NOT = 0 READ NEXT CARD INTO CN AT STATEMENT 210
  150 IF (M(4)-0) 160,160,210
  160 A(9) = (A(8)-B(8))/(5280.* (A(1)-B(1)))
C      PUNCH ZONE AL
  165 PUNCH 3,L(1),L(2),L(3),L(4),L(5),A(1),A(2),A(3),A(4),A(5)
  166 PUNCH 4,A(6),A(7),A(8),A(9),A(10),A(11)
C      IS DISTANCE BETWEEN GIVEN STATIONS GREATER THAN SEG. LENGTH
```

```
C     175 IS NO, 190 IS YES
  169 DIST=A(1)-B(1)
  170 IF(DIST-FSL)175,175,190
C     IS THIS LAST DATA FOR RIVER 176 YES, 180 NO
C     IF (M(2)-0) ERROR,END, NOT END
  175 IF(M(2)-0) 176,176,180
  176 B(9)=A(9)
C     PUNCH LAST CARD FROM BM
  177 PUNCH3,M(1),M(2),M(3),M(4),M(5),B(1),B(2),B(3),B(4),B(5)
  178 PUNCH4,B(6),B(7),B(8),B(9), B(10),B(11)
C     GO TO NEXT RIVER
  179 GO TO 100
  180 DO 181 1=1,5
  181 L(1)=M(1)
  182 DO 183 1=1,11
  183 A(1)=B(1)
C     180-183 TRANSFER B TO A THEN READ NEXT DATA
  184 GO TO 140
  190 L(2)=L(2)-ISL
  191 RATIO = FSL/(A(1)-B(1))
  192 A(2)=A(2)+RATIO*(B(2)-A(2))
  193 A(4)=A(4)+RATIO*(B(4)-B(5)-A(4))
  194 A(5)=0.0
  195 A(6)=A(6)+RATIO*(B(6)-A(6))
  196 A(7)=A(7)+RATIO*(B(7)-A(7))
  197 A(8)=A(8)+RATIO*(B(8)-A(8))
```

