A POLLUTION MODEL OF THE CHARLES RIVER BASIN

by

William W. Walker, Jr.

Submitted in Partial Fulfillment
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Signature of Author : ....
Department of Chemical Engineering
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Certified by : ....
Thesis Supervisor

Accepted by : ....
Chairman,
Departmental Committee on Graduate Thesis

ABSTRACT

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The polluted condition of the Charles River Basin can be traced to three factors: its low dilution capacity, its impoundment, and the wastes which it accepts from the surrounding city. Like many urban rivers, the basin is subject to combined sewer overflows and storm-water runoff. Information about the quantities and origin of the pollution sources in the basin is needed in order to evaluate plans for enhancing water quality.

A mathematical model of the basin is developed for the purpose of quantifying sources of biochemical oxygen demand and determining their distribution. The results indicate that 40% of the BOD entering the basin can be attributed to storm-water runoff and 60% to sanitary sewage escaping in combined overflows. Programs designed to enhance water quality in the basin should thus focus both on eliminating combined overflows and on reducing the pollution potential of storm-water runoff by improving the sanitary conditions of the city.

Thesis Supervisor: Robert C. Reid

Professor of Chemical Engineering

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1. Summary

The Charles River Basin has three distinguishing characteristics which relate to its present state of pollution. First of all, it is a relatively small river with a low dilution capacity and which flows through a highly populated and paved area. Second, the basin is impounded, rendering it susceptible to sedimentation, vertical stratification, and algal activity. Finally, the basin is subject to inflow from both urban storm-water runoff and combined sewer overflows.

Many plans for increasing the recreational and aesthetic value of the basin have been proposed. Typically, these plans have focused on the elimination of one or more pollutant sources. The sources have been generally characterized but not sufficiently quantified to provide an adequate basis for comparison and evaluation of the varous abatement proposals.

In this interest, a mathematical model of the basin has been developed for the purpose of determining the distribution of carbonaceous BOD sources in the basin. The model employs a mass balance concept and utilizes experimental measurements of BOD_5 taken at various locations in the basin by the M.D.C. (3). The model is applied to data taken before and after the activation of the South Charles Relief Sewer. The results reflect a statistically significant 20% reduction in the total source quantities as a result of the activation of this major sewer.

The BOD sources calculated for various segments of the river are also found to reflect the characteristics of the sewage systems in the local drainage areas. The local drainage area for any segment is defined as the area of land draining directly into that segment. A significant correlation is developed relating the yearly quantities of BOD contributed to each segment per acre of local drainage area to the percent of the area served by separate sewers. The results indicate that combined sewer systems contribute, on the average, 6.2 times the quantity of BOD contributed by separate systems per unit area. Overall, 71.5% of the land draining directly into the basin is served by separate sewers, and 28.5% is served by combined sewers.

This information is used to determine the split of the total BOD sources between storm-water runoff and sanitary sewage which escapes in combined overflows. The results indicate that about 40% of the carbonaceous BOD entering the river originates from runoff and about 60% originates from sanitary sewage. Using typical concentrations for urban runoff and sanitary sewage, the source distribution is determined for suspended solids, total nitrogen, total hydrolyzable phosphate, and coliform bacteria. In every case except the latter, urban runoff makes up a significant percentage of the total quantity of each material entering the river.

On this basis, unfortunately, it is not clear that even complete sewer separation would solve the pollution problems of the Charles. The characteristics of the sewage systems in the area cannot be blamed entirely for the river's condition. The pollution potential of urban runoff depends on many factors relating to the overall sanitary conditions of a city. In its street cleaning and garbage collecting procedures, the city can control some of these factors. However, many, such as littering, spillage, or dustfall, are inherent in human nature or in the nature of the city. These factors are basically uncontrollable.

These results indicate, then, that the best plan for pollution abatement in the Charles is one which proposes to remove both combined overflows and storm-water runoff, i.e., the Boston Deep Tunnel Plan (35). This conclusion could obviously have been reached without the above considerations, but this work demonstrates that storm-water alone is a significant problem and that relatively drastic measures, such as the Deep Tunnel Plan, might have to be taken in order to clean up the Charles. The prohibitive expense of this plan, of course, eliminates it as a realistic recommendation. Instead, the recommendation is made that efforts to improve Charles River water quality focus not only on eliminating or treating combined overflows, but also on reducing the pollutional threat of urban runoff by improving the sanitary conditions of the city.

2. Introduction

The Lower Charles River has the misfortune of flowing through highly populated Metropolitan Boston. It is one of many in the country and in the world which have become victims of urbanization. The condition of the lower section of the river can be traced not to industry or agriculture, but to people and pavement. The storm and sanitary sewage collection facilities have been inadequate to handle the rampant population growth which the area has endured over the past twenty-five years. Pavement alone has caused problems by producing greater quantities of storm-water runoff which, in turn, has carried the litter and dustfall from the city into the river. The result has been the deterioration of water quality to the extent that bathing beaches which were enjoyed as recently as 1949 now lie strewn with rubber tires, oil, and putrid debris.

The task of improving the Lower Charles, with which this work is primarily concerned, is a very difficult one. It is the same task which many other cities must face in an effort to improve the urban environment as a whole by increasing the recreational and aesthetic values of urban rivers. Concern over problems of this sort has erupted much too late - after the planning stage, the prime time for the most economical and efficient preventative measures. Relatively expensive and inefficient reparative measures must now be adopted. In the past, efforts to solve these problems have been stymied by a lack of funding both for use of existing technology and for research to produce new, more efficient, and more economical technology. Recently, the situation has begun to improve, as the city, state, and federal governments and the people themselves have begun to focus more on urban and environmental problems. The Charles River, as this work will demonstrate, is a prime example of the interactions between land, air, and water pollution and of what a lack of consciencious urban planning can do to the environment.

The Charles originates in Hopkinton, southeast of Boston, and winds eighty miles to the sea as it drains about three hundred square miles of eastern Massachusetts. The upper portion of the river, defined as the seventy-mile section above the Moody Street Dam in Waltham, suffers from industrial and sewage pollution as it passes through rural areas and relatively small towns. Most of the waste sources in this reach have been clearly defined and placed on implementation schedules by the state pollution control agency, which has a program to upgrade the water quality of the river (1).

The Lower Charles consists of three segments: (a) a 2.9-mile section between the Moody Street Dam in Waltham and the Watertown Dam in Watertown, (b) the "Charles River Basin", an 8.6-mile impounded section with no elevation change between the Watertown Dam and the Charles River Dam at the Museum of Science in Boston, and (c) a 1.2-mile estuarine portion between the Charles River Dam and the mouth of the river in Boston Harbor. The land which drains into the Lower Charles is for the most part densely populated. This section of the river is not subject to any known appreciable pollution of industrial origin. The characteristics of the sewage systems in the area are held primarily responsible for the river's condition.

As a study by Process Research, Inc. (2) points out, the Lower Charles has received a definite lack of attention relative to the Upper Charles. Most of the water quality surveys have been concentrated on the upper portion of the river, despite the fact that 95% of all the water in the Charles lies below the Watertown Dam and 70% of all the people who live in the watershed reside in areas which drain into the Lower Charles. With restrictions in manpower and funding, perhaps it has been considered more logical to concentrate on the upper portion of the river first, particularly since the pollution sources in this reach are quite clearly defined and the technology for reasonably economical abatement of these sources has been developed. The poor definition of sources and the lack of economical abatement technology characterize the problems of the Lower Charles. The major water quality surveys of the Lower Charles to date consist of two continuing programs by the Metropolitan District Commission (3,4), a program undertaken by the Federal Water Quality Administration in the summer of 1967 (5,6), and an extensive survey of the basin done by Process Research, Inc. of Cambridge during the summer of 1969 (2).

This work is concerned primarily with the Charles River Basin. There are three distinguishing characteristics of the basin which, in one way or another, account for its condition:

(a) It has relatively little flow.

(b) It is impounded.

(c) It is subject to pollution from urban storm-water runoff and combined storm and sanitary sewage overflows.

In describing the basin, it would be essential to consider all of these factors and to demonstrate their influence on water quality. It would also be of interest to relate these characteristics to those of other urban rivers. This would help to put the Charles into perspective and to provide some insight as to just how tragic the relationship between Boston and the Charles River is, relative to analogous relationships in other locations.

2.1 Consequences of Flow

A river's capacity to accept and assimilate wastes is strongly dependent upon the amount of dilution is can provide. High concentrations of wastes can create conditions which will halt desirable biological purification processes. The Charles is a relatively small river flowing through a highly populated area; it is thus in a relatively susceptible position to being seriously overburdened by wastes directly attributed to people: sanitary sewage and storm sewage.

In order to appreciate how susceptible the Charles is on this basis, it would be useful to calculate its "dilution parameter", defined by Fair and Geyer (7) as the stream flow in cubic feet per second divided by the watershed population in thousands. A search of the literature has provided the necessary information to calculate dilution parameters for other urban rivers. These values are presented in Table 2-1. 4 cfs per 1,000 population is the recommended minimum value of this parameter (7). The interpretation of this minimum value is that 4 cfs are required for every 1,000 population equivalents of waste entering the river in order to avoid "objectionable conditions". Of course, not all rivers are forced to accept all of the wastes produced in their watersheds,so the parameters listed for the various rivers indicate pollution potential rather than actual waste loadings.

The Lower Charles, fortunately, is not subject to industrial pollution, as are most of the other rivers cited. The significance of the dilution parameter is that, in the interest of clean water, large cities that are built near small rivers must take adequate measures to prevent any appreciable wastes from entering the river. Low dilution parameters necessitate drastic measures, i.e., highly efficient waste collection and treatment facilities. The Charles, unfortunately, both has a low dilution capacity and is prone to a sewer system that is in many ways outmoded and overburdened.

TABLE 2-1

Dilution Parameters for Various Urban Rivers

River	Watershed[a] Population (thousands)	Mean Flow (cfs)	Dilution Parameter[b] (cfs/1,000 pop.)
Potomac (Washington)	3,000	11,000	3.66
Hudson (New York)	6,000	21,500	3.58
Connecticut (Hartford)	162	16,070	9.90
Cuyahoga (Cleveland)	739	852	1.15
Passaic (N.E.New Jersey)	1,600	1,180	.74
Charles (Boston)	600	280	.47

a - in metropolitan area only

b - recommended minimum value = 4 cfs/1,000 population (7)

2.2 Consequences of Impoundment

The impoundment of any river has a significant effect on the physical, chemical, and biological processes which otherwise occur (8). The Charles River Basin, formed in 1910 with the completion of the Charles River Dam, behaves more like a lake than a river. The impoundment of the river essentially sealed it off from the natural flushing action of the tides and caused it to become a large, stagnant, and vertically stratified pool. Before considering some of the specific influences of impoundment on water quality, a general description of the dam, its history and operation is in order.

The impoundment of the Charles occurred at the turn of the century, partially as a result of popular opinion to eliminate the unsightly and foul-smelling mud flats which had been exposed at low tide. There is little doubt that the foul odors were a result of anaerobic degradation processes occurring in the mud. The organic materials in the mud were of sewage origin. The construction of the dam could be viewed as an attempt to isolate the undesirable effects of an inadequate sewage system, essentially by covering them over with water from the Charles. To this day, the river has served this purpose.

There was apparently little knowledge of (or concern for) the possible effects of such an impoundment on water quality. The Charles will never be allowed to return to its natural estuarine state, since most of the construction in the area surrounding the basin is dependent upon a constant water table. Aside from this, in the event that the impoundment were eliminated, the Boston Harbor, in its present condition, would probably supply more undesirable materials than the flushing action of the tides would carry away.

The Metropolitan District Commission has responsibility for the operation and maintainence of the dam. The dam is equipped with one lock and one sluiceway, and operation is aimed at maintaining the basin elevation at 2.38 feet above mean sea level. Since the dam is not equipped with pumping facilities, the basin cannot be drained for approximately four hours during each tidal cycle, when the sea level is above the basin level. Heavy rainstorms and high runoff into the basin at high tide can result in flooding; this occurred in August of 1955 and March of 1968. As a precaution against such flooding, the basin is predrained in anticipation of rainstorms. In cases where the anticipated rainfall does not occur, sea water is allowed to enter the basin in order to keep the level at 2.38 feet. Sea water also enters the basin through leakage and operation of the locks.

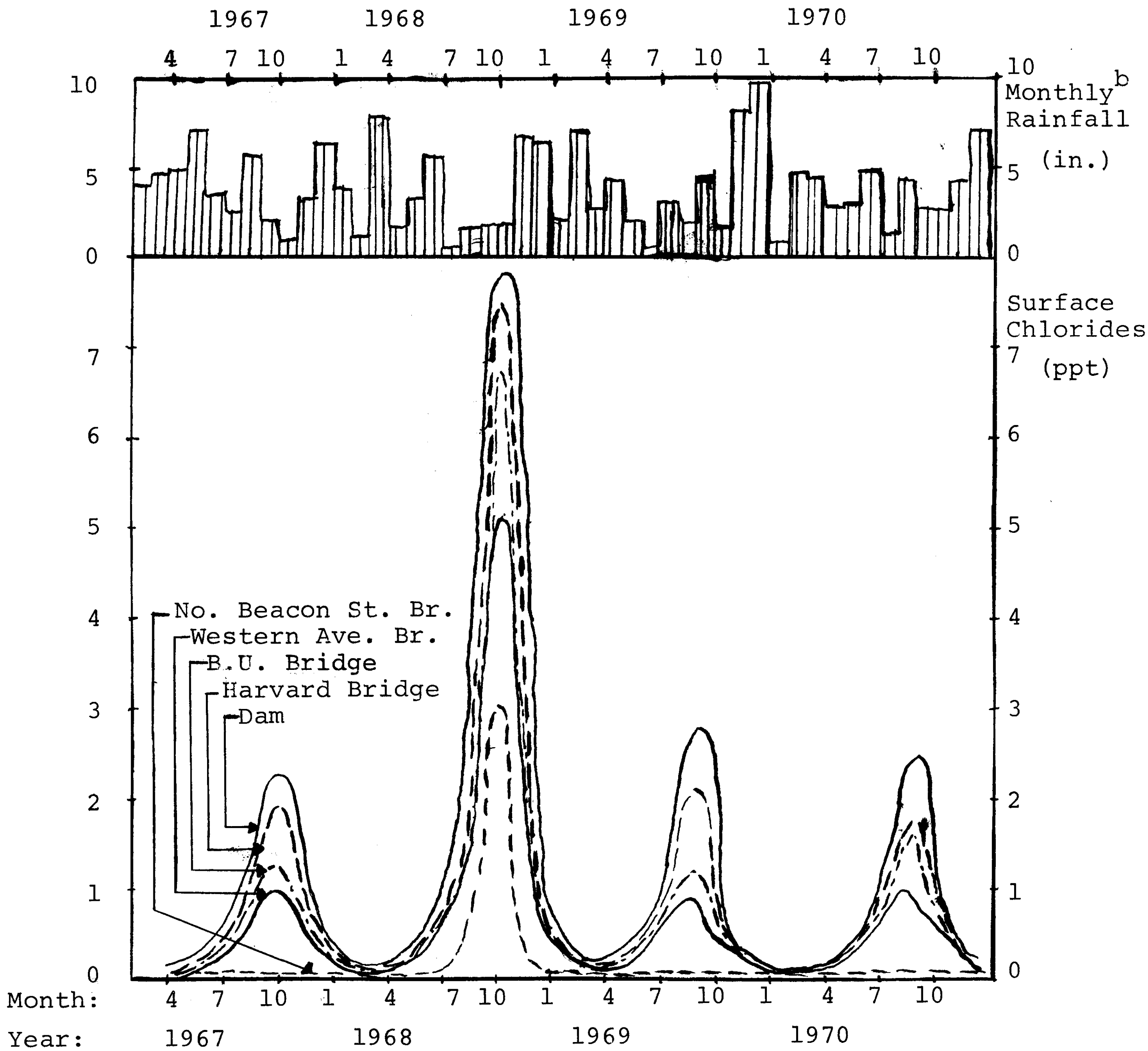
A study done by Charles A. Maguire Associates (9) revealed that between July and October of 1957, a particularly dry season, about 620 million pounds of salt entered the basin and about 380 million pounds left, a net increase of 240 million pounds. About three quarters of the net amount of salt entering was due to lockings and about one quarter was due to sluicing for elevation control. In October of 1957, the basin was estimated to contain about 60% sea water.

The extent of salt accumulation during any summer apparently depends on rainfall, as is shown in Figure 2-1, a plot of the surface chlorides measured by the M.D.C. (3) at five locations in the basin over the past four years. As shown, the chloride concentrations decrease with increasing distance upstream. In the summer of 1968, surface chloride concentrations were significantly higher than in other years. A significant increase in chlorides was detected as far upstream as the North Beacon Street Bridge, some seven miles from the dam. The total rainfall for the months of July, August, and September in 1968 was 3.97 inches, compared with an average of 9.16 inches for the years 1931-70 (10). In the summer of 1957, when the Maguire study was done, the total rainfall was only 2.70 inches. The significance of the intrusion of salt is in its effect on the mixing properties of the basin; this subject will be dealt with presently.

The overall effects of the impoundment on water quality can be divided into three categories: sedimentation, vertical stratification, and algal activity.

FIGURE 2-1

Seasonal Variation of Surface Chloride Concentrations[a] in the Charles River Basin



a - measured by the M.D.C. (3)

b - measured at Logan Airport (10)

2.2.1 Sedimentation

Impounding the Charles has had the effect of increasing the effective cross-sectional flow area, thereby reducing flow velocities. Velocities on the order of 0.6 fps are required to prevent sedimentation of suspended solids in a river, while velocities of about 1.2 fps are required to effectively scour the river bottom of solid deposits (11). If flow velocities are too low, rivers subject to pollutant sources containing solid materials deposit and accumulate these solids. If they are of an organic nature, the process of biological oxidation of these materials will cause depletion of dissolved oxygen at the bottom of the river. The anaerobic degradation processes which follow not only retard the rate of assimilation of these organic materials, but produce noxious gases, such as hydrogen sulfide and ammonia. Such bottom conditions effectively exclude fish and produce foul odors at the river's surface, as commonly observed near the Charles.

The flow velocities in the Charles River Basin are much too low to prevent sedimentation. A time-of-travel study done by the Federal Water Quality Administration in the summer of 1968 (12) showed that at a flow of 342 cfs, measured at the U.S.Geological Survey Gauge in Waltham, the mean surface velocity of the river was .151 fps between the Watertown Dam and the B.U.Bridge and .078 fps between the B.U.Bridge and the Charles River Dam. Assuming that velocity is approximately proportional to volumetric flow, flows of about 1200 cfs and 2400 cfs are required to prevent sedimentation in the upper and lower sections, respectively. The average annual flow of the Charles at Waltham is about 280 cfs. An examination of the mean daily flow records at Waltham revealed that since October 1, 1962 only twenty five days recorded flows greater than 1200 cfs, and only three days recorded flows greater than 2400 cfs(13). The entire basin, particularly the lower section, is therefor subject to sedimentation and sludge accumulation.

The sources of solid materials which are liable to settle out are both external and internal. The storm-water runoff and combined sewer overflows which enter the basin from the surrounding area contain suspended solids, as does the water entering from upstream. The Upper Charles is not as subject to sedimentation because of narrower channel widths, steeper elevation gradients, and resultant higher flow velocities. The internal source of sediment is primarily algae, which have been detected in excessive amounts in the lower basin by Process Research (2) and the F.W.Q.A. (6). The biological degradation of organic materials in the river produces carbon dioxide which, in combination with phosphate and nitrate nutrients in the water, stimulates algae growth. As the algae die, they settle to the bottom.

The materials which settle out of the basin surface waters either accumulate or decay. The Army Corps of Engineers (14) estimates that sediment is accumulating in the basin at a rate in excess of 8,000 tons per year and that if sedimentation continues at its present rate, the volume of the basin will be significantly reduced by the year 2020. Some of the material of organic nature which settles out is subject to degradation, either areobic or anaerobic, depending on the availability of oxygen in the sediment. Anaerobic activity probably dominates, since oxygen levels in the depths of the basin are low, particularly in the downstream section (2).

2.2.2 Vertical Stratification

The most interesting and significant effect of the dam on water quality is in its effect on vertical mixing properties. Impoundments are commonly characterized by a lack of vertical mixing (8). Mixing is inhibited by the density difference between surface and bottom layers. In the case of the Charles, this density difference is caused by two factors: thermal and saline stratification. A simplified view of the vertical stratification divides the basin into two distinct zones: an upper region where the active flow of the river occurs, and a lower, more dense, stagnant region relatively high in salt content and low in temperature.

The most conclusive evidence of this stratification is contained in studies by Process Research, Inc. (2) and the F.W.Q.A. (12). Some of the results of the latter study are contained in Appendix A. These studies illustrate the lack of vertical mixing in the basin during the summer months. Little or no evidence is available, however, indicating whether this is the case during other seasons of the year.

Impoundments not subject to saline intrusion commonly exhibit thermal stratification during the warm seasons. As the air temperature drops in the fall, the surface waters cool and approach the temperature of the bottom layer. As a result, the so-called thermocline is then destroyed and the lake or impoundment effectively turns over and becomes vertically mixed.

The Charles is subject to both thermal and saline stratification. The amount of salt remaining in the bottom layers throughout the year may be enough to prevent turn-over and vertical mixing. It is surprising that there is no published evidence concerning this question.

The relative importance of thermal verses saline variation to flow stratification may be partially determined by their effects on the density of water (7). Temperature variation between the top and bottom layers is generally on the order of 5° C during the summertime; the absolute maximum variation is about 10° C. The difference in density between water at 10° C and water at 20° C is approximately $.9997 - .9982 = .0015$ g/cm^3 (7). This represents the maximum effect of thermal stratification on density. In June of 1968, according to the F.W.Q.A. study (12), salinity varied from about 1 part per thousand at the surface to more than 20 ppt in the bottom layers of the basin. This represents a density difference of roughly $1.020 - 1.001 = .019$ g/cm^3 due to saline variations, as compared with a maximum of .0015 g/cm^3 due to thermal variations. This tends to indicate that saline gradients are more important in inducing vertical stratification of flow. The question still remains whether the salt has time to diffuse out of the lower layer during the late fall, winter, and early spring, when the primary source of salt, lock operation, is cut off.

The only evidence that the basin remains vertically stratified throughout the year is indirect. Reference to the Maguire study (9) reveals that a significant amount of salt remained in the basin over the winter season preceeding the summer of 1957. The basin was estimated to contain approximately 60% sea water in October of 1957; this is equivalent to a volume of 264 million cubic feet, assuming a total basin volume of 440 million cubic feet (14). Since the salinity of sea water is 30 ppt, this is equivalent to a total accumulation of 494 million pounds. Maguire estimates that the net amount of salt entering the basin during the summer of 1957 was 240 million pounds. According to this calculation, a total of 254 million pounds of salt must have been in the basin at the beginning of the summer. Assuming that the surface salinity had fallen to low values during the previous winter, as the M.D.C. data presented in Figure 2-1 indicate for later years, most of the 254 million pounds of salt had apparently remained in the lower depths of the basin over the winter. There is still no assurance, however, that this occurs every year.

Nevertheless, there is another piece of indirect evidence pointing to year-round stratification. The quality of the water in the lower depths of the basin in the summer is very low; it is essentially depleted of dissolved oxygen and high in hydrogen sulfide content. The reasons for this will be discussed presently. It a turnover does occur during the fall or spring months, one would expect to find that the water quality at the surface deteriorates significantly. The monthly surface samples taken by the M.D.C. over the past five years do not indicate this (3).

The primary consequence of the stagnation of the lower reaches of the basin caused by flow stratification is in the lack of oxygen transfer to the bottom section. Molecular diffusion of oxygen does not occur at a rate sufficient to keep up with oxygen consumption caused by the biodegradation of organic materials. Turbulent diffusion processes are necessary to prevent anaerobic conditions. This point is illustrated by calculations outlined in Appendix B. These calculations show that even at organic concentrations and resulting oxygen consumption rates one fifth as great as those found in Charles River water, stagnant water will be depleted of dissolved oxygen less than 10 cm from flowing, oxygen saturated regions. The consequence of oxygen depletion is the development of anaerobic conditions producing ammonia and hydrogen sulfide, toxic compounds which effectively exclude fish and can yield foul surface odors, particularly if the bottom is disturbed.

The evidence that the salt wedge is anaerobic during the summertime is quite conclusive (2). The fact that salt seems to remain in the lower reaches of the basin over the winter does not necessarily indicate that the wedge remains anaerobic throughout that period. The loss of salt from the wedge and the accompaning decrease in biological deoxygenation rates with temperature may be sufficient to reduce the size of the anaerobic layer a great deal. Vertical profiles of temperature, salinity, and dissolved oxygen should be taken during all seasons to determine conclusively whether mixing and aeration of the bottom layers does occur.

Kojima Bay in Japan is an example of an impoundment which is quite similar to the Charles River Basin, in that it has a high surface area with relatively low fresh water flow and it is subject to saline intrusion through locking. Okuda (15) has studied the change in the salinity distribution in the bay since its closing. The bay is characterized by a stable interface zone between surface river water and lower sea water. An aqualung survey revealed a "very sharp difference in temperature and suspended matter between surface and bottom water". The level of the interface in Kojima Bay is controlled by the height of the sill of the sluice through which fresh water passes on its way to the sea. This means that the equilibrium upper level of the salt wedge is determined by the vertical position of the outlet. This evidence tends to strengthen a proposal by Process Research (2) which states that to minimize the basin salt wedge a barrier should be built to lower the level of the sluice outlet. The plans for the new dam to be built at Warren Avenue (14) should incorporate this design or its equivalent. The proposed dam is supposedly designed to cut down on saline intrusion by a factor of about two thirds. It is unclear, however, whether this alone is sufficient to prevent the formation of a stable anaerobic salt wedge. The most effective way of preventing salt accumulation in the basin is by lowering the effective outlet and making sure there are no stable deep pockets within the basin bottom topography in which salt could accumulate.

If facilities for draining the anaerobic layer from the basin are constructed, care should be exercised in how and when they are activated. Assuming that the anaerobic salt wedge takes up the volume of the basin below 12 feet in depth, it is estimated that the total volume of the wedge is 1.6×10^8 cubic feet. It drainage of the wedge were to occur by its displacement with water from upstream at a rate of 300 cfs, it would take as long as 46 days to deplete the layer. The quantities of hydrogen sulfide released during these days might make Boston unbearable! Drainage of the wedge should take place gradually in the spring when its size is at a minimum and when the fresh water flow into the basin is maximum.

2.2.3 Algal Activity

The third influence of the impoundment of the Charles on water quality is the stimulation of algal activity. This is related to the sedimentation and vertical stratification effects. The increased surface area of the impoundment provides additional exposure of the water to the sun, and this, plus increased residence times, serves to stimulate algal activity. The problems of excessive algae growth, as related to the process of eutrophication, are problems generally attributed more to lakes and impoundments than to rivers. The Charles River Basin, with its abundance of nutrients (2), is an ideal setting for algal blooms, which produce foul odors and aesthetically displeasing water.

The contributions of algae to the overall oxygen balance in reserviors and estuaries like the Charles cannot be ignored. Photosynthesis and atmospheric reaeration provide the oxygen which is consumed by the biodegradation of organic materials. In his work on the Baltimore Harbor, Hull (16) calculates that in the summertime algae produce 600,000 pounds of oxygen daily,whereas atmospheric reaeration provides only 187,000 pounds per day. Algae may be as important a source of oxygen in the Charles as they are in the above case.

Nevertheless, algal consumption of oxygen cannot be ignored. Symons et all (8) state that the oxygen demand of the algal population in water takes three forms: (a) respiration that occurs while photosynthesis is progressing,

(b) respiration that occurs at night when photosynthesis is absent, and (c) oxygen uptake caused by bacteria that metabolize the algal bodies upon their death. Verduin (17) estimates that if all the algae stayed in the upper waters of an impoundment, the net 24-hour contribution to the oxygen balance would be near zero. However, there generally is a net contribution of oxygen to the surface waters because many algae fall to the bottom during a given 24-hour period. The algae which leave the surface layers either exert their oxygen demands attributed to respiration and degradation in the bottom layers or, in the absence of oxygen in the bottom layers, merely accumulate as natural sediment. A simplified view of this process is that in order for net algal production of oxygen to occur, dead algae must accumulate as sediment.

Virtually all of the measurements of dissolved oxygen in the basin have shown that the surface layer is high in oxygen content during the day. In fact, in conjunction with work done with the Interdisciplinary Enrivonmental Projects Laboratory at M.I.T., the author has measured supersaturated values of dissolved oxygen in October near the Harvard Bridge and at depths up to eight feet. The supersaturation can only be attributed to algae. Supersaturated levels of dissolved oxygen are detrimental to the oxygen balance of basin because of the accompaning loss of oxygen to the atmosphere. Mechanical mixing to prevent supersaturation by combining surface and relatively oxygen deficient bottom waters has been investigated as a means of preventing this moss of oxygen (8).

Some fundamental questions about the behavior of algae in the Charles River Basin must be answered before any conclusions about their effects on water quality can be drawn. Aesthetically, their effects could only be detrimental. The extent of their proliferation must be determined conclusively as a function of season and depth. Their contribution to the oxygen balance of the basin must be examined by determining where and when their consumption and production of oxygen occurs. If significant numbers of algae remain in the surface waters at night, the dissolved oxygen levels in these regions may be drastically depressed. If most of the algae settle into the bottom layers at night and accumulate there as natural sediment, they may be viewed as important and beneficial source of oxygen to the basin, despite their effects on the bottom. Much useful information could be obtained about the behavior of algae in the Charles from vertical profiles of dissolved oxygen taken over daily cycles.

2.3 Pollution Sources

The 600,000 people residing in the Lower Charles watershed contribute wastes to the river in two primary forms: sanitary sewage and storm sewage. Sanitary sewage enters the basin when combined sewer systems in the area overflow during periods of rainfall. Storm sewage carries the litter and dustfall from the pavements and rooftops of the city into the river through both combined and separate sewer systems. Each of the two types of waste has its own particular characteristics and effects on Charles River Water quality. The problems of the Charles are directly related to the amount and content of combined sewer overflows and urban runoff. Before considering in detail how each of these sources contributes to the Charles, it would be interesting to determine what kinds of generalizations can be made from studies made elsewhere.

2.3.1 Urban Storm-water Runoff and Combined Sewage Overflow-General Treatment

The content, collection, and disposal of urban storm-water runoff and combined sewage overflow are subjects which are of definite relevance to the health of urban waterways. In 1962, of the 11,400 sewered communities in the United States, 9,083 had separate sewer systems, 1,305 had combined systems, and 618 had a mixture of both (18). On a population basis, in 1967 it was determined that between 54 and 55 million people in the United States were served wholly or partially by combined systems, 36 million were served directly by combined sewers, and between 60 and 65 million were served by separate storm sewers (19). The overflow of sewage from combined systems can contribute significant quantities of organic materials, nutrients, and disease-causing bacteria and viruses. The notion that separate sewage systems necessarily solve pollution problems is, however, not valid, since the quality of urban runoff depends on many factors relating to the overall sanitary condition of a city. In certain situations, interception and partial treatment of combined sewer overflows may be more advantageous than complete sewer separation. A number of cities are facing the question of what to do about pollution due to combined sewer overflow and urban runoff. As a result, many studies have been published on the characteristics of these pollutant sources.