```
  198 A(10)=0.0
  199 A(11)=0.0
  200 A(1)=A(1)-FSL
C     L(1,3,4,5),A(8,9) REMAIN THE SAME
  201 GO TO 165
  210 F(1)=1.0
C     READ NEXT CARD INTO CN
  211 READ3,N(1),N(2),N(3),N(4),N(5),C(1),C(2),C(3),C(4),C(5)
  212 READ 4,C(6),C(7),C(8),C(9),C(10),C(11)
  215 RATIO= (A(1)-B(1))/(A(1)-C(1))
  216 B(2)=A(2)+RATIO*(C(2)-A(2))
  217 B(4)=A(4)+B(5)+RATIO*(C(4)-C(5)-B(5)-A(4))
  218 B(6)=A(6)+RATIO*(C(6)-A(6))
  219 B(7)=A(7)*B(4)/A(4)
  220 B(8)=A(8)+RATIO*(C(8)-A(8))
C     M(1-5),B(1,3,5,10,11,)  MUST BE FURNISHED BY DATA
  221 M(4)=0
  222 GO TO 160
  225 F(1)=0.0
  226 DO 227 1=1,5
  227 M(1)=N(1)
  228 DO 229 1=1,11
  229 B(1)=C(1)
  230 GO TO 160
C     225-230 TRANSFER SIDETRACTED DATA FROM CN TO BM
  231 STOP
  232 END
```

Program Two

Waste Assimilation Program For A River Without Tides

The purpose of this program is to compute the waste assimilation capacity for the streams in a river basin. It will also compute the dissolved oxygen profile of the river if applied BOD loadings are furnished.

Program statements beginning with "C" are comment statements which do not affect the program processes.

Program Notation Key

ZoneAL1	ZoneAL2	ZoneAL3	ZoneBM	
L(1)	L(6)	L(11)	M(1)	River Number
L(2)	L(7)	L(12)	M(2)	River Mile
L(3)	L(8)	L(13)	M(3)	Always Zero
L(4)	L(9)	L(14)	M(4)	Complete Data Flag
L(5)	L(10)	L(15)	M(5)	Order of River
A(1)	A(18)	A(35)	B(1)	River Mile (Miles)
A(2)	A(19)	A(36)	B(2)	Temperature (Degrees C)
A(3)	A(20)	A(37)	B(3)	River Quality Standard (p.p.m.)
A(4)	A(21)	A(38)	B(4)	Flow (c.f.s.)
A(5)	A(22)	A(39)	B(5)	Flow Added In Segment (c.f.s.)
A(6)	A(23)	A(40)	B(6)	Mean Depth of River (ft.)
A(7)	A(24)	A(41)	B(7)	Area of Cross Section (sq. ft.)
A(8)	A(25)	A(42)	B(8)	Elevation (ft.)
A(9)	A(26)	A(43)	B(9)	Slope (ft./ft.)

A(10)	A(27)	A(44)	B(10)	Initial D.O. Deficit (p.p.m.)
A(11)	A(28)	A(45)	B(11)	Added BOD (lbs/day)
A(12)	A(29)	A(46)	B(12)	Dissolved Oxygen Level (p.p.m.)
A(13)	A(30)	A(47)	B(13)	Previous Segment K1 Value
A(14)	A(31)	A(48)	B(14)	Previous Segment Total Allowable BOD (lbs/day)
A(15)	A(32)	A(49)	B(15)	Total Applied BOD Previous Segment (lbs/day)
A(16)	A(33)	A(50)	B(16)	Previous Segment Total Basin BOD Load (lbs/day)
A(17)	A(34)	A(51)	B(17)	Total Passage Time (days)

ABOD BOD Added to Segment (lbs/day)

ADDDO Change in D.O. Level Casued by XTRAO (p.p.m.)

ADDQ Flow Added in Segment (c.f.s.)

ADDV Weight of Water Added to Segment Per Day (lbs.)

ADEEP Mean Depth in Segment (ft.)

AD021 Added Oxygen Per Day with River at RQS (lbs/day)

AD022 Added Oxygen for D.O. Plot

ALOAD Allowable BOD Load that Could be Added to Previous Segment (lbs/day)

APBOD Actual BOD Applied to Segment - Upstream + Added (lbs/day)

ATEMP Average Temperature (Degrees C)

DEF1 Oxygen Deficit to River Quality Standard (p.p.m.)

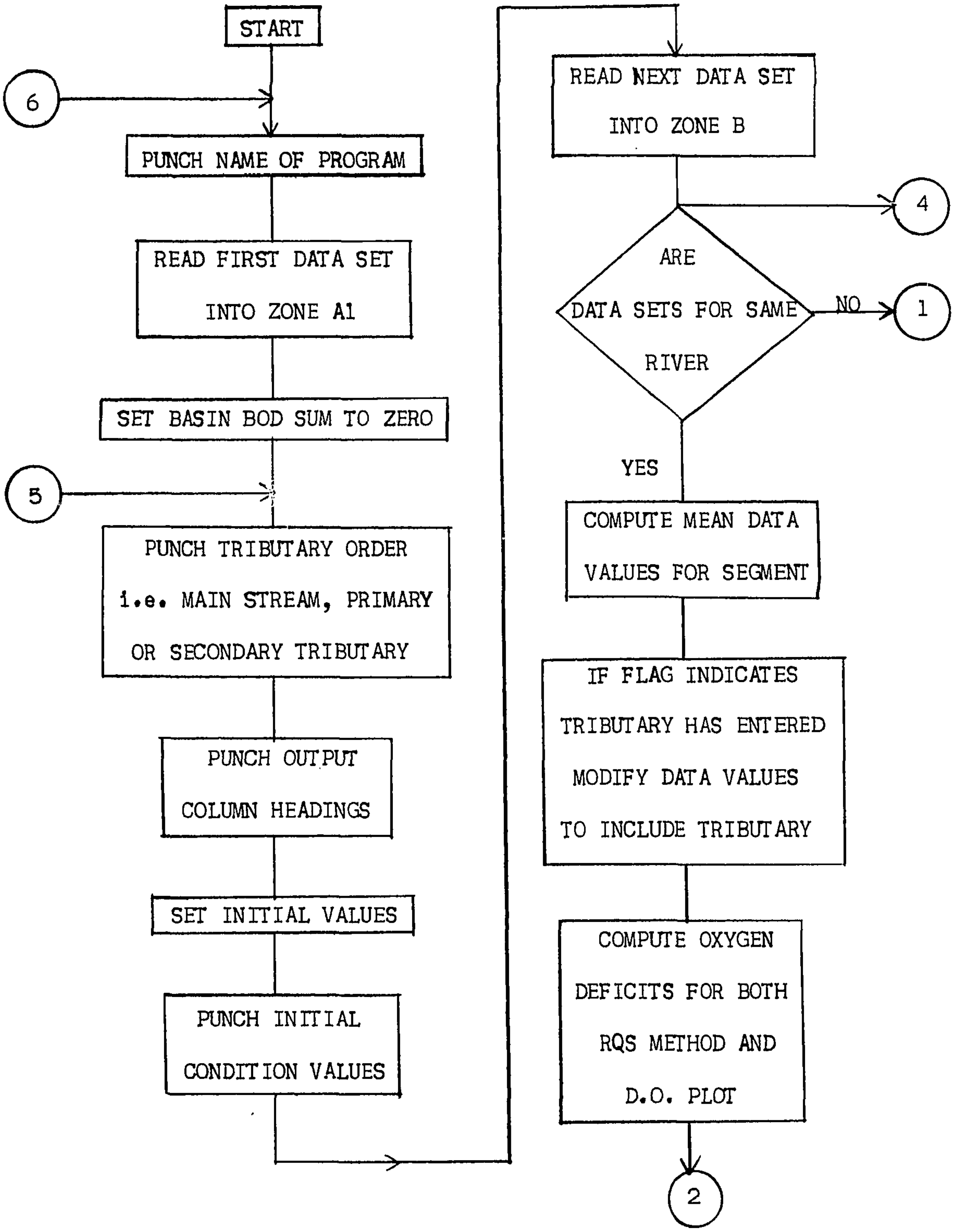
DEF2 Oxygen Deficit to Stream D.O. Level (p.p.m.)

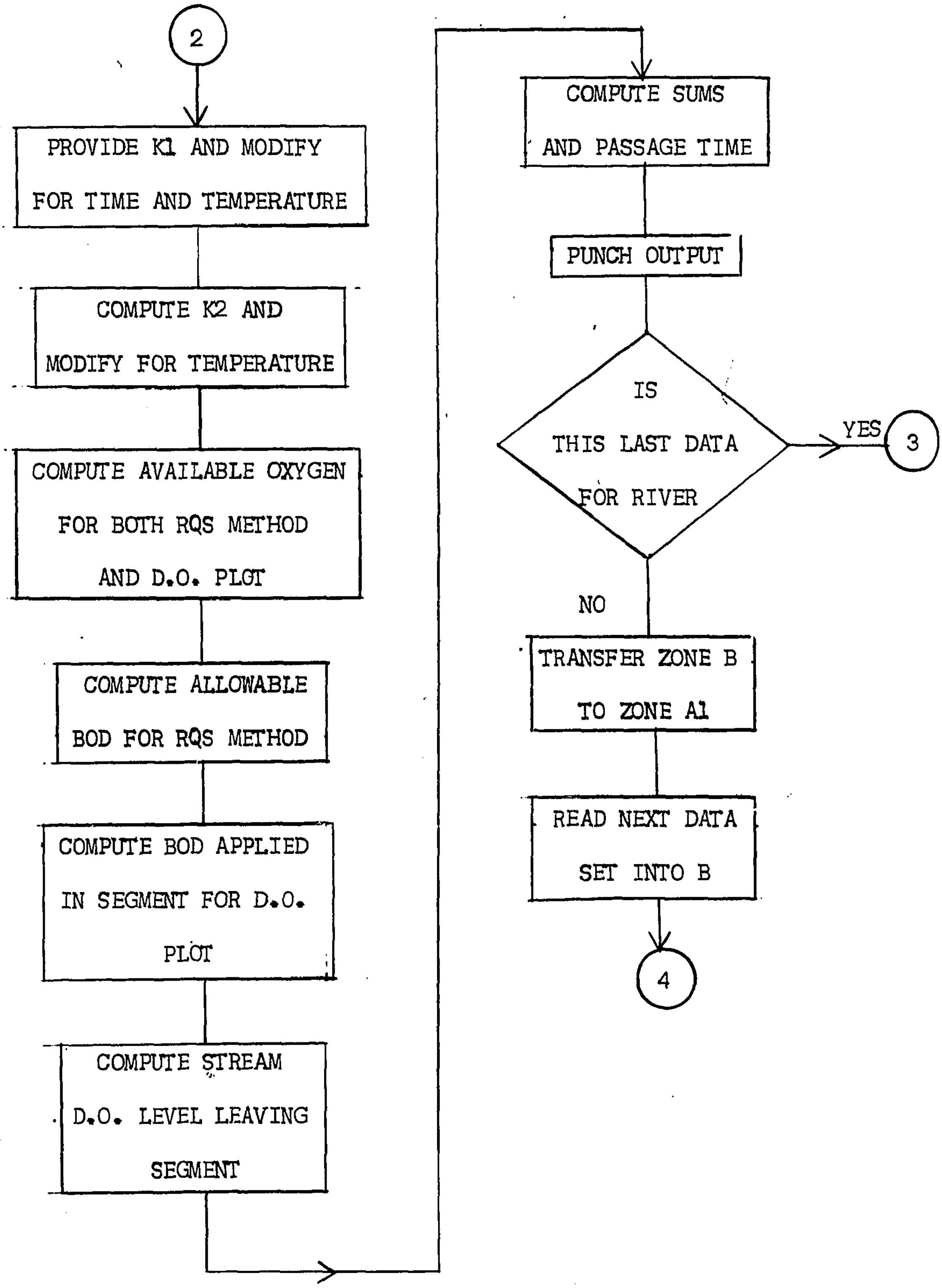
D02 Dissolved Oxygen Originally in Stream at Given Temperature (p.p.m.)

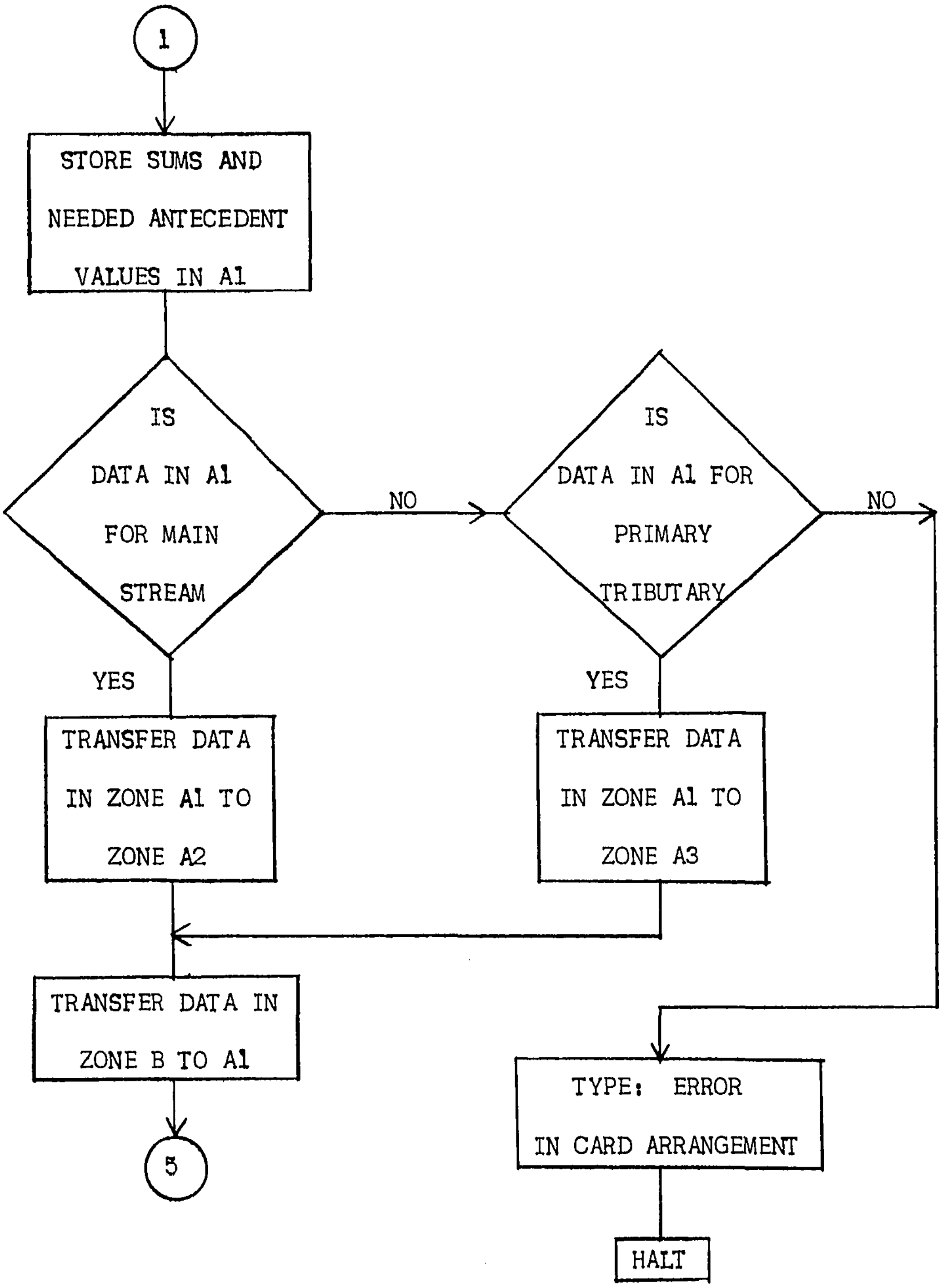
EFBOD	Effective BOD Applied to Segment for D.O. Plot (lbs/day)
FKTT	K1 Value Adjusted for Time and Temperature
FKTTL	K1 for Previous Segment
FK20	K1 at 20 Degrees Centigrade
FK20T	FK20 Modified for Time in Segment
HOBOD	Residual Allowable BOD from Upstream Segment (lbs/day)
ORDEF	Original Deficit in Inflow Water (p.p.m.)
ORDF2	Oxygen Deficit between Saturated D.O. and RQS
PTIME	Total Passage of Time (days)
Q	Mean Flow in Segment (c.f.s.)
QMEAN	Mean Flow in Segment (c.f.s.)
RABOD	Residual Applied BOD from Upstream Segment for D.O. Plot (lbs/day)
RBOD	Residual Allowable BOD From Tributary (lbs/day)
REO2	Reoxygenation with River at RQS (lbs/day)
REO2P	Reoxygenation for D.O. Plot (lbs/day)
RQS	River Quality Standard for Dissolved Oxygen (p.p.m.)
SATDO	Saturation Dissolved Oxygen Content in Water at Given Temperature (p.p.m.)
SEGL	Segment Length (miles)
SKT	K2 at Given Temperature
SK20	K2 at 20 Degrees Centigrade
SLOAD	Total Allowable BOD on River Being Considered (lbs/day)
SLOPE	River Channel Slope (ft./ft.)
TABOD	Total Allowable BOD Loading in Previous Segment (lbs/day)
TLAOP	Available Oxygen for D.O. Plot

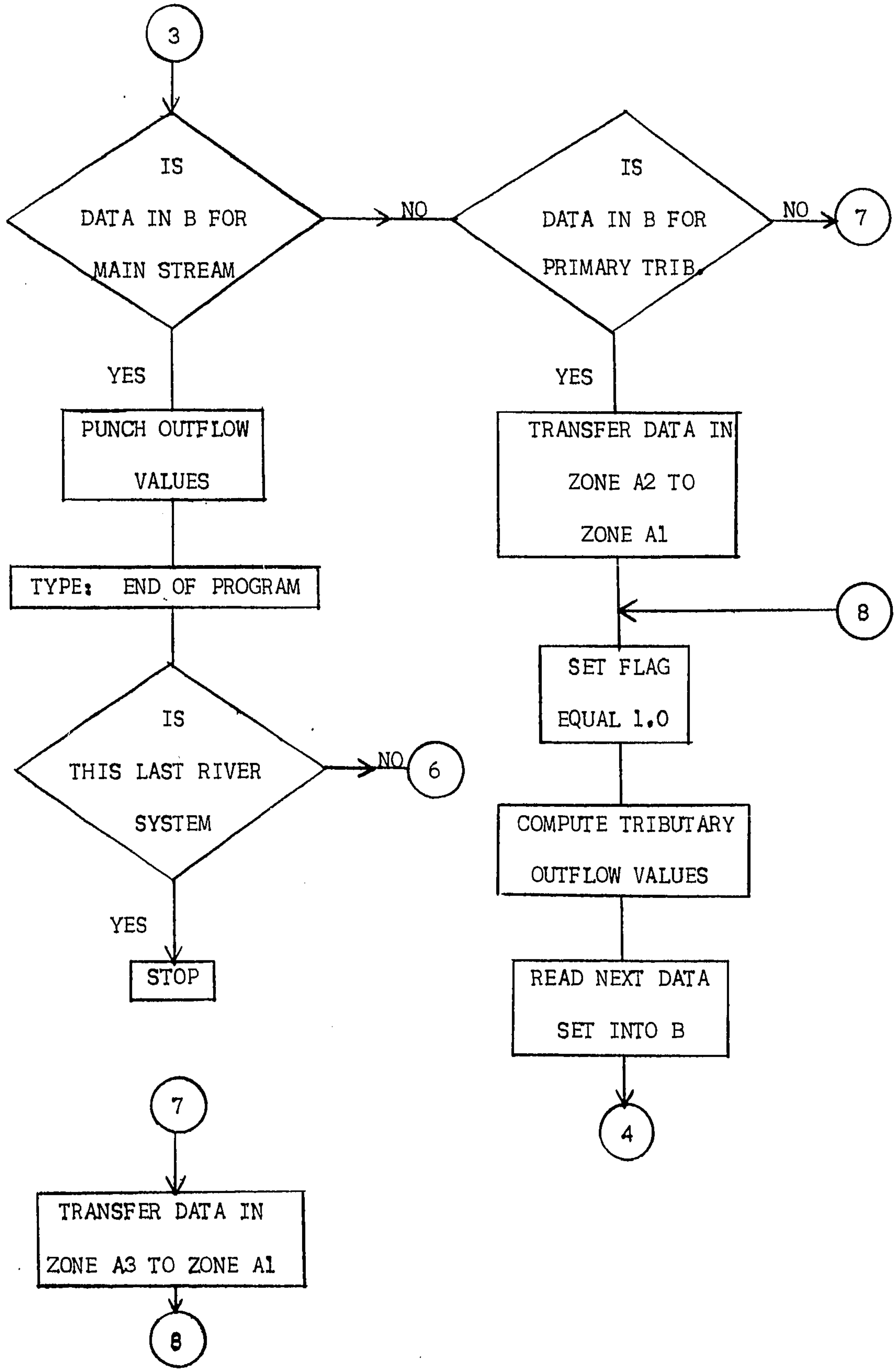
TLOAD	Total Allowable BOD for Entire Basin up to Considered Segment (lbs/day)
TOTAO	Total Available Oxygen Per Day with River at RQS (lbs/day)
TRDEF	Oxygen Deficit from Tributary (p.p.m.)
VMEAN	Mean Velocity in Segment (ft./sec.)
VOL	Weight of Water in Segment (lbs.)
XTRAO	Excess or Deficit of Reoxygenation over Oxygen Demand (lbs/day)

Flow Chart


START
6
PUNCH NAME OF PROGRAM
READ FIRST DATA SET
INTO ZONE A1
SET BASIN BOD SUM TO ZERO
5
PUNCH TRIBUTARY ORDER
i.e. MAIN STREAM, PRIMARY
OR SECONDARY TRIBUTARY
PUNCH OUTPUT
COLUMN HEADINGS
SET INITIAL VALUES
PUNCH INITIAL
CONDITION VALUES
READ NEXT DATA SET
INTO ZONE B
4
ARE
DATA SETS FOR SAME
RIVER
NO
1
YES
COMPUTE MEAN DATA
VALUES FOR SEGMENT
IF FLAG INDICATES
TRIBUTARY HAS ENTERED
MODIFY DATA VALUES
TO INCLUDE TRIBUTARY
COMPUTE OXYGEN
DEFICITS FOR BOTH
RQS METHOD AND
D.O. PLOT
2


2
PROVIDE K1 AND MODIFY FOR TIME AND TEMPERATURE
COMPUTE K2 AND MODIFY FOR TEMPERATURE
COMPUTE AVAILABLE OXYGEN FOR BOTH RQS METHOD AND D.O. PLOT
COMPUTE ALLOWABLE BOD FOR RQS METHOD
COMPUTE BOD APPLIED IN SEGMENT FOR D.O. PLOT
COMPUTE STREAM D.O. LEVEL LEAVING SEGMENT
COMPUTE SUMS AND PASSAGE TIME
PUNCH OUTPUT
IS THIS LAST DATA FOR RIVER
YES
3
NO
TRANSFER ZONE B TO ZONE A1
READ NEXT DATA SET INTO B
4


1
STORE SUMS AND NEEDED ANTECEDENT VALUES IN A1
IS DATA IN A1 FOR MAIN STREAM
NO
IS DATA IN A1 FOR PRIMARY TRIBUTARY
NO
YES
TRANSFER DATA IN ZONE A1 TO ZONE A2
YES
TRANSFER DATA IN ZONE A1 TO ZONE A3
TRANSFER DATA IN ZONE B TO A1
5
TYPE: ERROR IN CARD ARRANGEMENT
HALT


3
IS
DATA IN B FOR
MAIN STREAM
NO
IS
DATA IN B FOR
PRIMARY TRIB.
NO
7
YES
PUNCH OUTFLOW
VALUES
YES
TRANSFER DATA IN
ZONE A2 TO
ZONE A1
TYPE: END OF PROGRAM
8
SET FLAG
EQUAL 1.0
IS
THIS LAST RIVER
SYSTEM
NO
6
COMPUTE TRIBUTARY
OUTFLOW VALUES
YES
STOP
READ NEXT DATA
SET INTO B
4
7
TRANSFER DATA IN
ZONE A3 TO ZONE A1
8

Program Listing