One of the basic difficulties which has plagued studies of this nature has been the lack of knowledge of what particular parameters are the most important to measure. This in turn stems from a general lack of information concerning what particular materials are the most harmful to the aquatic ecosystems and what constitues a lethal dosage. Most of the studies have more or less ignored trace contaminants and focused on gross parameters, such as biochemical oxygen damend (BOD), suspended solids, total coliform bacteria, and, in some cases, nutrients. The reasons for chosing these particular parameters are partially historical. They are not necessarily the most important measurements, though each is indicative of a possible harmful effect on water quality. BOD is used as an indication of the concentrations of organic materials which are subject to biodegradation and pose a threat to the oxygen balance of a river. Suspended solids tend to increase the turbidity of a waterway, thereby decreasing its aesthetic value and the availability of sunlight to desirable aquatic plants. Coliform bacteria, while in themselves not harmful, are used to indicate the possible presence of other, potentially disease-carrying organisms; coliforms are the basis around which water quality standards are designed in many states. Nutrients are also considered, though perhaps to a lesser extent. Phosphates and nitrates are thought to play a leading role in the stimulation of algae blooms and in the eutrophication of lakes.

There is one distinct aspect inherent in the nature of the pollutional threat imposed by urban runoff and combined overflows which merits consideration. While the total quantities of materials contributed by these sources may, in many cases, not appear to be significant on a yearly average basis, the fact that these materials do not enter the waterway continuously must be remembered. The shock loadings imposed on the waterway by a severe storm may, for example, be sufficient to depress dissolved oxygen levels enough to kill fish, to endanger water supplies, or to bring coliform counts in a recreational area up to a level which standards deem unsafe. If the same total quantities of pollutants were discharged continuously over a year no harmful effects may be observed.

Since combined overflows are partially made up of urban runoff, it would be most sensible to consider the characteristics of the latter first. The only sound generalization that can be made about urban runoff is that its quality and, therefore, its pollution potential are reflections of the sanitary conditions of the city. These conditions are, in turn, reflections of many factors, including littering by the ordinary citizen, industrial and commercial spillage control and waste disposal practices, and air pollution (as related to dustfall). The extent to which ordinances against potentially harmful practices are enforced and the frequency and efficiency of the garbage collection and street cleaning operations are the responsibilities of the city officials and determine, in part, the extent of the pollution problem posed by these sources. Of course, because of the dimensions of the problem, government cannot be held wholly responsible and much of the burden lies on the conscience of the private citizen and industrialist. With so many parameters in the problem, it is no wonder that studies have shown a wide variation in the quality of urban runoff. Investigations dealing with urban runoff have approached the problem in two ways: sampling and analyzing the sources and materials on the city streets which are susceptible to being washed away with storm-water, or sampling and analyzing the runoff itself. Studies of each nature are required to successfully examine the extent of the problem and to provide the information necessary to pose reasonable solutions.

An idea of the total quantities of solid material generated in a typical urban area is provided by a study of a ten acre area in Chicago by the American Public Works Association (19). It was estimated that approximately 179 tons of waste solids were generated in the test area per year. Air pollution dustfall contributed 2.9%, domestic sanitary wastes 16.1%, garbage 15.4%, rubbish 56.0%, street sweepings 5.7%, and catch basins 2.9%. It was estimated that public sanitary sewers could remove no more than 20% (sanitary wastes and ground garbage) and that at least part of the remaining 80%, if not promptly removed or stored, could add to storm-water pollution. The objectives of the A.P.W.A. study were to demonstrate that control of urban runoff must be consistent with an optimal waste management program which would give simulataneous consideration to the land, air, and water resources of an urban area.

The A.P.W.A. study also considered the organic content of street litter materials. It was estimated that the runoff from a two hour storm with a previous 14 day accumulation period could carry with it sufficient BOD from the dust and dirt fraction of the street materials to produce a total BOD loading on the receiving waterway equivalent to 160% of the raw sanitary sewage production rate in the area. This shock loading effect could produce significant oxygen sags in the receiving waterway, and is perhaps typical of what might happen to any urban waterway subjected to runoff. Runoff is less of a threat to rural waterways generally because there is less of it, i.e., most of the rainfall soaks into the ground and is therefore filtered before entering the stream through groundwater. The A.P.W.A. study further demonstrated that by preventing the accumulation of dust, dirt, and litter, street cleaning could significantly reduce the pollutional threat imposed by urban runoff.

Examples of concentrations of 5-day BOD, suspended solids, and coliform bacteria commonly found in urban runoff are presented in Table 2-2. A wide variation in concentrations is apparent. Typical concentrations of these materials found in sanitary sewage and in the Charles River Basin are presented for the sake of comparison. Total pollutant quantity estimates will be presented and compared with similar esti-

TABLE 2-2

The Quality of Urban Runoff

City	BOD_5 (mg/liter)	Suspended Solids (mg/liter)	Total Coliforms (number/100 ml)
	Average Concentrations		
Chicago, Ill. [a]	87	613	11,800
Washington, D.C.[a]	126	2,100	---
Seattle, Wash.[a]	10	-	1,610
Oxney, England[a]	100[b]	2,045[c]	-
Detroit, Mich. (29)	96-234	102-213	930,000[b]
Moscow, U.S.S.R. [a]	186-285	1,000-3,500[c]	-
Leningrad, U.S.S.R.[a]	36	14,541	-
Stockholm, Sweden[a]	17-80	30-8,000[c]	20,000[b]
Pretoria, So.Africa[a]	30-34	-	23,500
Tulsa, Okla. (28)	1-39	40-2,000	5,000-400,000
Cincinnati, Ohio (20)	17	227	58,000
Typical Sanitary Sewage (11)	200	200	25,000,000
Typical Charles River Basin (3)	2-7	8-12	11,000-56,000

a - quoted from a table in reference (19)
b - maximum value
c - total solids

mates for combined systems after a brief consideration of the characteristics of combined sewage systems.

Combined systems, designed to intercept both sanitary wastes and runoff, were most reasonable in the days when horses were the primary means of transportation and the city was not covered with pavement. The runoff from a modern city, with its relatively high percentage of impervious surfaces, is generally too much for combined systems to handle. For densely populated areas, combined sewers designed to intercept all of the storm-water runoff would require capacities over 50 times the average dry-weather flow of sanitary sewage(21). This is generally not economically feasible, particularly in view of the fact that it would also require treatment plants which could handle efficiently the greatly expanded rainy day flows. Interceptors and treatment works are generally designed to handle 2 to 5 times the average dry-weather flow and to permit overflows of mixed sewage and storm-water at the points of interception during and immediately following rainstorms. These overflows may represent a significant pollution threat to receiving waterways.

Aside from the collection problem, combined sewers pose treatment difficulties. The highly diversified and fluctuating characteristics of combined sewage can cause problems at the treatment works. The biological systmes commonly used to oxidize wastes are in many ways delicate and require time to adjust to wastes of various forms. Highly dilute wastes, such as might be received after a storm, lead to relatively inefficient treatment and often effectively wash desirable bacteria cultures out of these systems. Combined sewers represent a major stumbling block in the effort to improve biological treatment plant efficiencies.

One consequence of the fact that the dry-weather flow is one half to one fifth the capacity of a combined sewer. is that dry-weather flow velocities are usually too low to prevent sedimentation of sewage solids within the system. These solids accumulate within the system until a storm washes them out. In some cases, these solids are carried out of the system during the early minutes of a storm before the interceptors reach capacity. In others, the scouring of these materials seems to continue for hours after the beginning of a storm and long after overflows have begun.

Combined sewage overflow consists , then, of three components: storm-water runoff, sanitary sewage, and scoured solids. The relative importance of each source and thus the quality of the overflow may vary widely from system to system. At a given location, overflow quality may vary with different storms and in a given storm, with time. These considerations account for the wide differences observed in the quality of overflows, as presented in Table 2-3. These figures may be compared with those presented in Table 2-2 for urban runoff.

An idea of how the quality of a given overflow may vary from storm to storm and with time within a given storm is

TABLE 2-3

The Quality of Combined Sewer Overflows

City		Average Concentrations: BOD_5 (mg/liter)	Suspended Solids (mg/liter)	Total Coliforms (number/100 ml)
SanFrancisco, Cal.	(22)	36	224	50,000
Detroit, Mich.	(23)	153	274	4,300,000
Philadephia, Pa.	(24)	145-243	330-573	-
Buffalo, N.Y.	(25)	100-121	436-544	-
Northampton, Eng.	(26)	80	400	-
Bucyrus, Ohio	(27)	120	380	110,000

presented in Figure 2-2. These are the results of overflow analyses conducted by Noland and deCarlo in Bucyrus, Ohio (27). Figure 2-2 shows the characteristic behavior of BOD_5 concentrations in overflows during storms. Concentrations are generally high at the outset because of the relatively high ratio of sanitary sewage to storm-water and because scouring of organic materials from the sewer lines is at a maximum initially. The concentrations generally decrease as the overflow continues, the relative amount of storm-water increases, and the sediment in the sewers is depleted. Similar curves were developed for other pollutant concentrations. Various other studies (22,24,26) tend to support the conclusion that the first flushes of a storm through a combined sewer system are the most potent.

The quantities of material escaping in an overflow from a combined sewer per year would seem logically to depend on the capacity of the sewer for storm-water, or, more exactly, on the ratio of the capacity of the sewer to the average dry-weather flow of sanitary sewage. Because of the distribution of rainstorm intensities measured in any given year, this relationship is governed by a law of diminishing returns, as is shown in Table 2-4. These figures were calculated by Camp (21) from data gathered at Northampton, England. The BOD and suspended solid quantities escaping in overflows are presented as percentages of the total annual inflow to the combined system. Camp points out that the combined systems from which this data was gathered had rela-

FIGURE 2-2

Characteristic Behavior of BOD_5 Concentrations in Combined Overflows During Storms

Data Gathered by Noland and deCarlo in Bucyrus, Ohio (27)

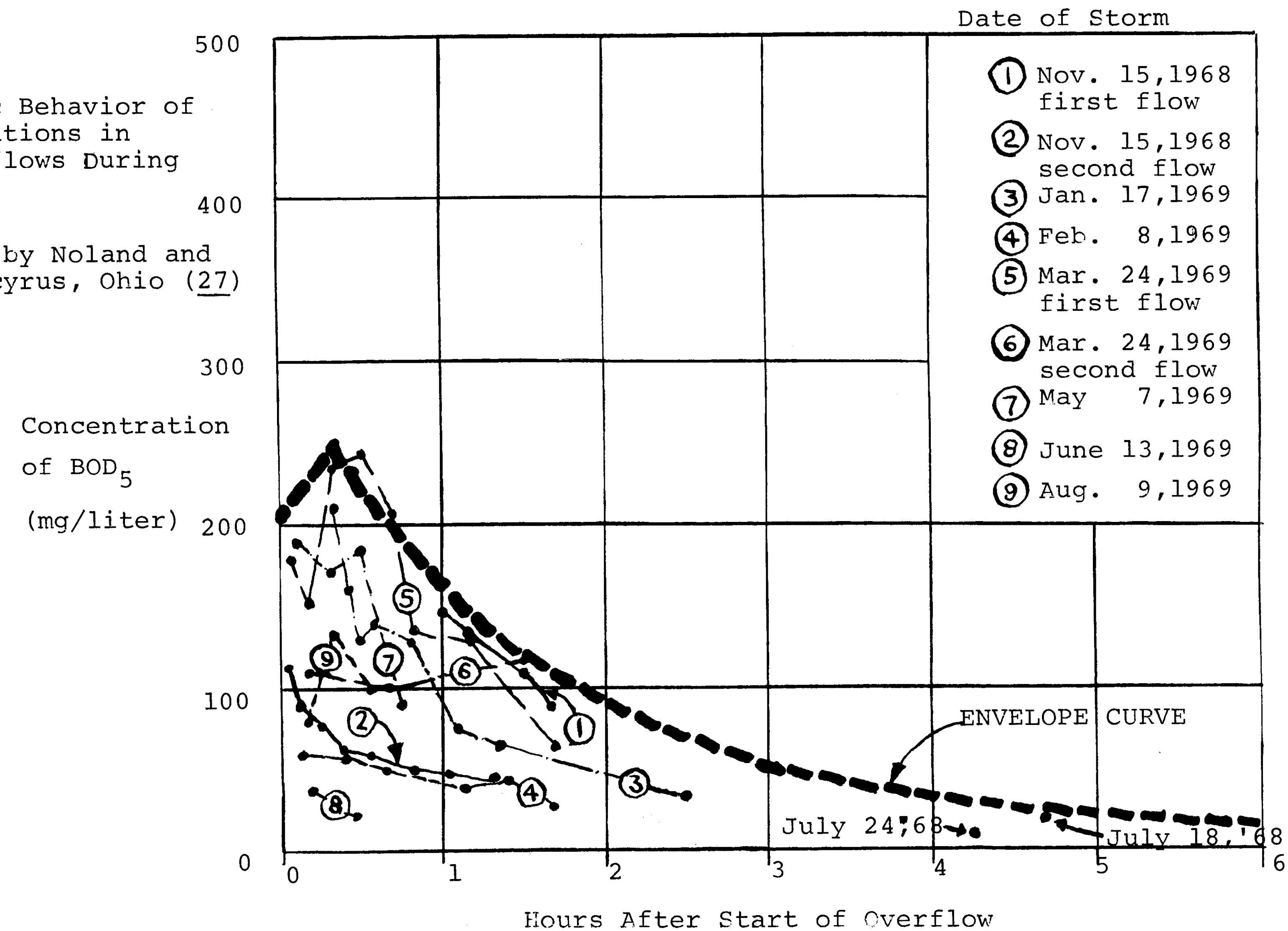


TABLE 2-4

Estimated BOD and Suspended Solids Loads in Overflows of Mixed Sewage and Storm-Water for Various Interceptor Capacities[a]

Interceptor Capacity[b]	Percent of Total Annual Inflow Escaping BOD	Suspended Solids
3	8.1	27.4
6	5.2	18.3
9	3.4	15.7
12	2.5	12.9
20	1.22	7.6
30	.49	3.2
45	.03	.22

a - Camp (21)
b - capacity in multiples of average dry-weather flow

tively high dry-weather velocities. Sginificantly higher losses of BOD and suspended solids might be expected from systems with flat slopes and low velocities, because of the sedimentation effect.

One of the most important kinds of camparisons that can be made between urban runoff or combined overflow and other pollutant sources is in terms of total quantities contributed by these sources per year. Since the quantities of waste produced in an urban envirnoment are related to, among other things, land area, the sources are also normalized on a per acre basis. Such a comparison is presented in Table 2-5 which shows the results of a study by Eckhoff, et. al. (22) of combined sewers in San Francisco. The table quotes results from one of the combined sewer areas studied. The measured load from the existing combined system is compared with loads from primary and secondary treatment plants and with the estimated quantities of material that would be expected if the area studied were served by separate storm sewers. The waste loadings from the combined system are comparable to and, in a few cases, significantly greater that the loadings from secondary treatment of the sanitary wastes produced in the area. The nutrient loadings are relatively slight. Urban runoff seems to make up a significant portion of the solid materials in combined overflows and a less significant portion of the BOD. The shock loading effect of the combined overflows and separate storm-water discharges must be re-

TABLE 2-5

Annual Mass Pollutant Discharges for an Urban Area in San Francisco[a]

Quantities in (lbs./acre-yr.)

Constituent	Primary Effluent	Secondary Effluent	Combined Overflow	Separate Storm-water
BOD_5	1,450	175	101	25
COD	2,420	280	447	188
Suspended Sol.	1,415	105	632	570
Volatile S.S.[b]	940	84	146	125
Grease	344	14	36	-
Total Nitrogen	250	175	10.6	7.0
PO_4 [c]	262	210	2.4	2.0

a - Eckhoff, et. el. (22)
b - Volatile Suspended Solids
c - soluble phosphate only

membered in comparing the yearly average values presented in Table 2-5.