```
C      WASTE ASSIMILATION PROGRAM
  107 DIMENSION A(51),B(17),L(15),M(5)
  100 PUNCH 1
    1 FORMAT (46H WASTE ASSIMILATION CAPACITY OF A RIVER SYSTEM)
  105 PUNCH 2
    2 FORMAT (28H PREPARED BY ROY W. HANN JR.)
C      FK=K1,SK=K2, ORDEF=INDEF,,QMEAN=MEANQ,VMEAN=MEANV
  108 TLOAD=0.0
C      READ FIRST SET OF DATA INTO ALl ZONE
  115 READ4,L(1),L(2),L(3),L(4),L(5),A(1),A(2),A(3),A(4),A(5)
    4 FORMAT (13,15,11,11,12,E12.5,E12.5,E12.5,E12.5,E12.5)
  116 READ 5,A(6),A(7),A(8),A(9),A(10),A(11)
    5 FORMAT (E12.5,E12.5,E12.5,E12.5,E12.5,E12.5)
  117 GO TO 260
  260 IF (L(5)-1) 261,265,270
  261 PUNCH 20
   20 FORMAT(12H MAIN STREAM)
  262 GO TO 119
  265 PUNCH 21
   21 FORMAT (18H PRIMARY TRIBUTARY)
  266 GO TO 119
  270 PUNCH 22
   22 FORMAT (20H SECONDARY TRIBUTARY)
  271 GO TO 119
  119 PUNCH 8
```

```
 120 PUNCH 28
   8 FORMAT(41H RN    RM    TIME    RQS      ALOAD      SLOAD)
  28 FORMAT(46X,26HTLOAD    DO2      FLOW     K2)
C      COMPUTEINITIAL SATDO,SET K1TTL,TABOD,APBOD,SLOAD,PTIME=0.0
 121 DO2=14.652-0.41*A(2)+0.008*((A(2))**2.0)-0.000078*(A(2)**3.0)
1121 DO2= DO2-A(10)
 122 FKTTL=0.0
 123 TABOD=0.0
 124 APBOD=0.0
 125 SLOAD=0.0
1125 ALOAD=0.0
1126 FKTT=0.0
 126 PTIME=0.0
 127 PUNCH7,L(1),L(2),PTIME,A(3),ALOAD,SLOAD,TLOAD,DO2,A(4)
 128 KFLAG=0
C      READ NEXT DATA
 130 READ4,M(1),M(2),M(3),M(4),M(5),B(1),B(2),B(3),B(4),B(5)
 131 READ 5,B(6),B(7),B(8),B(9),B(10),B(11)
C      TEST TO SEE IF THIS THE SAME RIVER AS THE PREVIOUS CARD
C      180 IS FOR THE SAME RIVER
 135 IF (M(1)-L(1)) 136,180,136
 136 A(12)=DO2
 137 A(13)=FKTTL
 138 A(14)=TABOD
 139 A(15)=APBOD
 140 A(16)=SLOAD
```

```
  141 A(17)=PTIME
C     135-141 SAVE SUMS AND DATA TO USE WHENDATA IN AL1 IS RECALLED
C     IF NOT SAME RIVER SEE IF RIVER IN AL1 IS MAIN, PRIM., OR SEC.
  142 IF(L(5)-1) 150,165,144
  144 PRINT 6
    6 FORMAT(45H EFFOR IN ARRANGEMENT OF DATA STATMENT NO 137)
  145 PAUSE
  146 GO TO 130
  150 DO 151 J=1,5
  151 L(J+5)=L(J)
  152 DO 153 J=1,17
  153 A(J+17)=A(J)
C     150-153 DATA IN AL1 IS FOR MAIN RIVER- TRANS TO A+17, L+5)
  155 DO 156 J=1,5
  156 L(J)=M(J)
  157 DO 158 J=1,17
  158 A(J)=B(J)
C     155-158 TRANS BM TO AL1
C     READ AND PUNCH NAME OF SECOND RIVER AND READ NEW DATA
  159 GO TO 117
  165 DO 166 J=1,5
  166 L(J+10)=L(J)
  167 DO 168 J=1,17
  168 A(J+34)=A(J)
C     165-168 DATA IN AL1 IS FOR PRIMARY TRIB.- TRANS TO A+34,L+10,
  169 GO TO 155
```

```
 180 SEGL= A(1)-B(1)
 181 ATEMP=0.5*(A(2)+B(2))
 182 RQS=A(3)
 183 QMEAN=0.5*(A(4)+B(4))
 184 ADDQ=B(4)-A(4)
 185 ADEEP=0.5*(A(6)+B(6))
 186 VMEAN=0.5*(A(4)/A(7))+0.5*(B(4)/B(7))
 187 SLOPE=A(9)
 188 ORDEF=A(10)
 189 ABOD=A(11)
C     180-189DEFINE VARIABLES NOT PREVIOUSLY ESTABLISHED
C     COMPUTE OXYGEN DEFICITE
C     FURNISH RQS IN PPM, ATEMP IN DEG C, DO2 FOR LAST SEG FOR DO PLOT
 470 SATDO=14.652-0.41*ATEMP+.008*((ATEMP)**2.)-.000078*((ATEMP)**3.)
 471 DEF1=SATDO-RQS
C     COMPUTE DEF FOR DO PLOT
 474 DEF2=SATDO-DO2
 190 IF(KFLAG- 0) 191,196,191
 191 ORDEF=TRDEF
1191 TABOD=TABOD+RBOD
 192 ABOD=ABOD+TRBOD
1192 ORDF2=SATDO-RQS
1193 DO2=A(12)
1194 FKTTL=A(13)
1195 TABOD=A(14)
1196 APBOD=A(15)
1197 SLOAD=A(16)
```

```
1198 PTIME=A(17)
1474 DEF2=SATDO-DO2
 193 KFLAG =0
 194 GO TO 480
 196 ORDF2=ORDEF
 197 GO TO 480
C     SUBROUTINE TO COMPUTE K1 FOR RIVER SYSTEM
C     FURNISHATEMP IN DEG C, MEAN V IN FT/SEC, SEGL SEG LENGTH IN MILES
C     COMPUTE TIME OF PASSAGE THROUGH SEGMENT
C     TIME IN DAYS=MILES*5280/24*3600*(MEANV-FT/SEC)
 480 TIME=0.06111*SEGL/VMEAN
C     K1 PER DAY AT 20C=.345 REPLACE NEXT CARD IF OTHER VALUE DESIRED
 485 FK20=0.345
C     HOLD PREVIOUS KITT VALUE
 486 FKTTL=FKTT
C     CORRECT FOR TIME OTHER THAN ONE DAY
 490 FKTT=((1.047)**(ATEMP-20.0))*FK20
C     CORRECT FOR TEMPERATURE
 495 FKTT=1.0-((1.0-FKTT)**TIME)
C     SUBR. TO COMPUTE K2 PER DAY BY THE OCONNER DOBBINS METHOD
C     FURNISH SLOPE IN FT/FT, ADEEP IN FT, MEANV IN FT/SEC, ATEMPIN DEG C
 500 IF(ADEEP-5.0)501,506,506
C     NON-ISOTROPIC CONDITION, K220 MEANS K2 AT 20 DEG C
 501 SK20=9.9792*((SLOPE)**.25)/((ADEEP)**1.25)
 502 GO TO 510
C     ISOTROPIC CONDITION
```

```
  506 SK20=12.96* (VMEAN**.5)/((ADEEP)**1.5)
  507  GO TO 510
C       CORRECT FOR TEMPERATURE
  510 SKT=SK20*((1.024)**(ATEMP-20.0))
C       SUBR. TO COMPUTE REAERATION IN A STREAM SEGMENT
C       FURNISH MEAN NQ IN CFS, TIME IN DAYS, DEF1 IN PPM
C       K2 PER DAY, ADD Q IN CFS, DEF2 IN PPM FOR DO PLOT
C       INDEF (INITIAL DEFICIT FOR ADDED Q)
C       COMPUTE REAERATION AS POUNDS OF OXYGEN= K2* DEF*LB H20/10**6)
C       WEIGHT OF WATER IN SEGMENT = (Q/DAY)(DAYS)(WT/CF)/10**6
  520 VOL=5.39136*QMEAN*TIME
  521 IF(ADDQ-0.0) 522,525,525
  522 ADD Q=0.0
C       COMPUTE REAERATION FOR RQS
  525 REO2=SKT*DEF1*VOL
C       COMPUTE REAERATION FOR DO PLOT
  526 REO2P =SKT*DEF2*VOL
C       COMPUTE ADDED 02
  530 ADDV=5.39136*ADDQ
C       ADD 02 IS FUNCTION OF ADDED Q/DAY*DEF
  535 ADO21 =ADDV*(DEF1-ORDF2)
  536 ADO22 =ADDV*(DEF2-ORDEF)
C       COMPUTE TOTAL AVAILABLE OXYGEN IN SEGMENT PER DAY
  540 TOTAO=REO2+ADO21
C       COMPUTE TOTAL AVAILABLE OXYGEN FOR DO PLOT
  541 TLAOP=REO2P+ADO22
```

```
C     THIS IS A SUBR. TO COMPUTE ALLOWABLE BOD LOAD PER SEGMENT
C     AND ALSO AVAILABLE O2 VS APPLIED BOD FOR DO PLOT
C     FURNISH TOTAO IN LB/SEG/DAY,KITT/DAY/SEG,TABOD FOR LAST SEGMENT
C     K1 FOR LAST SEG, APBOD (APPLIED BOD FOR LAST SEG)
C     ADDED BOD IN LB/DAY (ABOD)TLAOP IN LB/DAY FOR DO PLOT
C     COMPUTE HOLDOVER BOD
  600 HOBOD=TABOD*(1.0-FKTTL)
C     COMPUTE TOTAL BOD ALLOWED
  605 TABOD=TOTAO/FKTT
C     COMPUTE ALLOWABLE LOAD
  610 ALOAD=TABOD-HOBOD
C     COMPUTE RESIDUAL APPLIED BOD FOR DO PLOT
  615 RABOD=APBOD*(1.0-FKTTL)
C     COMPUTE TOTAL BOD APPLIED TO SEGMENT
  620 APBOD=RABOD+ABOD
C     COMPUTE EFFECTIVE BOD APPLIED TO SEGMENT
  622 EFBOD=FKTT*APBOD
C     COMPUTE OXYGEN EXCESS OR DEFICIT
  625 XTRAO=TLAOP-EFBOD
C     SUBR. TO COMPUTE CHANGE IN STREAM DO LEVEL
C     APPLICABLE FOR DO PLOT
C     FURNISH MEANQ IN CFS, DO2 INPPM, XTRAO IN LB/DAY
C     COMPUTE CHANGE IN DO IN LB/MILLION LB=PPM
  630 ADDO=XTRAO/(5.39136*QMEAN)
C     REVISE DO2
  635 DO2=DO2+ADDO
```

```
  636 IF (DO2-0.0) 637,638,638
  637 DO2=0.0
  638 GO TO 200
  200 PTIME=PTIME+TIME
  201 SLOAD=SLOAD+ALOAD
  202 TLOAD=TLOAD+ALOAD
  203 Q=QMEAN
C     ALL SUBR. FINISHED PUNCH RESULTS
C     ARRANGE FOR EXPONENT OUTPUT IF NEEDED
  205 IF(SLOAD-100000000.00) 206,210,210
  206 PUNCH 7,M(1),M(2),PTIME,RQS,ALOAD,SLOAD,TLOAD,DO2,Q,SKT
    7 FORMAT (I3,I5,F7.2,F6.2,F10.1,F10.1,E10.3,F6.2,F9.1,F6.2)
  207 GO TO 216
  210 IF(ALOAD-100000000.00) 211,215,215
  211 PUNCH 9,M(1),M(2),PTIME,RQS,ALOAD,SLOAD,TLOAD,DO2,Q,SKT
    9 FORMAT(I3,I5,F7.2,F6.2,F10.1,E10.4,E10.3,F6.2,F9.1,F6.2)
  212 GO TO 216
  215 PUNCH10,M(1),M(2),PTIME,RQS,ALOAD,SLOAD,TLOAD,DO2,Q,SKT
   10 FORMAT(I3,I5,F7.2,F6.2,E10.4,E10.4,E10.3,F6.2,F9.1,F6.2)
C     IF NOT LAST CARD IN RIVER TRANSFER B TO AL1 AND READ NEXT CARD
C     SIDETRACTED DATA TO A- 225PRIMARY, 240 SECONDARY
C     IF (M(2)-0) ERROR, END, NOT END
  216 IF(M(2)-0) 224,224,217
  217 DO218J=1,5
  218 L(J)=M(J)
  219 DO220 J=1,11
```

```
  220 A(J)=B(J)
  221 GO TO 130
  224 IF (M(5)-1)300,225,240
  225 DO 226 J=1,5
  226 L(J)=L(J+5)
  227 DO 228 J=1,17
  228 A(J)=A(J+17)
C     READ NEXT CARD
  230 GO TO 250
  240 DO 241 J=1,5
  241 L(J)=L(J+10)
  242 DO 243 J=1,17
  243 A(J)=A(J+34)
C     READ NEXT CARD    COMPUTE TRIBUTARY LOAD ON LARGER STREAM
  254 GO TO 250
  250 KFLAG = 1
C     COMPUTE O2 DEFICITE AND RESIDUAL BOD FOR TRIBUTARY
  251 TRBOD=APBOD*(1.0-FKTT)
 1251 RBOD=TABOD
  252 TRDEF=SATDO-DO2
  253 PUNCH 8
  254 PUNCH 28
  255 GO TO 130
  300 TRBOD=APBOD*(1.0-FKTT)
  301 TRDEF=SATDO-DO2
  302 PUNCH40,TRBOD,TRDEF,TABOD,FKTT
```

```
 40 FORMAT(5HTRBOD,E10.3,5HTRDEF,E10.3,5HTABOD,E10.3,4HFKTT,E10.3)
303 TYPE 11
 11 FORMAT(15H END OF PROGRAM)
304 PUNCH 11
305 STOP
306 END
```

Program Three

Tidal Zone Waste Assimilation Program

The purpose of this program is to compute the waste assimilation capacity of the tidal zone of a single river. This program does not provide for a plot of the dissolved oxygen profile.

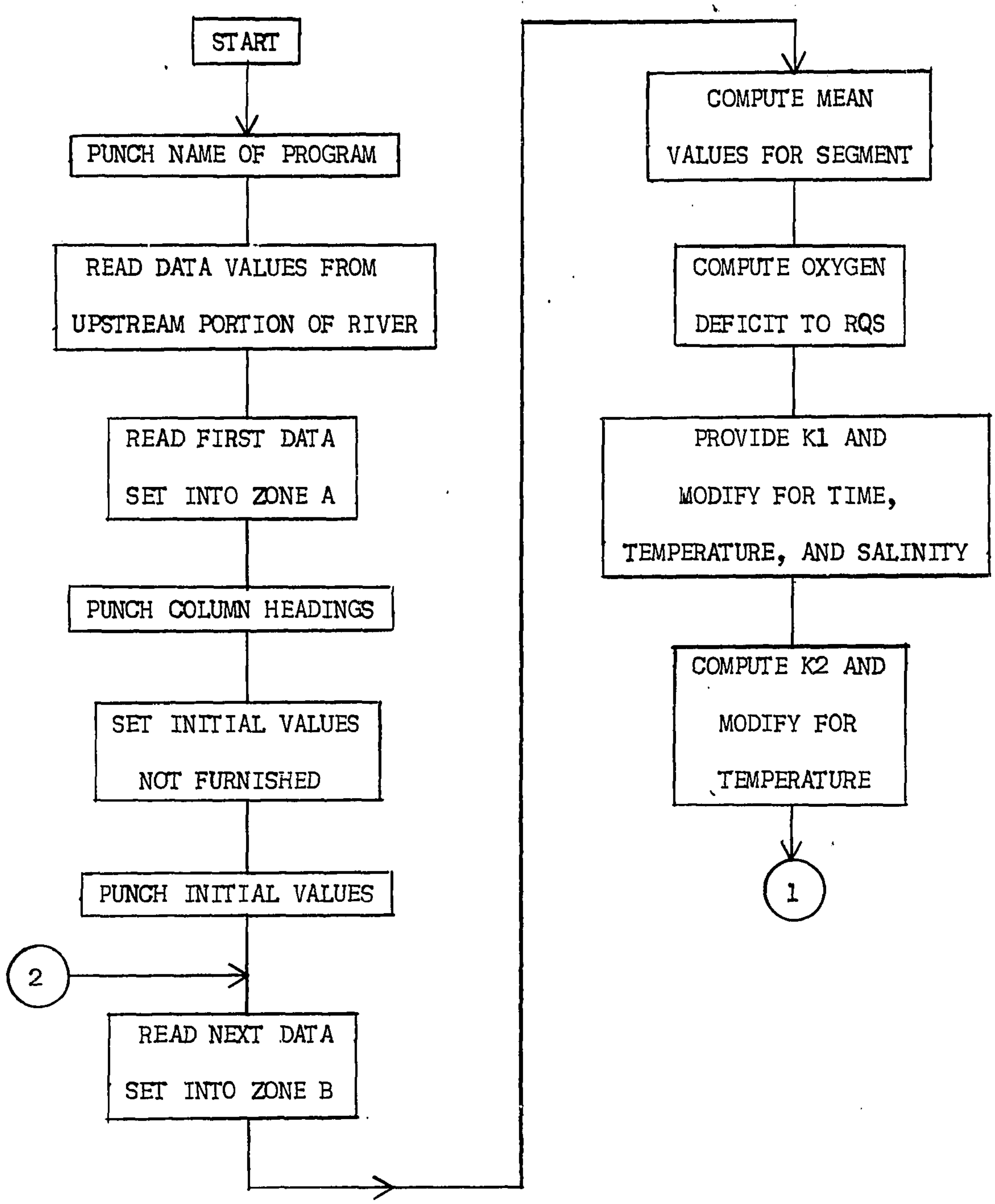
Program Notation Key

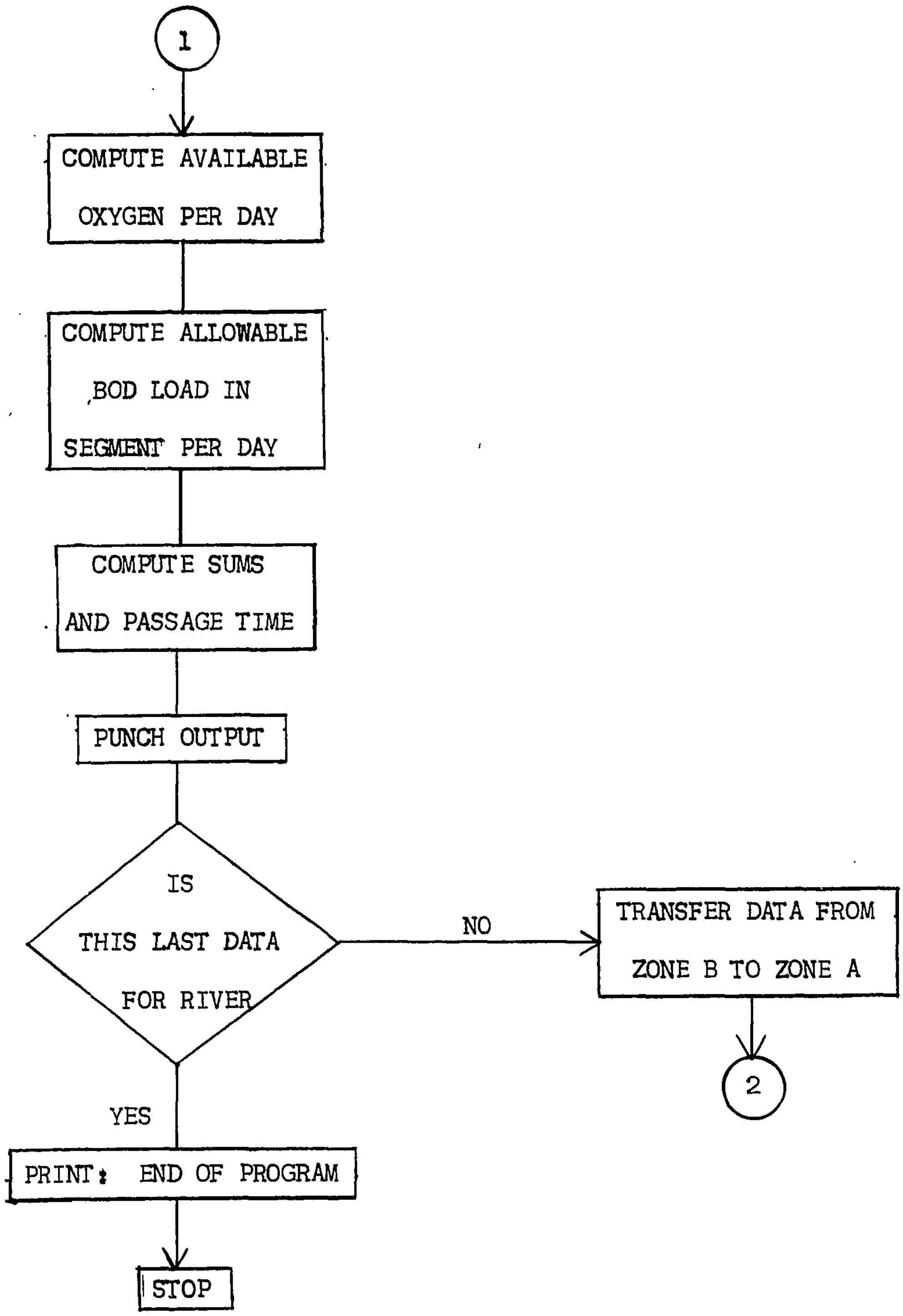
Zone A	Zone B	
L(1)	M(1)	River Number
L(2)	M(2)	River Mile
L(3)	M(3)	Complete Data Flag
A(1)	B(1)	River Mile
A(2)	B(2)	Temperature (Degrees C.)
A(3)	B(3)	River Quality Standard (p.p.m.)
A(4)	B(4)	Fresh Water Flow (c.f.s.)
A(5)	B(5)	Added Flow (c.f.s.)
A(6)	B(6)	Oxygen Deficit of Inflow Water (p.p.m.)
A(7)	B(7)	Mean Depth (ft.)
A(8)	B(8)	Mean Cross Section Area (sq. ft.)
A(9)	B(9)	Ebb Current (ft./sec.)
A(10)	B(10)	Time of Ebb (hours)
A(11)	B(11)	Flood Current (ft./sec.)
A(12)	B(12)	Time of Flood (hours)
A(13)	B(13)	Salinity (parts per thousand)
ADDQ	Fresh Water Flow Added in Segment (c.f.s.)	

ADDV	Weight of Added Flow Per Day (lbs/day)
ADEEP	Mean Depth (ft.)
ADO21	Total Available Oxygen in Segment Per Day (lbs/day)
ALOAD	Allowable BOD Load Which Could be Added to Segment (lbs/day)
ATEMP	Average Temperature in Segment (Degrees C.)
DEF1	Oxygen Deficit to RQS (p.p.m.)
DO2	Dummy Value Not Used in Program
EBBC	Ebb Current (ft./sec.)
EBBT	Duration of Ebb Current (hours)
FKTT	K1 Value Adjusted for Time, Temperature, and Salinity
FKTTL	K1 for Last Segment
FK20	K1 at 20 Degrees Centigrade
FK20T	K1 at 20 Degrees Centigrade Modified for Time
FLODC	Flood Current (ft./sec.)
FLODT	Duration of Flood Current (hours)
HOBOD	Residual Allowable BOD From Previous Segment (lbs/day)
PTIME	Passage Time of Fresh Water without Tide Action (days)
Q	Mean Flow
QMEAN	Mean Fresh Water Flow (c.f.s.)
REO2	Reoxygenation in Segment at RQS (lbs/day)
RQS	River Quality Standard for Dissolved Oxygen (p.p.m.)
SALT	Mean Salinity (parts per thousand)
SATDO	Saturation Dissolved Oxygen for given Temperature and Salinity (p.p.m.)
SEGL	Length of Segment (miles)
SKT	K2 at given Temperature and Salinity

SK20	K2 at 20 Degrees Centigrade
SLOAD	Total Allowable BOD on Considered River (lbs/day)
TABOD	Total BOD Load Allowed in Segment (lbs/day)
TIME	Mean Time in Segment (days)
TLOAD	Total Allowable BOD in River System (lbs/day)
VMEAN	Mean Effective Current Velocity for Reaeration Capacity Determination (ft./sec.)
VOL	Weight of Water in Segment (lbs.)

Flow Chart


START
PUNCH NAME OF PROGRAM
READ DATA VALUES FROM UPSTREAM PORTION OF RIVER
READ FIRST DATA SET INTO ZONE A
PUNCH COLUMN HEADINGS
SET INITIAL VALUES NOT FURNISHED
PUNCH INITIAL VALUES
2
READ NEXT DATA SET INTO ZONE B
COMPUTE MEAN VALUES FOR SEGMENT
COMPUTE OXYGEN DEFICIT TO RQS
PROVIDE K1 AND MODIFY FOR TIME, TEMPERATURE, AND SALINITY
COMPUTE K2 AND MODIFY FOR TEMPERATURE
1


1
COMPUTE AVAILABLE OXYGEN PER DAY
COMPUTE ALLOWABLE BOD LOAD IN SEGMENT PER DAY
COMPUTE SUMS AND PASSAGE TIME
PUNCH OUTPUT
IS THIS LAST DATA FOR RIVER
NO
TRANSFER DATA FROM ZONE B TO ZONE A
2
YES
PRINT: END OF PROGRAM
STOP

Program Listing