The yearly BOD_5 loadings estimated in studies of urban runoff and combined overflows in various cities are shown in Table 2-6. These values show that in general more oxygen-demanding organic materials are contributed per unit area by combined sewers than by separate sewers. This is due to the highly organic content of sanitary sewage. An urban waterway with an especially critical oxygen balance could therefore benefit if conversion from a combined to a separate sewer system were to occur.

In considering the relative merits of combined and separate systems, it should be noted that combined sewers actually protect urban waterways from certain kinds of pollution sources. Any waste discharged into a storm drain in a separate system will go directly to the river, whereas such material would have a good chance of passing through a treatment plant in the case of a combined system, particularly if the discharge occurs during dry weather. The discharges discussed here are, of course, illegal in nature and might include such damaging materials as crankcase oil, solvents, or highly toxic industrial wastes of one form or another. A realistic comparison of the two types of sewer systems would have to account for such irresponsibilities on the part of the public or industry.

In summary, it can be said that the pollutional threat imposed by urban storm-water runoff and combined sewer over-

TABLE 2-6

5-Day BOD Loading Factors for Combined Overflows and Separate Storm Sewers

Type of System	City	Loading Factor[a] (lbs. BOD_5/acre-year)
Combined	San Francisco, Cal. (22)	101 136
"	Detroit, Mich. (29)	360
"	Philadelphia, Pa. (24)	143 555
"	Bucyrus, Ohio (27)	222
Separate	Cincinnati, Ohio (20)	38
"	Ann Arbor, Mich. (29)	124
"	Tulsa, Okla. (28)	12-48[b]

a - total quantity of 5-day BOD escaping in combined overflow or discharged through a storm drain per acre of sewered area per year.
b - range of 15 separate areas studied

flow is in many ways difficult to express in definitive, quantitative terms. It depends, at one end, on the sanitary conditions of a city, and, at the other end, on the dilution and assimilation capacities of a receiving waterway. Pollution-conscious citizens and industries, along with efficient urban housekeeping, can help reduce the water pollution problems that are caused by these sources. From a health standpoint, combined sewer overflows generally represent a more severe menace than discharges from separate systems. An urban river subject to combined overflows and without sufficient dilution capacity will probably never be safe for swimming because of the danger of bacterial or viral contamination. Combined overflows and urban runoff more closely resemble each other in chemical and physical makeup than in bacterial. More information is needed about the possible harmful effects of some of the artificial, "man-made" materials associated with the city and carried in runoff to urban waterways, where they are found in trace or more abundant quantities.

2.3.2 Sewage Systems Contributing to the Basin

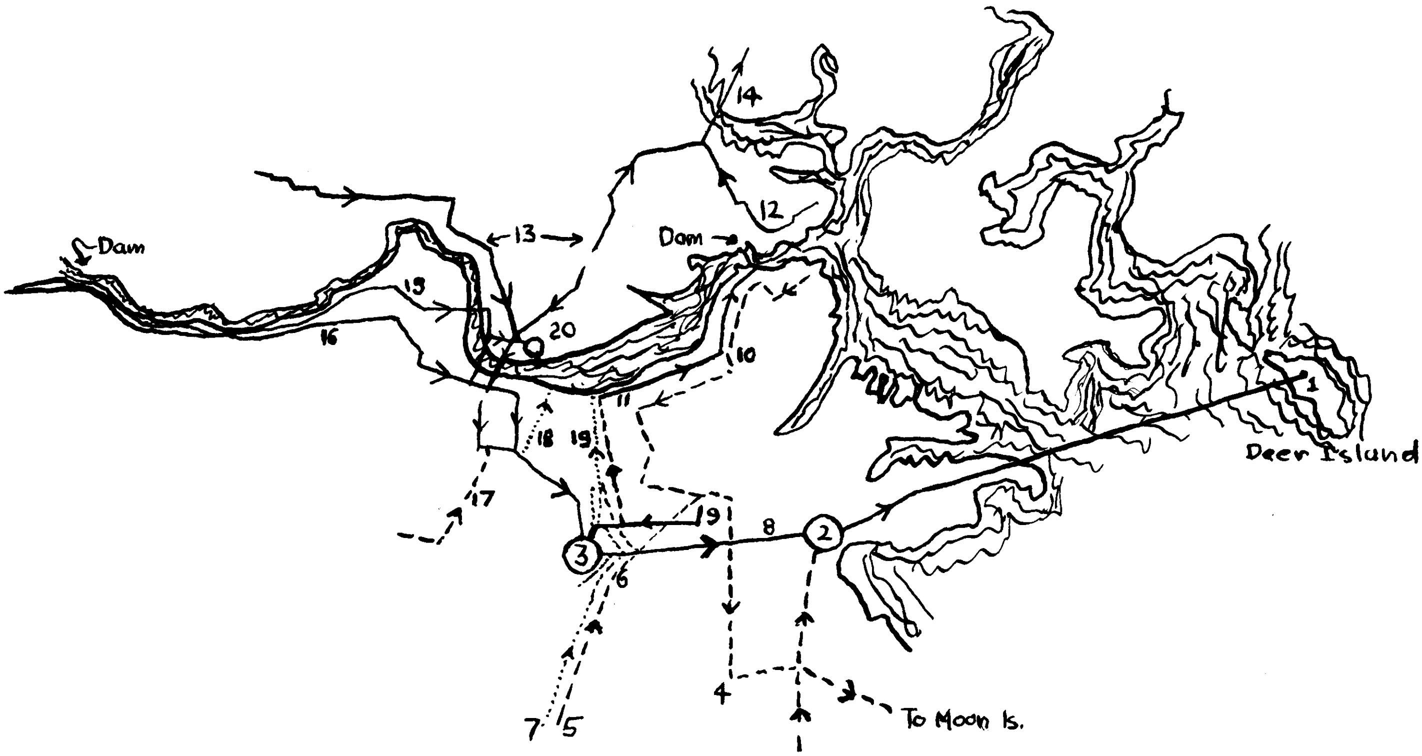
The highly populated area which drains into the Lower Charles River is a model example of an urban area in which combined sewer overflows and storm-water runoff create definite pollution problems. These represent the only known appreciable pollution sources in the Lower Charles, which, because of its low dilution capacity, has little resistance to them. In the interest of resurrecting this relatively dead body of water, many alternatives have been proposed. Some of them have even been adopted. An adequate understanding of the problem, at least to the limits of our present ability, is necessary before the optimum abatement steps can be selected. Such an understanding can be obtained in part from a quantitative demonstration of the relationship between pollution sources and observed water quality. Unfortunately, the pollution sources in the Lower Charles have not been sufficiently quantified. Before considering in detail how these sources might be measured, a general description of the sewer systems contributing to the Lower Charles is necessary.

The treatment of wastes in the Lower Charles Watershed is regionalized under the auspices of the Metropolitan District Commission. The M.D.C. maintains a system of major interceptors which lead to a primary treatment plant on Deer Island in Boston Harbor. Each individual city or town has the responsibility of maintaining the storm and sanitary sewage collection facilities within its borders. The inputs to the M.D.C. interceptors are of four basic varieties: (a) sanitary sewage from those areas with separate sewer systems, (b) combined storm and sanitary sewage from those areas with combined systems, (c) storm runoff from those areas with separate storm-water drainage systems which lead to these interceptors instead of the river, and (d) infiltration. Because of the system's age, the increasing population in the area, and the addition of towns served by the M.D.C., the collection system is, as a whole, overburdened. A general description of the components of the system will be followed by a discussion of the individual city sewers and then by a discussion of how the entire situation relates to the Lower Charles.

There are three major segments of the M.D.C. collection system of relevance to the Charles. The South Charles system consists of the Charles River Valley Sewer and the South Charles Relief Sewer, which run along the south bank of the Charles to the Ward Street Headworks. The Cambridge Branch handles the flow from the north banks of the Charles, bringing part of it to the Ward Street Headworks and part to the Charlestown Pumping Station. The M.D.C.Marginal Conduit discharges wet-weather flows from the Stony Brook Valley Sewer and the West Side Interceptor into the Charles above and below the dam. The dry-weather flow from these two sewers is intercepted and carried to Ward Street. Both the Charlestown and the Ward Street stations pump the sewage to Deer Island. Figure 2-3 is a map of the major sewers and storm

FIGURE 2-3

Sewers Affecting Charles River Water Quality[a]



1 Deer Island Sewage Treatment Plant
2 Columbus Park Headworks
3 Ward Street Headworks
4 Boston Main Interceptor
5 Stony Brook Valley Sewer
6 Stony Brook Valley Interceptor
7 Stony Brook Conduit
8 Boston Main Drainage Tunnel
9 Interceptor
10 West Side Interceptor
11 M.D.C. Marginal Conduit
12 Charlestown Branch Sewer
13 Cambridge Branch Sewer
14 Charlestown Pumping Station
15 South Charles Relief Sewer
16 Charles River Valley Sewer
17 Brookline Sewer
18 Muddy River Conduit
19 Back Bay Fens
20 M.D.C. Chlorinated Overflow

LEGEND

M.D.C. Sewer
City or Town Sewer
City or Town Storm Drain
Flow Direction

a - map quoted from reference (1)

drains in the area, as described in the March, 1971 report on the Charles issued by the Massacusetts Water Resources Commission, Division of Water Pollution Control (1).

Table 2-7 shows how heavily loaded these sewers are by comparing their estimated capacities with average dry-weather flows. These estimates were made by Mr. William Butler of the Federal Water Quality Administration, Needham Heights. They are based on information from a report by Charles A. Maguire Associates on the Boston area sewage disposal system (30). As previously noted, combined sewers are generally designed to handle about five times their average dry-weather flow. Only the relatively new South Charles System has a capacity to dry-weather flow ratio which approaches this value. Based on these figures alone it is anticipated that the quantities of material escaping to the Charles in combined overflows would be relatively great.

The M.D.C. has a continuing program to improve the collection system and reduce the pollutional threat it imposes on the Charles. Before considering the characteristics of the individual sewer systems contributing to the M.D.C. framework, it would be of interest to describe what this program has accomplished and hopes to accomplish.

The effort has been concentrated on the area above the Boston University Bridge. In 1967 the South Charles Relief Sewer was activated. This stopped the continuous discharge of an estimated two million gallons per day of raw sewage

TABLE 2-7

Capacities and Average Dry-Weather Flows of Major M.D.C. Sewers

Sewer	Estimated[a] Capacity (mgd)	Estimated[b] Dry-Weather Flow (mgd)	Capacity / Dry-Weather Flow
South Charles System	135.4[c]	30	4.5
	28.[d]	30	.9
Cambridge Branch[e]	19.6	12.8	1.5
M.D.C. Marginal Conduit	0-140[f]	-	-
Stony Brook Valley Sewer	48[g]	13.2	3.6
	113[h]	13.2	8.6
West Side Interceptor	33	17.5	1.9

a - (32)
b - 1967 conditions (32)
c - after activation of South Charles Relief Sewer, Aug., 1967
d - before " " " " " " " "
e - at the B.U. Bridge
f - accepts overflows from Stony Brook Valley Sewer and West Side Interceptor and discharges to tidal portion of river; capacity depends on tide
g - to produce overflows into the M.D.C. Marginal Conduit
h - to produce overflows into the Fens

into the Charles from the Charles River Valley Sewer. The relief sewer is designed to allow overflows from the south bank of the Charles above the B.U.Bridge only once every five years. There are plans for construction of a North Charles Relief Sewer which would intercept flows from the Cambridge Branch sewer above the B.U.Bridge. The new storm-water detention and chlorination station at the B.U.Bridge is presently undergoing tests. This facility is designed to accept storm flows from the South Charles Relief Sewer, the proposed North Charles Relief Sewer, and overflows from the Brookline Sewer. Partial removal of solids and floating materials in addition to chlorination will help reduce the potency of storm-water discharged into the Charles at the B.U.Bridge. Once this facility is in full operation, combined overflows above the B.U.Bridge will be reduced to a five-year frequency.

Plans for the abatement of overflows from the areas below the B.U.Bridge are perhaps less encouraging. The relatively recent activation of an interceptor to take the dry-weather flow from the West Side Interceptor and the Charles River Valley Sewer to Ward Street stopped the continuous flow of raw sewage from these sewers into the M.D.C.Marginal Conduit and thence into the Charles. This supposedly also reduced the quantity of overflows from these sewers into the Back Bay Fens. The M.D.C. Conduit is a serious problem. It is essentially flat and therefore subject to the accumulation of solids. During high tides, it becomes surcharged with sea water and therefore has essentially zero capacity. The sanitary wastes which it accepts from areas of Back Bay and Beacon Hill (14) are often discharged directly into the river above the dam, along with any overflow from the West Side Interceptor and with wet weather flow from the Stony Brook Valley Sewer. The construction of the new dam at Warren Avenue will necessitate a change in the M.D.C. Marginal Conduit. The M.D.C. has plans to install a new pumping station at the present site of the tide-water discharge from this conduit. This station would prevent the intrusion of sea water into the line and allow maximum discharge at all tides into the river below the Warren Avenue dam site.This will supposedly eliminate overflows from the conduit into the basin. However, the many connections between the West Side Interceptor and the M.D.C. Marginal Conduit may introduce enough storm-water to cause overflows into the basin in spite of the new pumping station. It is also doubtful that the new station will have enough effect on the overflows from the Stony Brook Valley Sewer into the Fens, which occur when the M.D.C. Marginal Conduit has reached capacity. The undesirable effects of sewage discharges into the tidal reach must also be considered in evaluating this plan, especially with the increasing interest in upgrading the water quality in Boston Harbor.

Some of the existing problems are related to the condition of the M.D.C. system, in addition to its capacity. During 1969 the average influent at Deer Island was 279 mgd (31).

From its chloride content, it was estimated that about 24% of this measured flow was sea water which had infiltrated the system as a result of inoperative or borken-down tide gates. At one location, the Charles River has been observed to flow into the sewer system during dry weather, apparently through a malfunctioning overflow device (32). If water can infiltrate the system,it might be expected that significant quantities of sewage may be running into the Charles continuously during dry weather. This problem is a result of the system's condition rather than its capacity, and is therefore relatively unecessary. The M.D.C.and the City of Boston have started a program to repair and/or install tide gates that influence Deer Island sewage flows. In 1970 a reduction in flow at Deer Island occurred as a result of this program (31). A thorough examination of the tide gates and overflow devices which might be causing continuous discharge into the Charles should be undertaken.