```
C     WASTE ASSIMILATION PROGRAM -- RIVER WITH TIDES
C     READ LOADS FROM UPSTREAM PORTION
   80 READ 81, TABOD, FKTT, SLOAD, TLOAD
   81 FORMAT (E10.3, E10.3, E10.3, E10.3)
   99 DIMENSION A(20), B(20), L(3), M(3)
  100 PUNCH 1
    1 FORMAT(38HTIDAL ZONE WASTE ASSIMILATION CAPACITY)
  105 PUNCH 2
    2 FORMAT(28H PREPARED BY ROY W. HANN JR.)
C     READ DATA
  115 READ4,L(1),L(2),L(3),A(1),A(2),A(3),A(4),A(5),A(6)
    4 FORMAT(I3,I4,I1,E10.3,E10.3,E10.3,E10.3,E10.3,E10.3)
  116 READ5,A(7),A(8),A(9),A(10),A(11),A(12),A(13)
    5 FORMAT(E10.3,E10.3,E10.3,E10.3,E10.3,E10.3,E10.3)
C     PUNCH HEADINGS
  119 PUNCH 8
  120 PUNCH 28
    8 FORMAT(41H RN    RM    TIME    RQS      ALOAD      SLOAD)
   28 FORMAT(46X,26HTLOAD    DO2      FLOW     K2)
  121 DO2=0.0
 1125 ALOAD=0.0
  126 PTIME=0.0
  127 PUNCH7,L(1),L(2),PTIME,A(3),ALOAD,SLOAD,TLOAD,DO2,A(4)
C     READ NEXT DATA
```

```
  130 READ4,M(1),M(2),M(3),B(1),B(2),B(3),B(4),B(5),B(6)
  131 READ5,B(7),B(8),B(9),B(10),B(11),B(12),B(13)
C     DEFINE AND MODIFY DATA
  180 SEGL= A(1)-B(1)
  181 ATEMP=0.5*(A(2)+B(2))
  182 RQS=A(3)
  183 QMEAN=0.5*(A(4)+B(4))
  184 ADDQ=B(4)-A(4)
 1184 ORDF2=B(6)
 1185 AREA=0.5*(A(8)+B(8))
  185 ADEEP=0.5*(A(7)+B(7))
  186 EBBC=0.5*(A(9)+B(9))
  187 EBBT=0.5*(A(10)+B(10))
  188 FLODC=0.5*(A(11)+B(11))
  189 FLODT=0.5*(A(12)+B(12))
  190 SALT=0.5*(A(13)+B(13))
 1190 VNET=QMEAN/AREA
  191 VMEAN=(EBBT/(EBBT+FLODT))*((.583*EBBC)**0.5)
  192 VMEAN=VMEAN+(FLODT/(EBBT+FLODT))*((.583*FLODC)**0.5)
  193 VMEAN=VMEAN**2.0
  470 SATDO=14.652-0.41*ATEMP+.008*((ATEMP)**2.)-.000078*((ATEMP)**3.)
 1470 SATDO=SATDO-SALT*(0.0841-.00256*ATEMP+.0000374*((ATEMP)**2.0))
  471 DEF1+SATDO-RQS
  480 TIME=0.06111*SEGL/VNET
  485 FK20= 0.345
  486 FKTTL=FKTT
```

```
  495 FKTT=((1.047)**(ATEMP-20.0))*FK20
C     CORRECT FOR SALINITY
 1495 SALT2=SALT
 1496 SALT= 1.81*SALT
  496 IF(SALT-6.0)1497,1497,497
 1497 IF(SALT-3.0)1498,497,497
 1498 SALT=6.0-SALT
  497 FKTT=FKTT*(1.111-.01233*((SALT-3.0)**2.0))
  498 GO TO 506
  499 FKTT=FKTT*(1.0-(.021*(SALT-6.0)))
  500 FKTT= 1.0-((1.0-FKTT)**TIME)
C     EVALUATE K2 AND MODIFY
  506 SK20=12.96*( VMEAN**.5)/((ADEEP)**1.5)
  510 SKT=SK20*((1.024)**(ATEMP-20.0))
 1510 SALT=SALT2
 2100 REDUC= 1.0-0.059*((SALT)**0.5)
 2101 SKT=SKT*REDUC
C     COMPUTE AVAILABLE OXYGEN
  520 VOL=5.39136*QMEAN*TIME
  521 IF(ADDQ-0.0)522,525,525
  522 ADDQ= 0.0
  525 RE02=SKT*DEF1*VOL
  530 ADDV=5.39136*ADDQ
  535 AD021= ADDV*(DEF1-ORDF2)
  540 TOTAO=RE02+AD021
  600 HOBOD=TABOD*(1.0-FKTTL)
```

```
  605 TABOD=TOTAO/FKTT
C     COMPUTE ALLOWABLE BOD LOAD
  610 ALOAD=TABOD-HOBOD
C     COMPUTE SUMS
  200 PTIME= PTIME+TIME
  201 SLOAD=SLOAD+ALOAD
  202 TLOAD=TLOAD+ALOAD
  203 Q=QMEAN
C     PUNCH OUTPUT
  205 IF(SLOAD-100000000.00)206,210,210
  206 PUNCH7,M(1),M(2),PTIME,RQS,ALOAD,SLOAD,TLOAD,DO2,Q    ,SKT
    7 FORMAT(I3,I5,F7.2,F6.2,F10.1,F10.1,E10.3,F6.2,F9.1,F6.2)
  207 GO TO 216
  210 IF(ALOAD-100000000.00)211,215,215
  211 PUNCH9,M(1),M(2),PTIME,RQS,ALOAD,SLOAD,TLOAD,DO2,Q    SKT
    9 FORMAT(I3,I5,F7.2,F6.2,F10.1,E10.4,E10.3,F6.2,F9.1,F6.2)
  212 GO TO 216
  215 PUNCH10,M(1),M(2),PTIME,RQS,ALOAD,SLOAD,TLOAD,DO2,Q    SKT
   10 FORMAT(I3,I5,F7.2,F6.2,E10.4,E10.4,E10.3,F6.2,F9.1,F6.2)
  216 IF(M(2)-M0Z(I)) 224,224,217
  217 DO 218 J=1,3
  218 L(J)=M(J)
  219 DO 220 J=1,13
  220 A(J)=B(J)
  221 GO TO 130
  224 GO TO 303
```