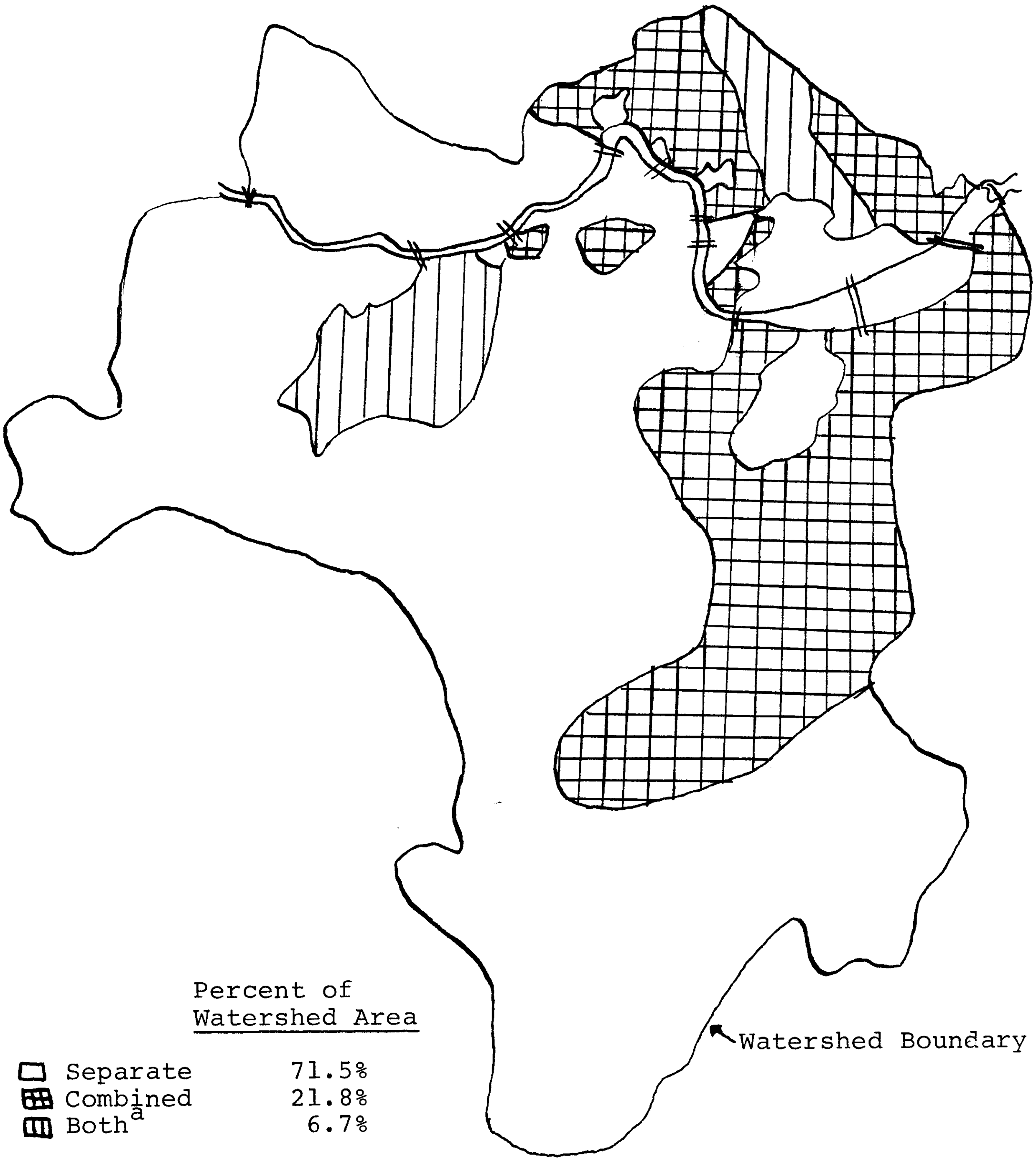
The sewage and storm-water which enters the M.D.C. system originates in the individual cities and towns. Overflows from the M.D.C system are not so much the M.D.C. 's fault as they are the fault of those cities with combined systems. Boston, Cambridge, Brookline, and Somerville are among them.

The job of separating the combined areas in these cities is an ambitious, time-consuming, and expensive one. Cambridge has a five-year plan to separate its combined areas, as recommended by Maguire Associates (33). Cambridge has not yet initiated the program, however, despite the fact that the city is under implementation by the state to do so. Brookline is in the process of completely separating its sewers, as recommended by Mecalf and Eddy (34). The cost of separating the Boston system was found to prohibitively expensive by Camp, Dresser, and McKee (35), who recommended the Deep Tunnel Storage Plan as an alternative to solving the storm sewage disposal problems from combined areas in Boston and the surrounding area. The cost of this plan also appears to be quite prohibitive. The City of Boston presently has no definite plans for abatement of the problems which its combined sewer areas pose for the M.D.C system and for the Charles River.

Figure 2-4 is a map compiled from information in the Maguire Report to Cambridge (33),the Metcalf and Eddy report to Brookline (34), and the Camp, Dresser, and McKee report to Boston (35). This map show the extent of sewer separation in the areas which drain into the Charles River Basin. Of the total drainage area of 39.3 square miles, approximately 71.5% is served by spearate systems, 21.8% by combined systmes, and 6.7% by separate systems which discharge stormwater into an M.D.C. main instead of the river. The approximate areas were determined with a planimeter. Areas within the third category have the same effect as combined areas in causing overflows from the M.D.C system. Alleviating the storm loads from these areas could be accomplished with relatively little expense, since it would probably only involve the installation of lines to the river from the present points of discharge of these storm sewers into the M.D.C. conduits.

FIGURE 2-4

Extent of Sewer Separation in Areas Draining into the Charles River Basin[b]



	Percent of Watershed Area
Separate	71.5%
Combined	21.8%
Both[a]	6.7%

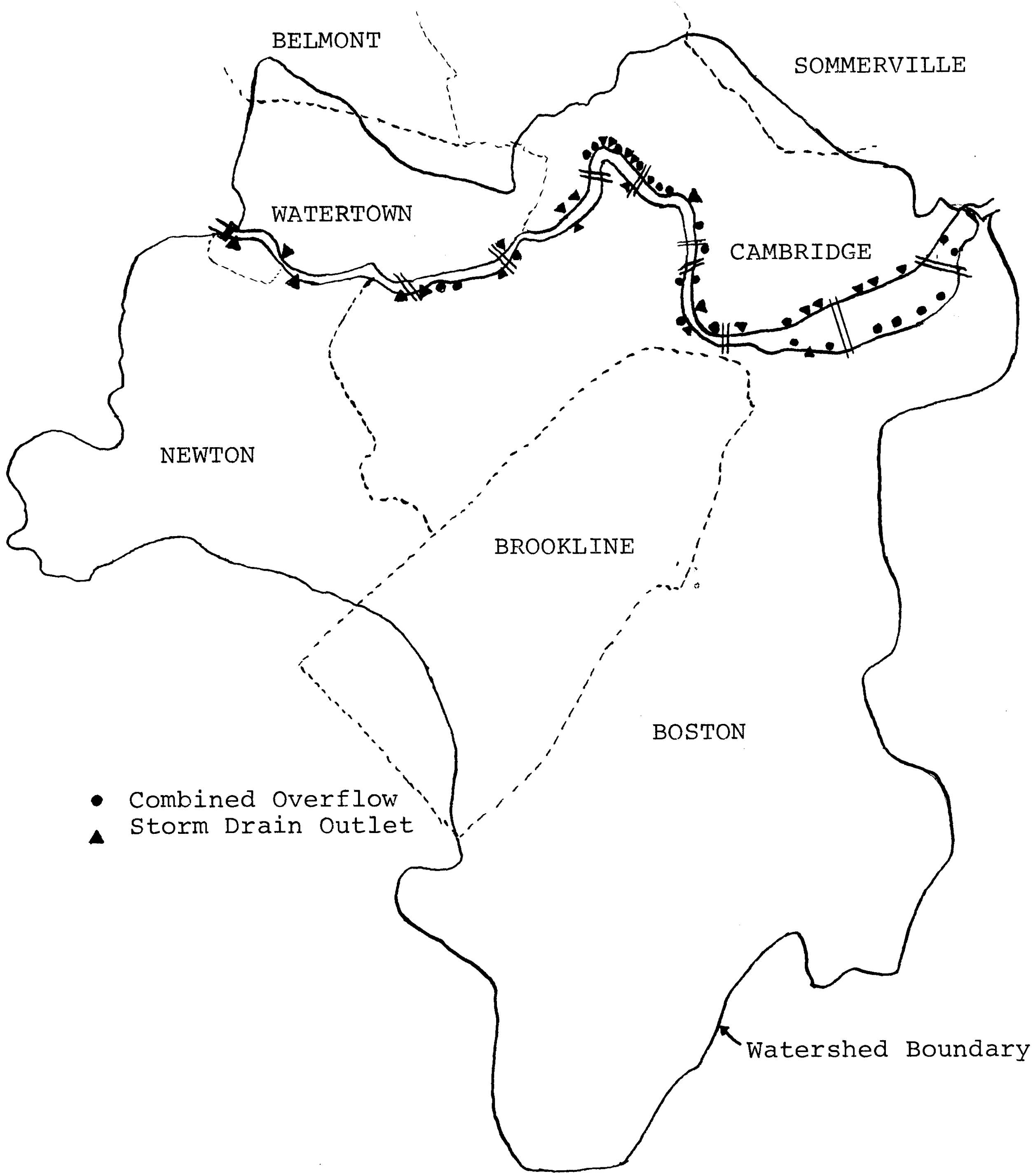
a - separate areas discharging storm-water into major M.D.C. Interceptors

b - compiled from information in references (33),(34),(35) The possibility of continuous discharges from these city systems into the Charles cannot be ignored. Maguire (33) estimates that about 1.76 mgd are continuously discharged into the Charles from the Cambridge system through broken-down overflow gates and cracked lines. This is the only documented evidence of such occurrences under the present conditions, though there is a good possibility that other cities may also contribute in a similar manner. Illegal sanitary connections to storm drains may constitute some dry-weather sources. The cross connections between the Stony Brook Valley Sewer and the Stony Brook Conduit, a storm drain, may be causing continuous discharges into the Back Bay Fens (32). Information about possible continuous waste loads from boathouses and other buildings situated directly on the banks of the river should be gathered.

Figure 2-5 is a map showing the approximate location of storm drain outlets and combined sewer overflows in the Charles River Basin. This was compiled from maps obtained from the M.D.C., the Camp, Dresser, and McKee report (35), and the Maguire report (33). The locations shown are only approximate and there is no guarantee that all of the existing discharge cites are shown, or that all of those shown actually exist. Most of the combined overflows lead to off-shore, sub-surface discharge locations; they are relatively inaccesible to inspection. Many may be inoperative due to sediment accumulation.

FIGURE 2-5

Location of Storm Drain and Combined Overflow Outlets[a]



a - compiled from information in references (33),(34),(35)

2.3.3 Methods of Source Estimation

The sanitary and storm-water collection systems described above represent the pollution sources of the basin. In the interest of developing an adequate description of the problems of the Charles, a more quantitative definition of these sources in needed. Following is a description of how these sources might be estimated. There are three fundamental approaches to the problem.

The first approach involves taking measurements of the sources themselves. This would mean locating, sampling, and analyzing all continuous and storm-dependent sources. This would obviously be a formidable task, due to the great numbers of measurements that would have to be made of the relatively inaccessible overflows and storm drains. One could never be sure that all of the sources were being taken into account. Despite the difficulties, an intelligent sampling program could minimize the number of necessary measurments. An approach of this kind, if properly executed, could yield probably the most concrete estimate of the sources, though it would be a most time consuming method. In conjunction with the evaluation of the new storm-water detention and chlorination station, the M.D.C. has a continuing sampling program which involves overflow sampling, but it is not of the scale necessary to provide estimates of total pollutant loadings to the basin (4).

The second general approach to the problem of quantifying sources would be to examine the sewer end of the sewer-

Charles River interface. This would first involve estimating or measuring the capacities, dry-weather flows, and dry-weather pollutant concentrations of the contributing combined sewer systems. The areas and runoff coefficients for the urban areas contributing storm-water to each system would be estimated. The runoff coefficient of a given area is defined as the fraction of the rainwater falling on the area which reaches the system. An estimate or measurement would be made of the pollutant concentrations in runoff. One could then examine mathematically the response of the sewer system to a rainstorm of a given intensity and duration. The flows and concentrations of overflows would thus be calculated. A calculation of this type was done by Mr. William Butler of the Federal Water Quality Administration for the three major M.D.C. systems contributing to the Charles. These calculations estimated the total BOD escaping in overflows for the period July-August, 1967. They represent the only real effort made to date on quantifying Charles River pollutant sources. These calculations are quite useful in comparing the contributions of the three systems and in demonstrating the relationships between dry-weather flow, interceptor capacity, and overflow quantity and quality. The approach does not account for all of the sources. The sedimentation of solids within the systems during dry weather is difficult to estimate and therefore ignored by these calculations. Any continuous or storm-dependent sources outside the M.D.C. system are also neglected.

The third general approach to the problem of source estimation would almost necessarily take into account all of the pollutant sources. This method would involve connecting observed water quality in the river itself to these sources with a mathematical model which would take into account pollutant sources and sinks. With sufficient input data, a model of this sort could be used to indicate where and when the most significant problems are. The fundamental concept behind this approach is the material balance. A partial test of the validity of such a model would be to examine the source quantities calculated for a given section of the river and to see how well they correlate with the characteristics of the sewage systems contributing to that section. Calculated sources should also reflect changes in the sewer systems with time, such as the activation of a new major relief sewer. The remainder of this work will be concerned with the development, application, and verification of such a model. The information obtained from the model will then be used to evaluate proposals for pollution abatement in the Charles.

3. Development of the Model

The model may be described as a segmented, mass-balance model which accepts as input concentrations of biochemical oxygen demand (BOD) measured as locations in the Lower Charles. These concentrations are used to estimate carbonaceous BOD sources in various segments of the river. Steady-state conditions are assumed. Hydrologically, the basin is assumed to consist of two vertical layers: an upper, vertically mixed, aerobic portion, comprising all depths up to twelve feet, and a lower, stagnant, and anaerobic portion. The upper region is modelled as a plug flow reactor completely mixed in its cross-section. The lower layer is treated essentially as if it were the river bottom, subject to accumulation and loss of organic materials. The BOD sources contributing to the upper layer are assumed to be distributed uniformly along the length of each river segment. The rate of destruction of BOD within the upper layer is assumed to follow a first order reaction. Within the model itself, no distinction is made between BOD contributed directly from the sewers and that contributed indirectly, i.e., from the bottom deposits and the anaerobic portions of the river. The calculated source values also include decaying algae.

The model is basically a simple one, founded on the basis of observation, intuition, and reason. It is not necessarily the only plausible model for this particular situation. It satisfies the basic criterion that all of its parameters have physical as well as mathematical meaning. Under the general framework presented above, the model will be developed by first justifying its focus on the BOD parameter, by examining each built-in assumption and the evidence supporting it, by presenting the derived equations and estimating techniques, and finally by discussing possible applications.

3.1 The BOD Parameter

Some consideration must be given to the choice of BOD as the focus of the model. The dissolved oxygen concentrations measured in the surface layer of the basin have been generally high. Thus, the oxygen balance in this portion of the river does not appear to be very critical. BOD, in itself, is not a particularly harmful pollutant unless the oxygen balance is critical. It would seem, then, that a model focusing on this parameter would not be especially relevant to the river's problems. Some justification for the choice of this particular parameter is therefore in order.

First of all, it must be considered that the primary objective of the proposed model is to estimate pollutant source quantities and to indicate where and when the most significant sources occur. Even though BOD may not be particularly critical in the basin, calculated source quantities would still be useful for comparing the contributions of various sewer systems to the river. Generally, sources contributing excessive amount of BOD would also be expected to contribute excessive amounts of other pollutants which might have a more detrimental effect on water quality in the basin. BOD may be used as a yardstick to estimate quantities of these other materials, which may include suspended solids, coliform bacteria, and nutrients.

The second justification for the choice of the BOD parameter is that it is a relatively easy one to model. Equations describing the rate of BOD assimilation under aerobic conditions have been formulated and generally accepted. This cannot be said of other water quality parameters. The assimilation of BOD is considered to occur in two stages: a first stage which deals with the oxidation of carbonaceous materials and a second stage which deals with the oxidation of nitrogenous materials. These stages generally sequentially, the nitrogenous demand not being exerted until the carbonaceous demand is satisfied. Within the basin, the nitrogenous stage is probably never reached because there are carbonaceous sources distributed along the entire length. Only the carbonaceous BOD is considered in the model. The most widely used form describing the rate of assimilation of carbonaceous BOD is a simple, first order reaction. The relative ease with which this form can be handled is an obvious advantage.

A third reason for the concern with the BOD parameter is that there is a relatively large accumulation of data on its concentrations in the basin. The M.D.C. (3) has taken measurements of BOD at several locations over the past five years. Since this parameter is a relatively easy one to measure, sampling programs designed specifically to gather data for this model would involve a minimal amount of effort.