```
C 303 TYPE 11
   11 FORMAT(15END OF PROGRAM  )
  304 PUNCH 11
  305 STOP
  306 END
```

Program Four

Cubature Program

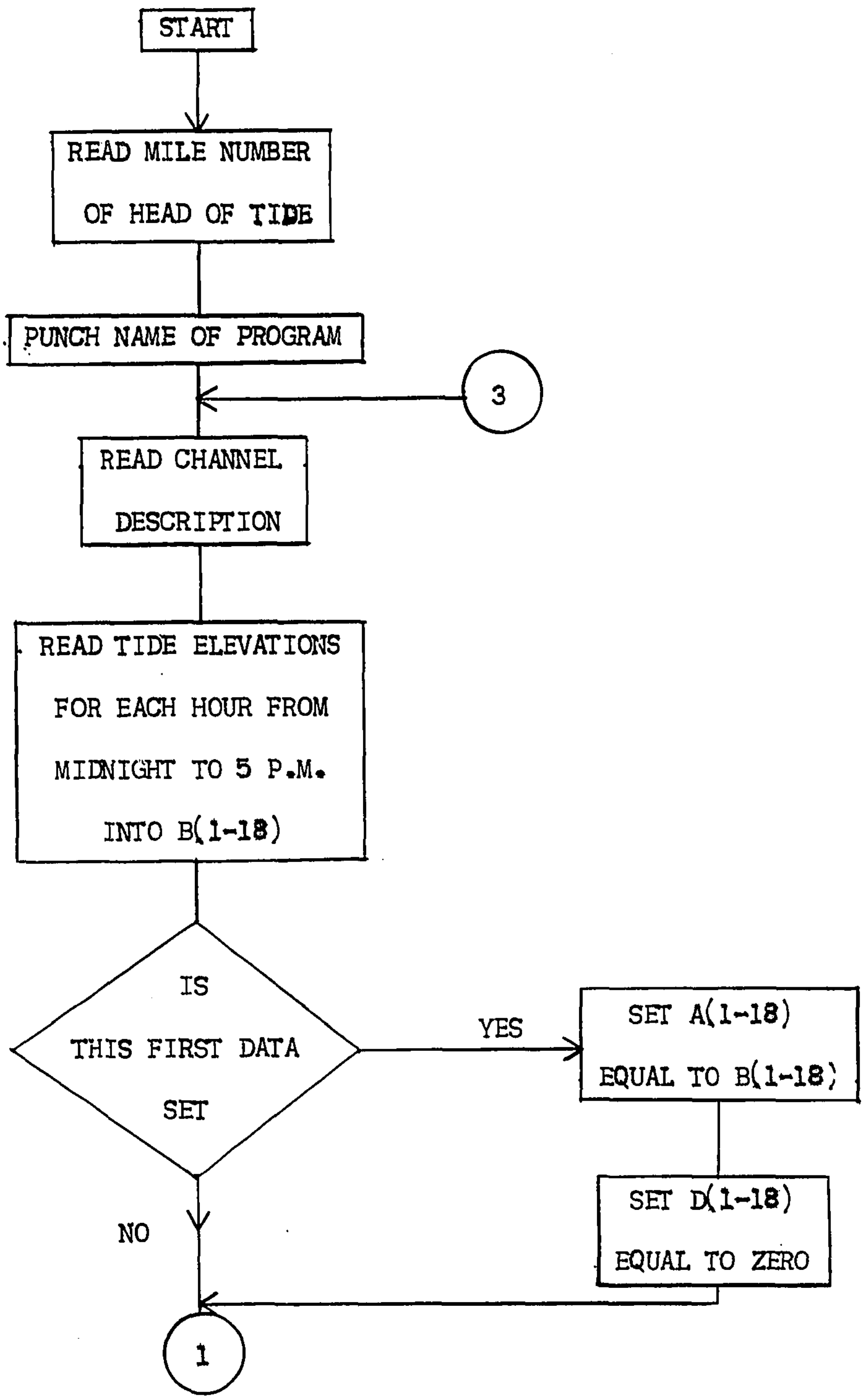
The purpose of this program is to compute the mean current velocities for each hour of the tidal cycle at cross sections of the tidal zone of a river.

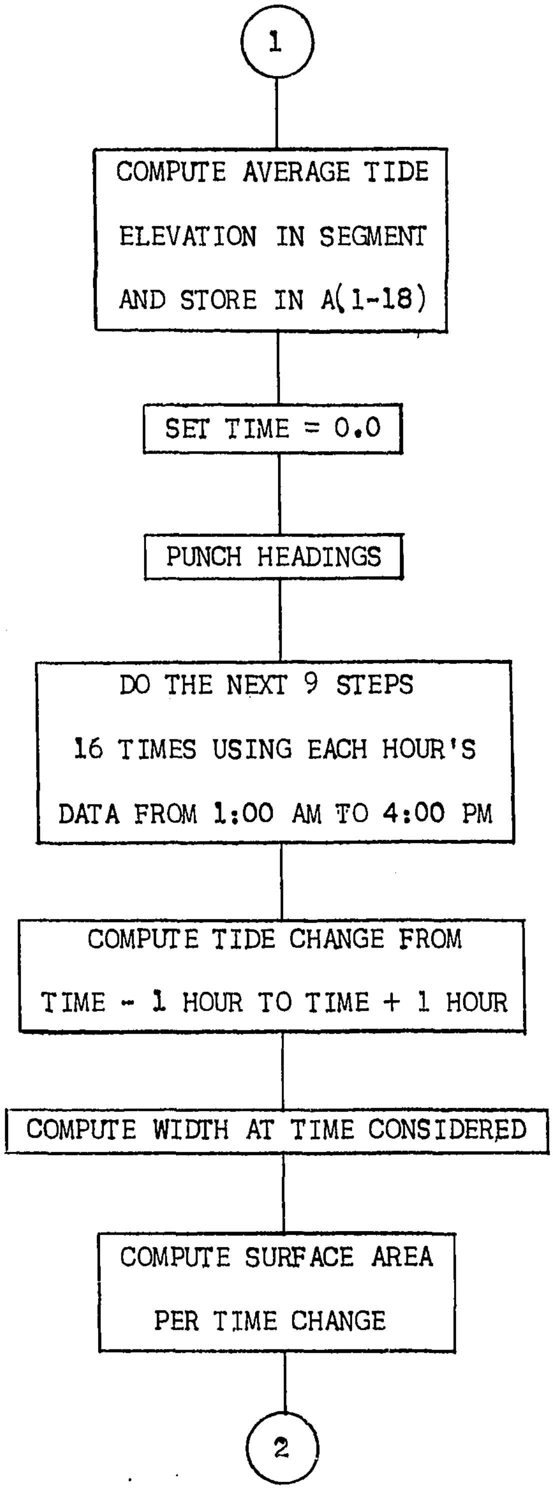
Program Notation Key

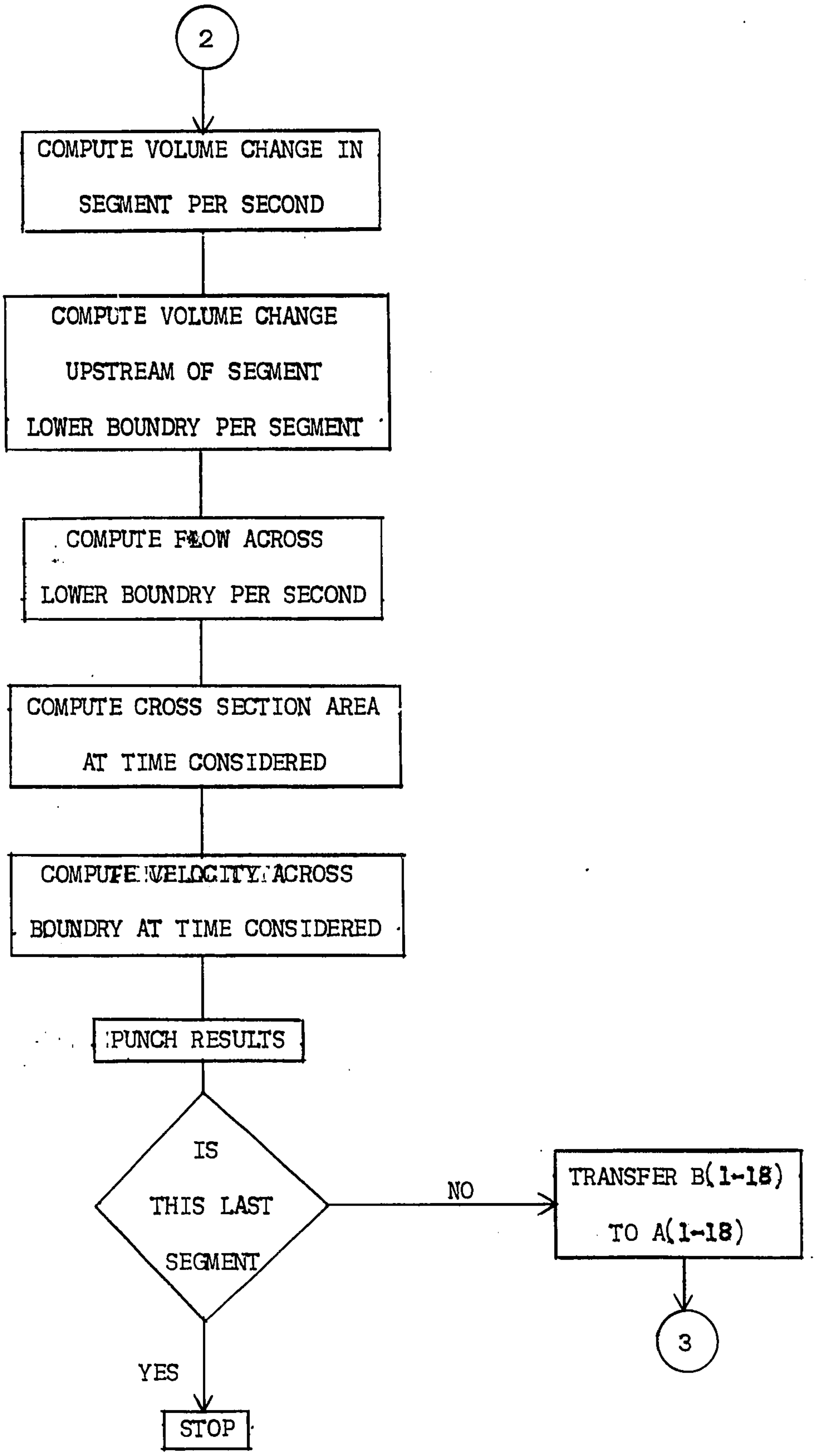
A(1) to A(18)	Mean Tide Elevation in Segment for Each Hour from Midnight to 5 P.M.
B(1) to B(18)	Tide Elevation at Downstream End of Segment for Each Hour from Midnight to 5 P.M.
D(2) to D(17)	Tide Change in Segment During Period from One Hour Before to One Hour After Considered Time
AREAL	Area of Cross Section at Low Tide (sq. ft.)
DELTQ	Volume Change in Segment Per Second (cu. ft.)
ELEVH	Elevation of High Tide (ft.)
ELEVL	Elevation of Low Tide (ft.)
RFLOW	River Flow (c.f.s.)
RM	River Mile
SDQ	Summation of Volume Change Per Second above Downstream End of Segment (cu. ft.)
TIDE	Mean Tide Elevation during Considered Time (ft.)
TIDEC	Tide Change in Time Period (ft.)
U	Surface Area per Time Period
V	Velocity through Cross Section during Time Considered (ft./sec.)

W	River Width at Time Considered (ft./sec.)
WIDEH	River Width at High Tide (ft.)
WIDEL	River Width at Low Tide (ft.)
X	Cross Section Area at Time Considered (sq. ft.)

Flow Chart


START
READ MILE NUMBER OF HEAD OF TIDE
PUNCH NAME OF PROGRAM
3
READ CHANNEL DESCRIPTION
READ TIDE ELEVATIONS FOR EACH HOUR FROM MIDNIGHT TO 5 P.M. INTO B(1-18)
IS THIS FIRST DATA SET
YES
SET A(1-18) EQUAL TO B(1-18)
SET D(1-18) EQUAL TO ZERO
NO
1


1
COMPUTE AVERAGE TIDE ELEVATION IN SEGMENT AND STORE IN A(1-18)
SET TIME = 0.0
PUNCH HEADINGS
DO THE NEXT 9 STEPS 16 TIMES USING EACH HOUR'S DATA FROM 1:00 AM TO 4:00 PM
COMPUTE TIDE CHANGE FROM TIME - 1 HOUR TO TIME + 1 HOUR
COMPUTE WIDTH AT TIME CONSIDERED
COMPUTE SURFACE AREA PER TIME CHANGE
2


2
COMPUTE VOLUME CHANGE IN SEGMENT PER SECOND
COMPUTE VOLUME CHANGE UPSTREAM OF SEGMENT LOWER BOUNDRY PER SEGMENT
COMPUTE FLOW ACROSS LOWER BOUNDRY PER SECOND
COMPUTE CROSS SECTION AREA AT TIME CONSIDERED
COMPUTE VELOCITY ACROSS BOUNDRY AT TIME CONSIDERED
PUNCH RESULTS
IS THIS LAST SEGMENT
NO
TRANSFER B(1-18) TO A(1-18)
3
YES
STOP

Program Listing

```
C     CUBATURE PROGRAM
   11 PUNCH 1
    1 FORMAT(42HCUBATURE PROGRAM PREPARED BY ROY W HANN JR)
   10 DIMENSION A(18),B(18),D(18)
  110 READ 9,PRM
    9 FORMAT(E10.3)
   13 KFLAG=1
   12 READ3,RM,ELEVL,ELEVH,WIDEL,WIDEH,RFLOW,AREAL
    3 FORMAT(E10.3,E10.3,E10.3,E10.3,E10.3,E10.3,E10.3)
   20 READ2,B(1),B(2),B(3),B(4),B(5),B(6),B(7),B(8),B(9)
   21 READ2,B(10),B(11),B(12),B(13),B(14),B(15),B(16),B(17),B(18)
    2 FORMAT(F6.2,F6.2,F6.2,F6.2,F6.2,F6.2,F6.2,F6.2,F6.2)
   22 SEGL=(RM-PRM)*5280.0
   31 IF(KFLAG-1)40,32,40
C     FLAG TO TELL IF FIRST DATA
   32 KFLAG=0
   33 DO 34 I=1,18
   34 A(I)=B(I)
  134 DO 35 I=1,18
   35 D(I)=0.0
C     CHANGE A(I)  TO MEAN VALUE
   40 DO 41 I=1,18
   41 A(I)=(A(I)+B(I))/2.0
   42 TIME = 0.0
```

```
1142 PUNCH 7
1143 PUNCH 8
   7 FORMAT(36H     RM    TIME    TIDE TIDE CHG         V)
   8 FORMAT36X,36H    DELTA Q         Q XSEC AREA      V)
  43 DO 68 I=2,17
  44 TIDEC=A(I-1)-A(I+1)
  45 TIME=TIME+1.0
  46 W=(((A(I)-ELEVL)/(ELEVH-ELEVL))*(WIDEH-WIDEL))+WIDEL
  47 U=W*SEGL/7200.0
  48 DELTQ=TIDEC*U
  49 D(I)=D(I)+DELTQ
  50 SDQ=D(I)
  60 Q=RFLOW+SDQ
  65 X=((A(I)-ELEVL)*((W+WIDEL)/2.0))+AREAL
  66 V=Q/X
  67 TIDE=A(I)
   5 FORMAT(F6.1,F6.1,F7.2,F7.2,F10.0,F10.0,F10.0,F10.0,F6.2)
  68 PUNCH5,RM,TIME,TIDE,TIDEC,U,DELTQ,Q,X,V
  69 PRM=RM
  70 DO 71 I=1,18
  71 A(I)=B(I)
  72 IF(RM-0.0)100,100,12
 100 PRINT 6
   6 FORMAT(14HEND OF PROGRAM)
 101 STOP
 102 END
```

Program Five

Point Load Versus Uniform

5a. Iteration on DA, DC, K2

5b. Iteration on Flow

The Purpose of these programs is to evaluate the reduction which must be made to the uniform allowable load if BOD loads are applied as point loads.

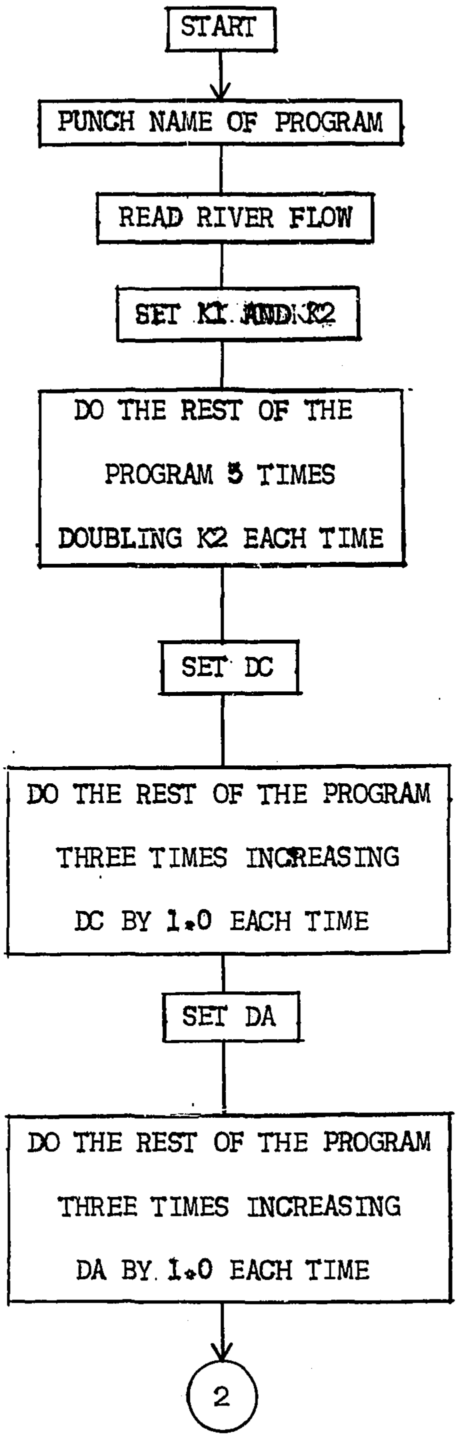
Program Notation Key

A,B,C	Convenience Names for Answers to Portions of Equations
ALOAD	Allowable Uniform BOD Load for Segment of Length TSEGB (lbs/day)
BOD1	Modified Point Load Value Using BOD2 as Previous Value (lbs/day)
BOD2	Initial Guess of Point Load Which May be Applied (lbs/day)
DA	Initial Oxygen Deficit (p.p.m.)
DAT	Initial Deficit (lbs/day)
DC	Oxygen Deficit at Critical Point (p.p.m.)
DFT	Dissolved Oxygen Deficit as Function of Time (lbs/day)
EFF	Ratio of Point Load to Uniform Load
F	Ratio of $\frac{K2}{K1}$
FKT	K1
Q	River Flow (c.f.s.)
RATIO	Ratio of Initial Guess and Next Guess is Trial and Error Solution

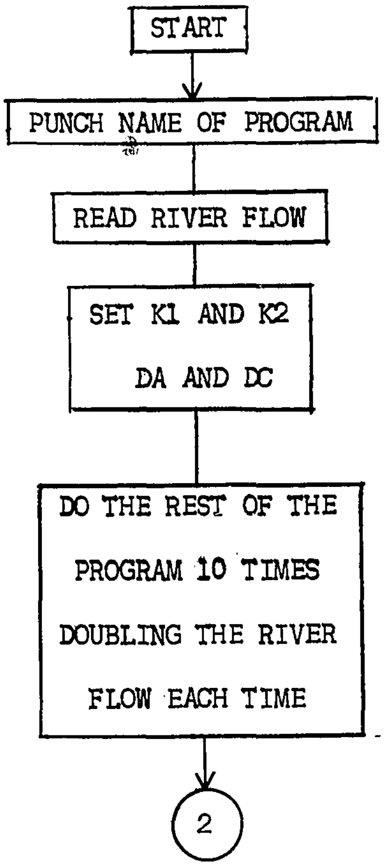
SKT	K2
TC	Time to Critical Point (days)
TSEGA	Value Less Than True River Recovery Time (days)
TSEGB	Value Midway Between TSEGA and TSEGC (days)
TSEGC	Value Greater Than True River Recovery Time (days)

Flow Chart

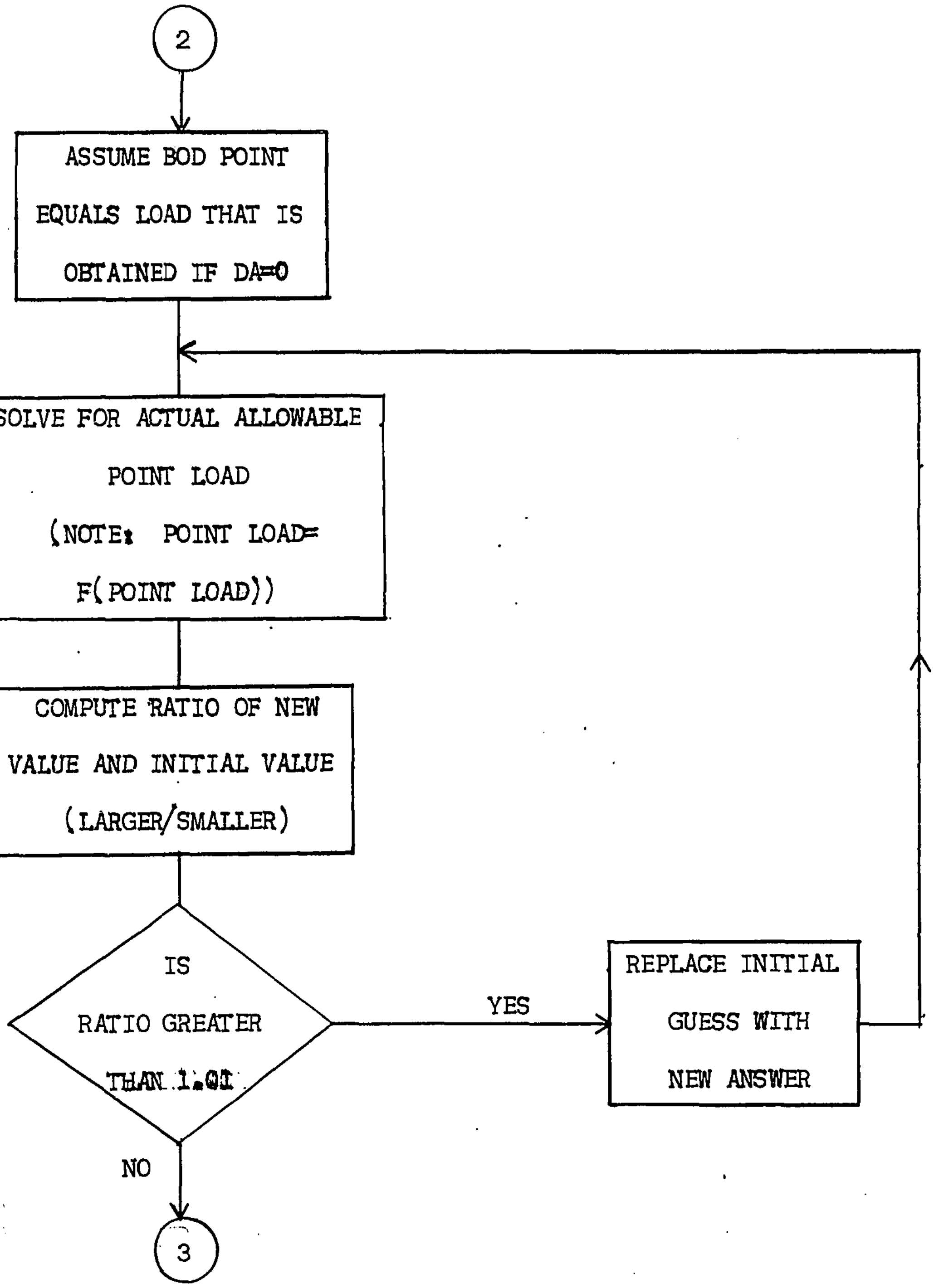
Program 5a



Program 5b



NOTE: Program 5a and 5b are identical from this point.



3

NOTE: POINT LOAD IN NOW DETERMINED TO ONE PERCENT ACCURACY

SOLVE FOR TIME TO CRITICAL POINT (TC)

LET TSEGA=TC

DOUBLE TSEGA UNTIL A TIME VALUE IS FOUND WHICH YEILDS AN OXYGEN DEFICIT LESS THAN DA.
THAT VALUE IS CALLED TSEGC AND THE PREVIOUS MULTIPLE OF THE ORIGINAL TSEGA REMAINS TSEGA

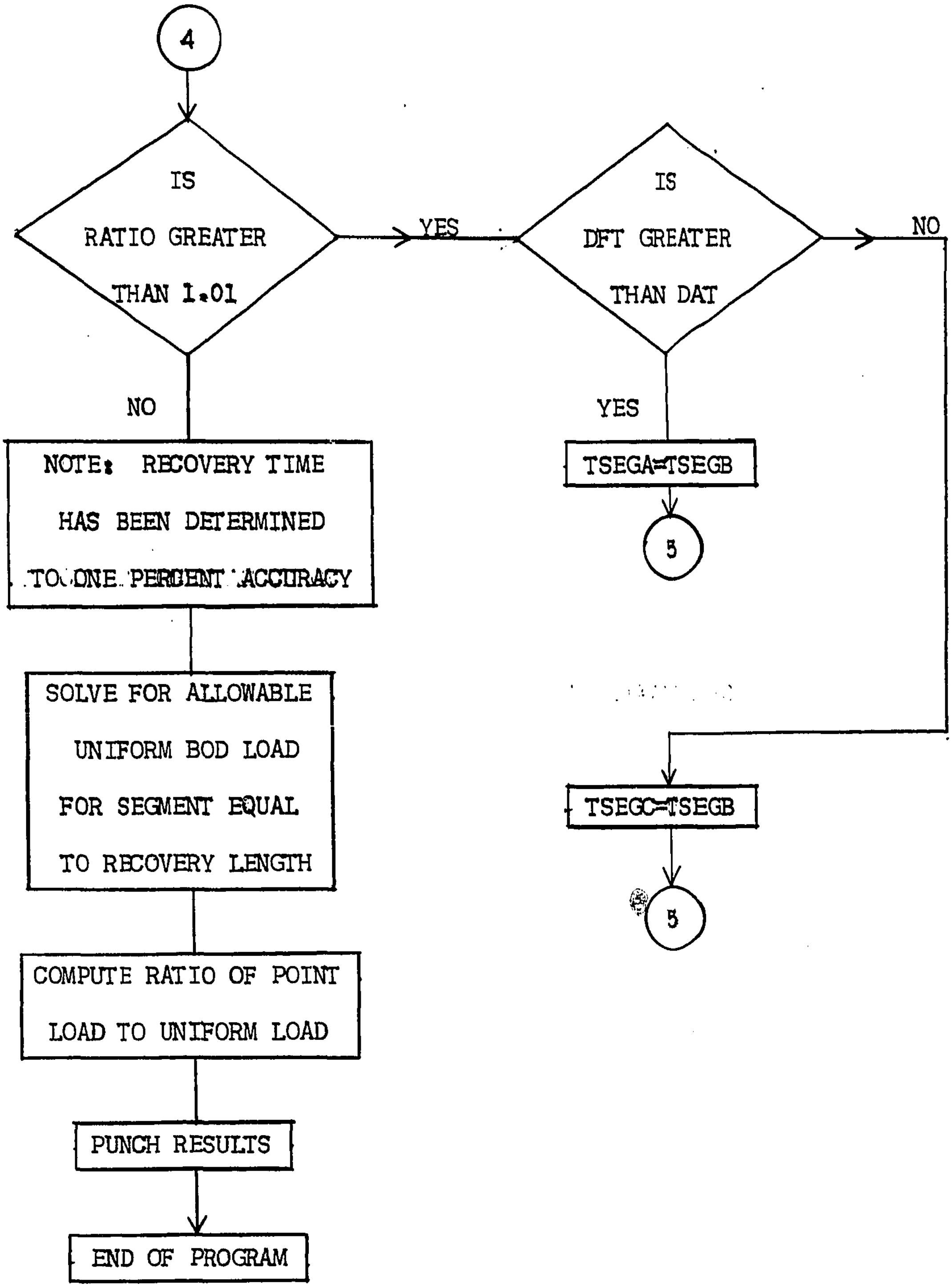
5

SET $TSEGB=\frac{TSEGA+TSEGB}{2}$

SOLVE FOR DFT

FIND RATIO OF DFT AND DAT (LARGER/SMALLER)

4


4
IS
RATIO GREATER
THAN 1.01
YES
IS
DFT GREATER
THAN DAT
NO
NO
YES
NOTE: RECOVERY TIME
HAS BEEN DETERMINED
TO ONE PERCENT ACCURACY
TSEGA=TSEGB
5
SOLVE FOR ALLOWABLE
UNIFORM BOD LOAD
FOR SEGMENT EQUAL
TO RECOVERY LENGTH
TSEGC=TSEGB
5
COMPUTE RATIO OF POINT
LOAD TO UNIFORM LOAD
PUNCH RESULTS
END OF PROGRAM

Program 5a Listing