BOD is determined by measuring the change in dissolved oxygen of an isolated sample aver a period of time, typically five days. The standard test is carried out at 20^{o}C and in the dark, to eliminate algal interference. The oxygen initially present in the sample is utilized in the biodegradation of the organic materials. By measuring the dissolved oxygen concentration at two different times after the start of the test, the test may be used to determine both the total quantity of carbonaceous BOD in the sample and the characteristic rate at which it decays. This is demonstrated by the rate equations used to describe oxygen consumption in a river or waste sample.

The rate of oxygen consumption during the carbonaceous stage is described by the following first order relation:

$$\frac{dC_t}{dt} = \frac{dO_t}{dt} = -k\ C_t \qquad (1)$$

where O_t represents the concentration of dissolved oxygen in the sample after time t and C_t is the total concentration of carbonaceous BOD remaining after time t. The basic assumption is that the rate of consumption of oxygen is directly proportional to the total concentration of organic materials in the sample and independent of the concentration of dissolved oxygen, as long as the D.O. concentration is above 1 mg/liter, compared with a saturation value of about 9 mg/liter at 20°C. The boundary conditions imposed on equation (1) are at t = 0,

$O_t = O_s$ = the saturation concentration of dissolved oxygen

$C_t = C_o$ = the total initial concentration of carbonaceous BOD.

Substitution of the equality:

$$0_s - 0_t = C_o - C_t \tag{2}$$

and integration of equation of equation (1) yields:

$$\Delta 0_t = 0_s - 0_t = C_o(1 - e^{-kt}) \tag{3}$$

where $\Delta 0_t$ is the measured decrease in dissolved oxygen concentration after time t. There are two unknowns in equation (3), C_o and k. In order to determine both, measurements have to be taken after two time periods, t_1 and t_2. C_o and k are then fixed by the equations:

$$\frac{\Delta 0_{t_1}}{\Delta 0_{t_2}} = \frac{1 - e^{-kt_1}}{1 - e^{-kt_2}} \tag{4}$$

$$C_o = \frac{\Delta 0_{t_1}}{1 - e^{-kt_1}} \tag{5}$$

Equation (4) may be solved for k by iteration and equation (5) for C_o by substitution of the determined value of k.

C_o is referred to as the total or ultimate carbonaceous BOD. It represents the total amount or oxygen required to oxidize all of the biodegradable carbonaceous materials in a unit volume of sample. This value is temperature independent.

Rate constants vary with the characteristics of the organic materials in the sample. At $20^{\circ}C$, typical values of k for river water fall in the range .20 to .35 days^{-1}. Empirical equations have been developed to describe the change in k with temperature. Many forms have been proposed. Kittrell (11) suggests:

$$\frac{k_T}{k_{20}} = 1.0241^{(T - 20)} \qquad (6)$$

The temperature T is measured in degrees C. This amounts to a 2.41% change in oxygen consumption rates for every degree Centigrade.

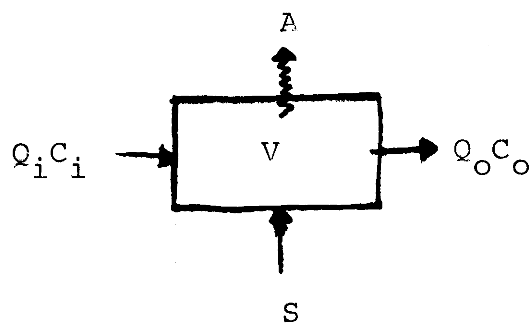
Many other forms have been proposed to represent oxygen consumption in a sample containing organic materials. The fundamental equations presented above are the most widely accepted and will be used in the development of this model.

3.2 The Material Balance

The fundamental concept behind the model is that of the material balance. The river is divided into a series of segments, each bounded upstream and downstream by a water quality monitoring station. For each segment, the control volume is defined as the aerobic portion of the river, i.e., the portion available for BOD assimilation. Figure 3-1 is a schematic representation of the control volume.

FIGURE 3-1

The Material Balance



The material balance equation, as applied here, is:

$$
\begin{aligned}
\text{Input} - \text{Output} &= \text{Accumulation} \qquad (7)\\
\text{Input} &= Q_iC_i + S\\
\text{Output} &= Q_oC_o + A\\
\text{Accumulation} &= \frac{d(V\bar{C})}{dt}
\end{aligned}
$$

Q_i, Q_o = water flow into and out of segment

C_i, C_o = carbonaceous BOD concentrations measured at upstream and downstream monitoring stations

S = unknown source rate of carbonaceous BOD

A = rate of BOD assimilation within segment

V = effective segment volume, in which the concentration of D.O. is >1 mg/liter

$\bar{C}$ = average concentration of BOD in segment; determined from inlet and outlet concentrations

t = time, in days

In order to evaluate S, A, and $\overline{C}$, some assumptions about the physical situation have to be made. The river will be modelled as a plug flow reactor, in which a first order reaction, corresponding to the assimilation of BOD, is occuring. The sources of BOD are assumed to be uniformly distributed along the length of the reactor. The solution to this problem will be presented after an examination of the basic simplifying assumptions.

3.3 Simplifying Assumptions

Within the framework of any model, assumptions have to be made to simplify the problem. It is often difficult to predict a priori the consequences of any particular simplifying assumption. Models are commonly developed by trial and error - comparing calculated results with anticipated or known values and making changes in the model to account for any severe discrepancies. In the end, the extent to which the problem is simplified depends upon the desired level of accuracy.

In the course of developing a model, in order to eliminate unnecessary complication, it is best to start with a relatively simple form. The form and accuracy of the data available for use in the model should also be considered, since there is little point in developing a model which requires data beyond the scope of that available.

This model, in its present form, is a relatively simple one. It will be used as a tool to estimate rather than to pin-point. Of more importance that the absolute magnitudes of the estimated sources are the comparisons that will be made among them. The model was also developed for application to existing data, the extent and characteristics of which do not justify a model of any higher degree of sophistication. The three basic simplifying assumptions made within the framework of the model are plug flow, uniform source distribution along each segment, and constant control volume. Evidence supporting each assumption will be presented.

Generally, a river mixes vertically and laterally before it mixes longitudinally. Vertical mixing in the region of active flow is induced by the rolling flow pattern of the water. The path of a given volume of water moving downstream has been described as that of a section on the rim of a rolling wheel (11). Dispersion and bends in the course of a river tend to enhance lateral mixing (11). Axial dispersion, or longitudinal mixing, would have to be considered in cases where the concentration verses length profile is being followed downstream with time after a pulse input of pollutants. The effect of axial dispersion would be a smoothing out of the input pulse as it travels downstream. In the present case, axial dispersion is ignored.

The plug flow assumption amounts to postulating that the concentrations of BOD are more likely to vary along the length of the river than within the cross-section at a given location. It must be remembered that the assumption only refers to the upper, aerobic layer of the basin. This assumption must be examined in a probablistic manner.

Consider one sampling program in which samples are taken at a number of spots within the cross-section of the river at a given bridge and another program in which one sample is taken at each of several bridges. On any given day, one might find that the concentrations seem to vary both within the given cross-section and along the length of the river. However, if samples are taken on a sufficient number of days, and if the river exhibits plug flow behavior, only the measurements along the length would show a consistent pattern.

This is illustrated by a statistical treatment of data taken by the M.D.C. in each of two sampling programs (3,4): one involving single surface samples at seven bridges, and the other, cross-sectional samples at three bridges. The data on 5-day BOD concentrations from these programs will be examined with the help of the "Student t-Test", a statistical test commonly used to examine the significance of any observed difference between two sampled populations. Given data taken at any two locations on several days, the t-test is used to determine whether the difference between the two data sets is a result of random fluctuations due to sampling or analytical problems, or whether, in fact, the difference is a result of sampling from two significantly different populations. The t-test is used to compare two sets of data at a time, and, in this application, essentially tests the hypothesis that the difference between the two locations on each day is zero. The parameters calculated by the test allow the use of statistical tables to determine the significance level, α. An α of .9 indicates that the difference between the two sets of data is greater than would be expected by chance 90% of the time if the sets were taken from the same population.

Table 3-1 shows the significance levels calculated from the M.D.C. data. The computations were done on an IBM 1130 computer with the aid of programs contained in the 1130 Scien--

TABLE 3-1

Significance Levels[a] of Differences in BOD_5 Concentrations Measured at Neighboring Locations

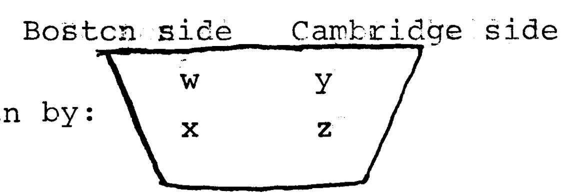
Length-wise Distribution : 47 Data Sets[b]

Location	Significance Level <	α <
Watertown Dam		
	.95	.975
No.Beacon Street Bridge		
	.975	.99
Eliot Bridge		
	.975	.99
Western Avenue Bridge		
	.975	.99
B.U.Bridge		
	.99	.995
Harvard Bridge		
	-	.75
Charles River Dam (upper)		

Cross-sectional Distribution : 9 Data Sets[c]

Location[d]	Harvard Bridge <	α <	B.U. Bridge <	α <	Western Avenue Bridge <	α <
wx	-	.75	-	.75	.995	.9995
wy	.90	.95	.75	.90	.75	.90
wz	-	.75	.75	.90	.995	.9995
xy	.75	.90	-	.75	-	.75
xz	-	.75	-	.75	.95	.975
yz	-	.75	.75	.90	-	.75

a - from tables in Brunk (36)
b - M.D.C. (3)
c - M.D.C. (4)
d - location in cross-section given by:



tific Subroutine Package. The table shows that five out of six determined significance levels for the lengthwise distribution of samples were greater than .95, as compared with three out of eighteen for the cross-sectional distribution. The plug flow assumption is by no means perfect, as indicated by the Western Avenue data. The cross-sectional differences indicated at the Western Avenue Bridge may be due to the presence of a significant source of BOD directly under or immediately upstream of that bridge. The proximity of the source would not allow sufficient time for cross-sectional mixing. The plug flow assumption between the Harvard Bridge and the Charles River Dam does not appear to be valid. This is what would be expected, in consideration of the back-mixing which probably occurs as a result of the dam's opening and closing and the wind sweeping over this portion, as it has a relatively high surface area compared to the other segments. The apparent difference in significance levels between the length-wise and cross-sectional distributions cannot be attributed to the difference in the number of samples, since the test itself takes this into account.

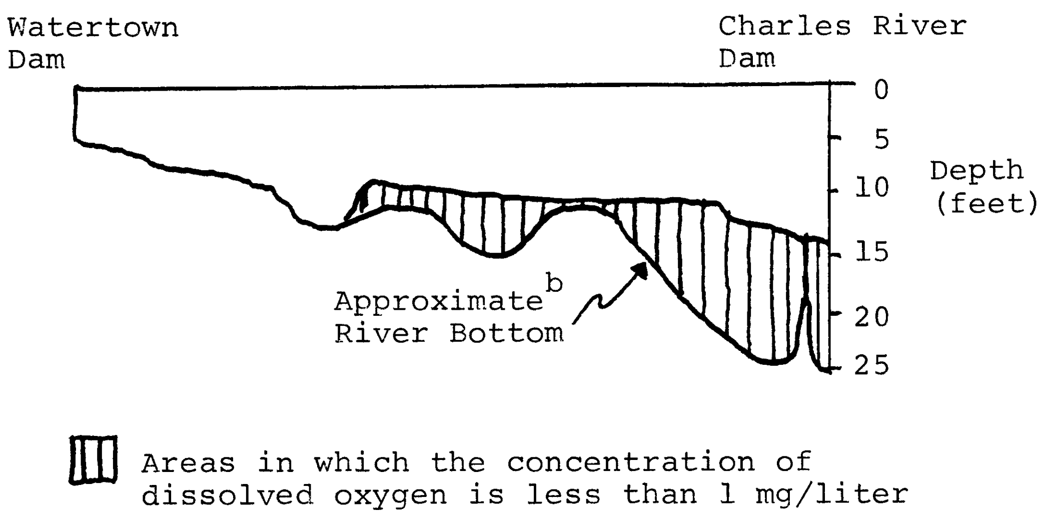
On the basis of the above evidence, the assumption of plug flow with cross-sectional mixing will be incorporated into the model. As has been demonstrated, it is not perfect, but it seems to be reasonable and sufficient for the prescribed purposes of the model. The cross-sectional mixing assumption was made on the basis of probablistic considerations. This incorporates into the model one very important point: it is a probablistic model, as opposed to a deterministic one. In other words, the results calculated by the model from data taken on any given day will mean little. Data taken on many days will be needed to "iron out" fluctuations which would result from such assumptions as cross-sectional mixing. Statistical techniques will have to be applied to the results to determine their internal consistency and significance.

The second fundamental assumption incorporated into the model is that the sources are uniformly distributed along the length of each segment. Figure 2-5 shows the relatively even distribution of overflow points and storm drains. The other direct sources of BOD, namely bottom deposits and algae, are characterized by even distribution. This assumption would appear to be the most reasonable one to make about source distribution. Significant errors would only be introduced in cases where a sampling station is located in the immediate vacinity of a large overflow or continuous sources. Highly erratic data from any one station would tend to indicate such a situation.

The final assumption which merits consideration is that of constant control volume. The aerobic portion of the basin is assumed to include all the water at depths up to twelve feet. This was made primarily on the basis of Figure 3-2, a plot of the variation of the average dissolved oxygen concentrations measured by Process Research (2) during the summer of 1969. The interface between the anaerobic and the aerobic layers of the basin was located between ten and fifteen feet in depth. As was discussed in Chapter 2, there is no evidence that the bottom layer remains anaerobic throughout the year, though there is evidence that some of the salt, which is instrumental in causing the anaerobic conditions, does remain. The upper bound of the aerobic volume is assumed to be constant because the M.D.C. operates the dam to maintain this level at 2.38 feet above mean sea level.

FIGURE 3-2

Anaerobic Volume of the Basin[a]



a - summer of 1969, Process Research, Inc. (2)
b - at deepest point across the river

3.4 Mathematical Solution of the Model

The assumptions discussed in the previous section make possible the evaluation of the material balance terms through the solution of a differential equation describing the situation. The following terms are defined:

A_c	=	cross-sectional area of segment, perpendicular to flow direction (ft^2)
L	=	total length of segment (mi.)
k	=	BOD rate constant at river temperature ($days^{-1}$)
Q_x	=	total water flow in the river at any point x along the length of the segment (cfs)
q	=	water flow contributed by source, per unit length (cfs/mi)
C_s	=	concentration of BOD in source (mg/liter)
C_x	=	concentration of BOD in the river at any point x (mg/liter)
i	=	subscript indicating inlet values, at x = 0
o	=	subscript indicating outlet values, at x = L

The following differential equation describing the model may be derived for the steady-state case and in a length element dx:

$$\frac{d}{dx}(C_x) + \frac{(q + kA_c)}{Q_x} C_x - \frac{q\ C_s}{Q_x} = 0 \qquad (8)$$

The water flow Q_x at any point x is given by:

$$Q_x = Q_i + qx \qquad (9)$$

Substitution of equation (9) into equation (8) and manipulation yields:

$$(1 + \frac{qx}{Q_i}) \frac{d}{dx}(C_x) + \frac{(q + kA_c)}{Q_i} C_x - \frac{q\ C_s}{Q_i} = 0 \quad (10)$$

The solution of the above equation for $C = C_i$ at $x = o$ is:

$$C_x = \frac{q\ C_s}{q + kA_c} - \left(\frac{q\ C_s}{q + kA_c} - C_i\right)\left((1 + \frac{qx}{Q_i})^{-(1 + \frac{kA_c}{Q_i})}\right) \quad (11)$$

Given concentrations C_i and C_o and water flows Q_i and Q_o at $x = 0$ and L, respectively, the above equation may be solved for the total steady-steady state source rate, S, in any segment:

$$S = q\ C_s\ L$$

$$= \frac{(Q_o - Q_i)\ \gamma\ (C_o - (\frac{Q_o}{Q_i})^{-\gamma}\ C_i)}{1 - (\frac{Q_o}{Q_i})^{-\gamma}} \quad (12)$$

$$\gamma = 1 + \frac{k\ V}{Q_o - Q_i}$$

The average concentration $\bar{C}$ within the segment may be evaluated as:

$$\bar{C} = \frac{1}{L} \int_0^L C_x\ dx \quad (13)$$

Integration of equation (13) and substitution of expressions for C_o and S yields:

$$\bar{C} = \frac{S + Q_i C_i - Q_o C_o}{k\ V} \quad (14)$$

Equation (14) is equivalent to the overall steady-state material balance equation (7), with the assimilation term,

$A = k\,\bar{C}\,V$. Equation (12) represents the solution of the mathematical problem defined by the model. Before it can be applied, some consideration must be given to the estimation of some of the parameters, namely segment volumes, water flow, and rate constants.

3.5 Evaluation of Parameters

The effective segment volumes were evaluated on the basis of a study done by Woods Hole Oceanographic Institute of the dimensions of the basin. This study is an appendix to reference (30). These dimensions are shown in Table 3-2. They were used to calculate the total and effective volume of the basin as a function of distance from the Watertown Dam, as shown in Figure 3-3. The total volume is defined as the total volume of water below the dam and the effective volume is defined as the total volume of aerobic water below the dam, calculated assuming a maximum depth of twelve feet.

TABLE 3-2

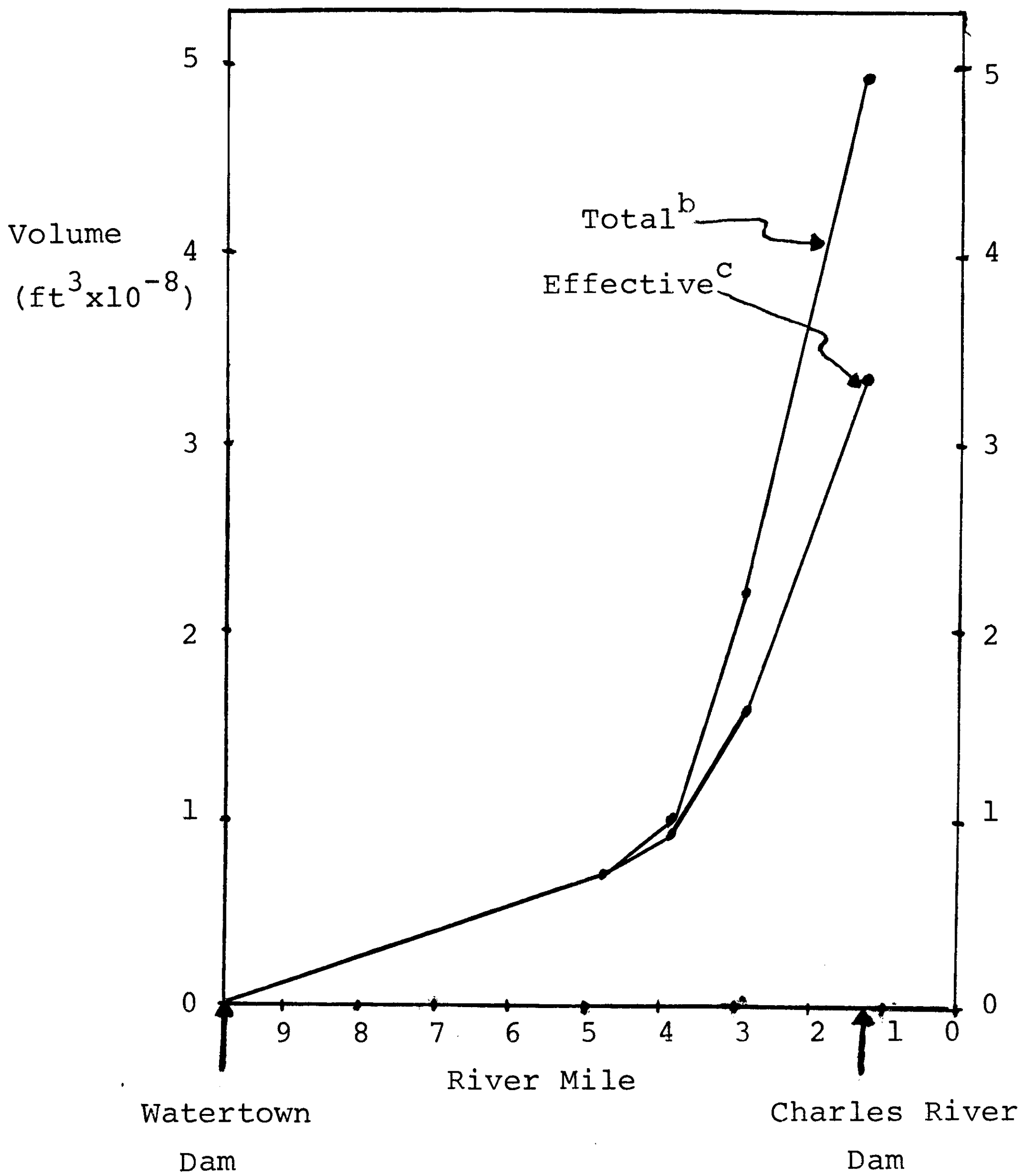
Dimensions of the Basin[a]

Segment	Length (in feet)	Width (in feet)	Total Depth	Effective Depth
Watertown Dam				
'	26,900	250	9	9
Western Avenue Br.				
'	4,750	450	17	12
B.U.Bridge				
'	5,280	1,100	21	12
Muddy River				
'	8,440	1,700	19	12
Charles River Dam				

a - reference (30)

FIGURE 3-3

Total and Effective Volumes of Basin[a]



a - calculated from dimensions in reference (30)
b - defined as total water volume below Watertown Dam
c - defined as total aerobic water volume below Watertown Dam, assuming maximum depth of 12 feet

The model also requires as means of estimating the flow of the river at each water quality monitoring station. The U.S. Geological Survey operates a flow gauge near the Moody Street Dam in Waltham. The data is published in the form of yearly summaries, containing daily and monthly average flows (13). The method most commonly used to estimate the flow of a river at any point downstream of a gauge is to assume that the flow is proportional to the total drainage area, i.e., to the total land area contributing water to the river (37). About 35% of the total runoff from the upper watershed is diverted to the Neponset and the Mystic Rivers (38). In order to use the drainage area approximation to estimate the flow at any point in the Lower Charles, diversion from the Upper Charles must be taken into account.Accordingly, the expression for the total runoff at Moody Street, R_m (cfs), is:

$$R_m = \frac{Q_m}{1 - .35} = \frac{Q_m}{.65} \tag{15}$$

where Q_m is the measured daily average flow at Moody Street in cfs. Assuming that the total runoff R_x at any point x downstream of Moody Street is proportional to the total drainage area at that point, A_x:

$$R_x = R_m \frac{A_x}{A_m} \tag{16}$$

where A_m is the total drainage area at Moddy Street, 249.2 square miles. Accounting for the 35% diversion above Moody Street, the expression for the total flow of the river at point x, Q_x is given by:

$$Q_x = R_m \frac{A_x}{A_m} - .35 R_m \tag{17}$$

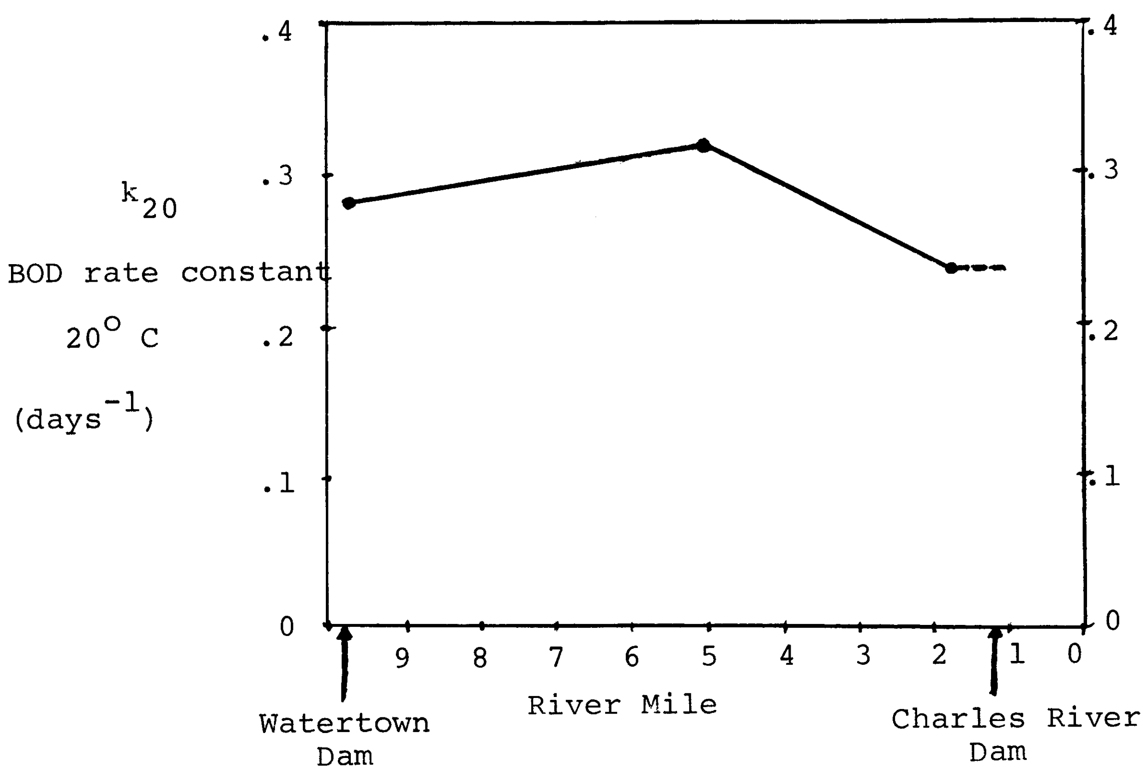
$$= \frac{Q_m}{.65} \left(\frac{A_x}{A_m} - .35\right)$$

The above equation provides only a rough approximation to flow values. It is not valid during rainstorms. The areas draining into the Lower Charles have a much higher percentage of impervious surfaces than those draining into the Upper Charles. The river responds much faster to a rainstorm in the lower section than it does in the upper (14). Thus, the approximation may not be valid during and immediately following a rainstorm. However, as shown in equation (12), the source values calculated in the model depend upon the difference between the flow in and the flow out of a segment. The total drainage area at the Charles River Dam is 304.2 square miles, as compared with 249.2 square miles at Moody Street (14). According to equation (17), this amounts to a 34% difference in flow between the two dams. If the river is divided up into a series of five segments, there will be only about a 7% difference between the flows entering and leaving each segment. The model thus is somewhat insensitive to errors introduced by this approximation. The local drainage areas, or those areas draining directly into each segment, will have to be determined from the characteristics of the storm and combined sewage systems in each area.

The model also requires an estimate of the BOD rate constant, k , in each segment. Equation (6) will be used to calculate the rate constants at the river temperature from values at 20°C, which have been determined from data contained in the F.W.Q.A. survey (5). In this survey, 2-day and 5-day BOD's were determined at three stations in the basin on thirteen dates. These data were applied to equation (4) to estimate k_{20} values at each location. The calculates values were found to vary significantly along the length of the river. The t-test, as applied here, revealed that the k_{20} values at adjacent stations were significantly different at the .95 level. Figure 3-4 is a plot of the average k_{20} values against distance. A linear extrapolation is used between points. In each segment, the k value applied in the source estimation equation (12) is determined as the arithmetic average of the k's at each bordering station. The correction of the k_{20} values to river temperature is made before the average is taken.

FIGURE 3-4

BOD Rate Constant Profile[a]



a - base e; determined from data in F.W.Q.A. study (5)

3.6 Applications

Possible applications of the model should be examined in consideration of its inherent limitations. The model is designed to calculate source quantities of BOD entering a given segment from information obtained at the borders of the segment. Because of the real possibility of random perturbations influencing the data or the behavior of the materials within each segment, the model must be applied in a probablistic manner. Many data sets will have to be examined in order to obtain valid information.

Theoretically, if continuous graphs showing the time variation of concentration and water flow were available, the model might be used to obtain some interesting information about the behavior of pollution sources in the Charles. Specifically, source quantities could be correlated with rainfall, street cleaning, toilet flushing frequency in Cambridge, or with any factor which might be of influence. This would provide an interesting picture of the general relationships between the city and the river. The size and nature of the most significant sources could be pointed out from their estimated quantities and observed response behavior. Shrinking the size of the control volumes by increasing the number of monitoring stations would serve to locate the sources, which could then be studied individually.

Unfortunately, continuous graphs of concentration and flow are not available. Most of the sampling done in the basin to date has been of the grab sample, dry-weather variety. Such programs cannot be expected to provide an adequate picture of water quality in the basin, since most of the sources have always been presumed to be connected with rainfall. Marked decreases in dissolved oxygen levels, for example, may occur during periods of significant sewage overflows. The present sampling programs tend to indicate sufficient dissolved oxygen for fish life, but it is unclear whether fish could survive a major storm.

Dry-weather data can be applied to the model, however, to obtain information about the source behavior. Such data would certainly reflect continuous discharges, which may include leakage from the sewage systems, direct sanitary lines into the Charles, BOD from bottom deposits, and algae. The BOD from bottom deposits would in turn be reflections of suspended solid organic materials which could have entered the river in a combined overflow or storm drain. In view of the vertical stratification of the basin and the fact that most of the combined overflows come up underneath the river, dry-weather data may also reflect any dissolved organic materials entering in combined overflows. The lower layer might act as a capacitor in storing the slugs of organic materials entering during storms and allowing them to slowly diffuse into the upper layer. Despite these considerations, there is still a fundamental uncertainty as to whether water quality in the surface regions significantly deteriorates during storms. This must be kept in mind in the application of the model to dry-weather data.

The model may also be used in a predictive sense, i.e., once the source values have been obtained for the present situation, the equations developed in the model could be used to predict the effects of various source abatement programs on BOD concentrations in the basin. However, as pointed out in Section 3.2, BOD does not appear to be a particularly critical parameter in the basin. The information provided by such a model would therefore be of questionable real value in evaluating programs designed to enhance overall water quality. The results of the model may be used in a predictive sense, however, by using BOD as a yardstick to estimate source quantities of other pollutants and showing the changes in these quantities resulting from various abatement programs.

4. Results

A Fortran computer program was written to perform the calculations prescribed by the model and to do a statistical analysis of the results. The data applied to the model were taken from a long-term water quality analysis program being carried on by the M.D.C. (3). This study involves monthly surface samples at seven locations in the basin. The 5-day BOD and temperature data used in combination with the mean daily flow records of the U.S.G.S. gauge in Waltham to estimate the source quantities of BOD entering each river segment on each sampling day. These source quantities were then employed to estimate yearly average values. As discussed previously, it is unclear what percentage of the sources is ignored because of the dry-weather sampling philosophy of the M.D.C study. The quantities ignored may be small because of the possible high detention times of the overflow materials in the depths of the basin.