```
100 PUNCH 1
  1 FORMAT(22H POINT LOAD COMPARISON)
101 PUNCH 2
  2 FORMAT(26H PREPARED BY ROY W HANN JR)
110 READ 10,Q
 10 FORMAT(E12.5)
111 FKT=.435
  3 FORMAT(I3)
124 SKT=0.025
C      ITERATE ON K2
130 DO 506 K=11,14
131 KA=K
132 PRINT 3,KA
133 SKT=2.0*SKT
134 DC=2.2
C      ITERATE ON DC
140 DO 506 L=21,23
141 LA=L
142 PRINT 3,LA
143 DC=DC+1.0
144 DA=(-0.5)
C      ITERATE ON DA
150 DO 506 M=31,33
151 MA=M
152 DA=DA+1.0
```

```
153 PRINT 3,MA
135 PUNCH 4
  4 FORMAT(38H      FLOW    K2   DC   DA POINT LOAD)
136 PUNCH 5
  5 FORMAT(38X,34HUNIFORM LOAD   EFF  SEG TIME RATIO)
280 DAT=5.39134*Q*DA
282 F=SKT/FKT
283 TC=LOG(F)
284 TC=(1.0/(FKT*(F-1.0)))*TC
285 BOD2=(FKT*TC)
286 BOD2=2.71828**BOD2
287 BOD2=5.39134*Q*DC*F*BOD2
300 TC=F*(1.0-((F-1.0)*(DAT/BOD2)))
301 TC=LOG(TC)
302 TC=(1.0/(1.0-F))*TC
303 TC1=2.71828**TC
304 BOD1=5.39134*DC*F*Q/TC1
305 IF(BOD1-BOD2) 320,325,325
320 TEST=BOD2/BOD1
321 GO TO 326
325 TEST=BOD1/BOD2
326 IF(TEST-1.01)330,330,327
327 BOD2=BOD1
328 GO TO 300
330 TC=(1.0/FKT)*(-TC)
331 TSEGA=TC
```

```
  332 TSEGB=2.0*TSEGA
  333 B=(-SKT)*TSEGB
  334 B=2.71828**B
  335 A=(-FKT)*TSEGB
  336 A=2.71828**A
  337 C=(FKT*BOD1/(SKT-FKT))
  338 DFT=C*(A-B)+DAT*B
  339 IF (DAT-DFT) 340, 350, 350
C      340DFT NOT LARGE ENOUGH  350 TOO LARGE
  340 TSEGA=TSEGB
  341 GO TO 332
  350 TSEGC=TSEGB
  351 TSEGB=0.5*(TSEGA+TSEGC)
  352 B=(-SKT)*TSEGB
  353 B=2.71828**B
  354 A=(-FKT)*TSEGB
  355 A=2.71828**A
  356 C=(FKT*BOD1/(SKT-FKT))
  357 DFT=C*(A-B)+DAT*B
 1357 IF(DFT-0.0)1358,358,358
 1358 PRINT 6
    6 FORMAT(22HEFFOR NEGATIVE DEFICIT)
  358 IF(DAT-DFT)400, 410, 410
  400 RATIO=DFT/DAT
  401 IF(RATIO-1.01) 500, 500, 405
  405 TSEGA=TSEGB
```

```
406 GO TO 351
410 RATIO=DAT/DFT
411 IF(RATIO-1.01) 500, 500, 415
415 TSEGC=TSEGB
416 GO TO 351
C     TSEG HAS BEEN EVALUATED
500 ALOAD=5.39134*Q*DC*TSEGB*SKT
501 BOD1 = BOD1*(1.0-(2.71828**(FKT*TSEGB)))
503 FKT=.435
505 EFF=BOD1/ALOAD
1505 RATIO=TC/TSEGB
  8 FORMAT(F10.1,F6.3,F5.2,F5.2,F12.1,F12.1,F6.3,F10.2,F6.3)
506 PUNCH8,Q,SKT,DC,DA,BOD1,ALOAD,EFF,TSEGB,RATIO
507 PRINT 7
  7 FORMAT(14HREAD NEXT FLOW)
511 GO TO 110
512 STOP
513 END
```

Program 5b Listing

```
100 PUNCH 1
  1 FORMAT(22H POINT LOAD COMPARISON)
101 PUNCH 2
  2 FORMAT(26H PREPARED BY ROY W HANN JR)
110 READ 10,Q
 10 FORMAT(E12.5)
111 FKT=.435
  3 FORMAT(I3)
124 SKT=0.1
125 DC=4.2
126 DA=0.5
127 DO 506 K=1,10
128 KA=K
129 PRINT 3,KA
130 Q=2.0*Q
135 PUNCH 4
  4 FORMAT(38H      FLOW    K2    DC    DA POINT LOAD )
136 PUNCH 5
  5 FORMAT(38X,34HUNIFORM LOAD    EFF    SEG TIME RATIO)
280 DAT=5.39134*Q*DA
282 F=SKT/FKT
283 TC=LOG(F)
284 TC=(1.0/(FKT*(F-1.0)))*TC
285 BOD2=2.71828**BOD2
286 BOD2=2.71828**BOD2
```

```
287 BOD2=5.39134*Q*DC*F*BOD2
300 TC=F*(1.0-((F-1.0)*(DAT/BOD2)))
301 TC=LOG(TC)
302 TC=(1.0/(1.0-F))*TC
303 TC1=2.71828**TC
304 BOD1=5.39134*DC*F*Q/TC1
305 IF(BOD1-BOD2)320,325,325
320 TEST=BOD1/BOD2
321 GO TO 326
325 TEST=BOD2/BOD1
326 IF(TEST-1.01)330,330,327
327 BOD2=BOD1
328 GO TO 300
330 TC=(1.0/FKT)*(-TC)
331 TSEGA=TC
332 TSEGB=2.0*TSEGA
333 B=(-SKT)*TSEGB
334 B=2.71828**B
335 A=(-FKT)*TSEGB
336 A=2.71828**A
337 C=(FKT*BOD1/(SKT-FKT))
338 DFT=C*(A-B)+DAT*B
339 IF (DAT-DFT)340,350,350
C     340DFT NOT LARGE ENOUGH  350 TOO LARGE
340 TSEGA=TSEGB
341 TO TO 332
```

```
 350 TSEGC=TSEGB
 351 TSEGB=0.5*(TSEGA+TSEGC)
 352 B=(-SKT)*TSEGB
 353 B=2.71828**B
 354 A=(-FKT)*TSEGB
 355 A=2.71828**A
 356 C=(FKT*BOD1/(SKT-FKT))
 357 DFT=C*(A-B)+DAT*B
1357 IF(DFT-0.0)1358,358,358
1358 PRINT 6
   6 FORMAT(22HEFFOR NEGATIVE DEFICIT)
 358 IF(DAT-DFT)400, 410, 410
 400 RATIO=DFT/DAT
 401 IF(RATIO-1.01) 500,500,405
 405 TSEGA=TSEGB
 406 GO TO 351
 410 RATIO=DAT/DFT
 411 IF(RATIO-1.01) 500, 500, 415
 415 TSEGC=TSEGB
 416 GO TO 351
C      TSEG HAS BEEN EVALUATED
 500 ALOAD=5.39134*Q*DC*TSEGB*SKT
 501 BOD1=BOD1*(1.0-(2.71828**(FKT*TSEGB)))
 503 FKT=.435
 505 EFF=BOD1/ALOAD
1505 RATIO=TC/TSEGB
```

```
  8 FORMAT(F10.1,F6.3,F5.2,F5.2,F12.1,F12.1,F6.3,F10.2,F6.3)
506 PUNCH8,Q,SKT,DC,DA,BOD1,ALOAD,EFF,TSEGB,RATIO
507 PRINT 7
  7 FORMAT(14HREAD NEXT FLOW)
511 GO TO 110
512 STOP
513 END
```