There is really no way of testing the validity of the model directly, i.e., by comparing results with known facts. The proof of the model comes indirectly, through a demonstration of the cause and effect relationships influencing the results. The calculated source values are examined for their response to various factors which are external to the model and which should have an influence on the source quantities. The predicted effect of the August, 1967 activation of the South Charles Relief Sewer on the abatement and redistribution of the sources is shown.Further calculations are done in an effort to relate the source values to the characteristics of the sewage systems contributing to each model segment. The estimates will be used in Chapter 5 to provide an overall picture of the pollution sources and to describe how they might be influenced by the various abatement proposals.

4.1 Input Data

The M.D.C. study was initiated in November of 1966. Since then, samples have been taken approximately every month at seven locations within the basin. Because the river was frozen over on some of the sampling days, samples could not be taken at each location on every day. The model is applied only to the data taken on days in which all locations were accessible. The data are divided into two portions, 14 sets taken before and 35 seta taken after the activation of the South Charles Relief Sewer in August of 1967. Tables 4-1 and 4-2 show the dates, mean daily flows, and BOD_5 concentrations measured at each of the seven locations.

The BOD_5 concentrations measured at the Eliot Bridge were highly erratic, indicating the existence of a major source in the immediate vacinity of the bridge. This station is not included in the model because the assumption of even source distribution in this case is not valid. It is unclear whether the samples taken at the Eliot Bridge are at all representative.

Flow data are available for all dates up to October 1, 1969. The flow on each subsequent date is estimated from available flow and rainfall data. For example, flow on a sampling date in June of 1970 is estimated as the mean flow for the month of June in a previous year. The previous year is selected as that in which the total rainfall during the month of June was closest to the total rainfall observed in

TABLE 4-1
BOD_5 Concentrations in mg/liter
M.D.C. Data
Before Activation of South Charles Relief Sewer

Date	Flow[a] (cfs)	Watertown Dam	No. Beacon St. Bridge	Eliot Bridge	Western Ave. Bridge	B.U. Bridge	Harvard Bridge	Charles River Dam (upper)
11/ 8/66	320	2.9	1.8	4.1	0.2	2.7	1.4	6.5
11/15/66	203	1.6	2.0	5.8	1.5	2.9	2.2	3.6
11/29/66	114	3.8	6.6	4.8	5.2	5.6	5.6	5.6
12/ 6/66	97	2.5	2.9	14.4	3.6	4.6	1.7	1.6
12/13/66	116	3.2	7.3	1.8	1.9	5.0	2.9	2.2
12/20/66	126	2.7	3.3	5.4	2.5	3.4	9.5	4.1
1/10/67	253	3.4	7.2	9.5	7.9	7.6	5.0	7.2
1/24/67	174	2.9	7.0	36.0	6.5	6.8	1.3	0.7
1/31/67	320	2.7	5.4	6.8	5.9	8.8	7.9	5.0
3/14/67	669	4.7	4.7	7.2	6.7	9.6	5.8	2.6
4/11/67	754	3.1	3.6	1.6	4.7	3.1	1.6	2.9
5/ 9/67	485	2.9	2.2	3.8	2.9	2.9	5.0	5.0
7/ 6/67	261	1.4	0.2	4.0	0.4	8.0	4.0	4.8
8/ 7/67	280	3.2	1.2	1.7	2.4	7.7	1.0	2.0
Average	-	2.92	3.95	7.63	3.73	5.62	3.92	3.84
Standard Dev. / Average		.279	.609	1.158	.654	.430	.677	.503

a - measured at Moody Street in Waltham (13)

TABLE 4-2

BOD_5 Concentrations in mg/liter

M.D.C. Data

After Activation of South Charles Relief Sewer

Date	Flow[a] (cfs)	Watertown Dam	No. Beacon St. Bridge	Eliot Bridge	Western Ave. Bridge	B.U. Bridge	Harvard Bridge	Charles River Dam (upper)
9/ 5/67	128	2.9	3.6	6.1	5.0	4.7	3.1	4.3
10/ 2/67	160	1.7	0.2	0.5	2.4	3.4	1.9	2.4
11/ 1/67	97	2.8	3.2	5.6	4.0	3.6	2.8	2.8
12/ 6/67	309	2.8	3.2	2.6	2.4	0.8	3.4	2.2
4/ 2/68	824	6.8	6.8	7.2	6.7	7.0	6.8	6.8
5/20/68	345	3.6	3.4	2.0	3.4	4.4	3.7	2.9
6/ 5/68	315	2.4	2.4	2.6	3.4	6.4	2.2	2.0
8/14/68	61	6.0	4.2	-	3.0	4.0	2.4	2.8
9/ 4/68	42	3.4	11.4	46.2	7.8	6.0	2.4	3.6
10/ 2/68	29	3.0	6.2	37.6	3.8	3.6	4.0	4.4
11/ 4/68	51	5.4	4.8	3.4	5.7	5.4	12.4	1.4
12/15/68	383	3.0	3.0	4.6	5.0	4.2	3.4	2.2
3/19/69	430	5.8	3.6	3.6	4.2	6.2	3.8	3.2
4/29/69	950	1.8	1.6	2.0	1.2	2.0	1.4	1.6
5/13/69	430	3.1	2.2	1.6	2.5	2.9	0.5	1.1
6/ 9/69	141	4.1	2.4	1.4	4.1	0.8	2.5	2.4
7/16/69	79	3.4	3.6	-	4.0	3.4	2.7	3.2
8/12/69	112	4.0	6.8	4.0	4.7	4.3	3.1	2.7
9/18/69	179	4.9	6.2	3.8	2.6	5.4	1.4	2.0
10/ 1/69	76[b]	4.4	4.0	5.4	5.2	2.8	3.0	2.6
11/13/69	120[b]	2.6	5.0	3.0	3.0	3.0	2.8	2.8
12/ 9/69	240[b]	5.2	7.1	7.3	3.5	7.1	4.6	3.0
2/19/70	271[b]	2.6	2.8	2.0	2.0	2.0	2.4	3.4
2/25/70	271[b]	3.4	3.0	5.0	2.2	2.6	2.6	3.0
3/ 5/70	271[b]	2.6	3.8	2.6	5.4	2.8	3.0	2.0
3/19/70	385[b]	3.8	3.6	2.8	2.4	3.8	2.6	2.6
4/ 8/70	577[b]	2.6	2.6	2.0	2.8	2.6	2.2	2.6
4/22/70	700[b]	3.2	3.4	3.6	3.2	3.6	2.6	2.8
5/ 5/70	283[b]	2.7	2.5	3.3	4.7	4.0	2.9	2.7
5/27/70	283[b]	2.8	3.0	3.4	3.2	4.8	3.4	7.0
8/25/70	162[b]	9.2	8.7	12.5	4.2	4.3	3.3	4.9
9/ 9/70	58[b]	6.4	0.8	1.6	4.4	6.6	6.0	4.4
11/19/70	37[b]	2.0	1.4	1.6	1.8	0.8	1.2	0.8
12/21/70	116[b]	0.9	3.1	1.7	1.0	1.7	2.4	1.9
3/ 2/71	245[b]	1.8	2.5	2.5	2.0	2.5	2.7	3.4
Average	-	3.56	3.88	5.91	3.63	3.82	3.22	2.96
Standard Dev. / Average		.477	.598	1.627	.425	.464	.635	.463

a - measured at Moody Street in Waltham (13)

b - flows estimated

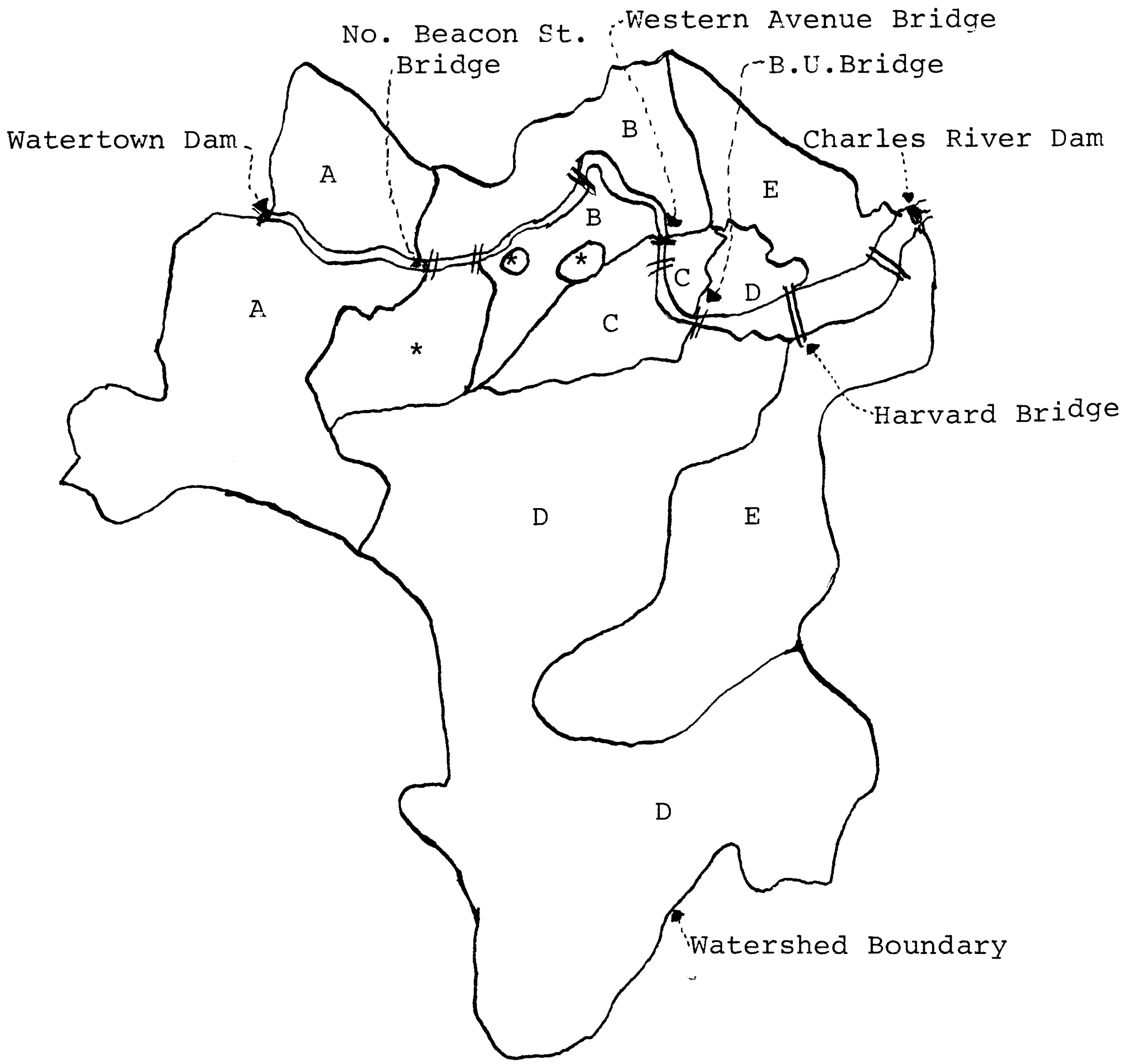
the month prior to the sampling date in June of 1970. This is only a rough estimate. As was discussed in Chapter 3, the source quantities are not highly dependent upon the accuracy of flow estimation.

The model consists of five segments. The relevant data on each are contained in Table 4-3. With the aid of a planimeter, the drainage areas were measured from a map composed from individual sewer maps, the same ones used to produce Figure 2-4. Figure 4-1 shows the distribution of the local drainage areas within the watershed. The total drainage area is defined at any point on the river as the total area of land draining into the river up to that point. The local drainage area is defined for any river segment as the land area draining directly into that segment.

The areas contributing to segments B and C were changed as a result of the activation of the South Charles Relief Sewer, which carried to the B.U.Bridge overflows originally discharged into segment B. The area contributing to segment E from the Boston side represents the combined sewer area served by the Stony Brook Valley Sewer and the West Side Interceptor, both of which discharge into the M.D.C. Marginal Conduit during wet weather. This, in turn, often overflows into the basin between the Harvard Bridge and the dam. The section of the segment D drainage area in the extreme lower portion of the map represents the separate sewer area served by the Stony Brook Conduit, which discharges into the basin above the Harvard Bridge. Further discussion

FIGURE 4-1

Drainage Area Distribution
for Application of M.D.C. Data



* drains into segment B before the activation of the South Charles Relief Sewer and into segment C, after.

of the characteristics of each individual drainage area and of how these characteristics relate to source quantities will follow a presentation of the results.

TABLE 4-3

Model Framework for Application of M.D.C. Data

Sampling Station	Segment	River Mile[a]	Assumed[b] k_{20} (day^{-1})	Total Drainage Area (mi^2)
Watertown Dam		9.77	.28	265.3
	A			
North Beacon St.Bridge		8.11	.28	273.0
	B			
Western Avenue Bridge		4.74	.31	278.6[c] 276.6
	C			
B.U.Bridge		3.75	.28	280.3
	D			
Harvard Bridge		2.75	.25	297.8
	E			
Charles River Dam		1.18	.24	304.6

Segment	Local Drainage Area (acres)	Effective[d] Volume ($x10^{-8}$ ft^3)
A	4936	.21
B	3605[c] 2332	.40
C	1060[c] 2333	.29
D	11203	.75
E	4328	1.63

a - miles above mouth of river
b - see Figure 3-4
c - before activation of South Charles Relief Sewer
d - see Figure 3-3

4.2 Calculation and Comparison of Source Quantities

Table 4-4 contains the results of the application of the model to data taken before and after the activation of the South Charles Relief Sewer. A wide spread in the calculated source values for each segment is apparent. The sources calculated from the first set of data have standard deviations between 88% and 297% of the calculated mean values, as compared with a 60% to 124% range for the second set of data. The total source quantities were subject to less variation. For a given segment, a standard deviation greater than 100% of the mean would indicate that the calculated source quantities were negative at least 16% of the time, assuming that 68% of the values lie within one standard deviation of the mean. This can be attributed somewhat to random fluctuations in the data and in the mixing properties of the river. Further implications of the calculated negative source quantities will be discussed later.

T-tests were applied to the two seta of results to determine the significance of the observed difference in average values for each segment. As shown, the reduction in the average values has a significance level greater than .75 for segment A and greater than .90 for segment C and for the total source quantities. The significance levels for the other segments are too low to be conclusive. The sources calculated from the second set of data, consisting of 35 sampling days, show considerably less variation than those calculated from the first set, which consists of only

14 sampling days. The generally low significance levels may be due to the fact that there are insufficient data from the period before the activation of the relief sewer to provide an adequate statistical basis for comparison. The differences in the average values are qualitatively what would be expected and will be discussed further in Chapter 5.

TABLE 4-4

Application of M.D.C. Data Taken Before and After Activation of South Charles Relief Sewer

Model Segment	Sources of BOD in lbs per day[a] 14 Data Sets Before Activation $\bar{S}_B$ [b]	$\sigma_B/\bar{S}_B$ [c]	35 Data Sets After Activation $\bar{S}_A$ [b]	$\sigma_A/\bar{S}_A$ [c]	$(\bar{S}_A - \bar{S}_B)$ [d]	Significance Level of Difference $<\alpha<$	
A	2923	1.435	1930	1.243	-993	.75	.90
B	3463	1.304	2610	1.148	-853	-	.75
C	7173	1.114	3702	.982	-3471	.90	.95
D	2696	2.966	4152	1.084	1456	-	.75
E	11098	.884	9607	.600	-1491	-	.75
Total	27352	.409	22001	.480	-5351	.90	.95

a - ultimate, carbonaceous BOD
b - average source quantities calculated for each segment
c - standard deviation of calculated value divided by mean
d - difference in average values for each segment as a result of new relief sewer; negative value indicates a decrease in the quantities of BOD reaching the river after the activation of the sewer.

4.3 Source-Drainage Area Relationships

Additional information is obtained from the results through a comparison of the calculated source quantities with the characteristics of the sewage systems discharging into each segment. The source values calculated from the more extensive and consistent second set of data will be used for this purpose. First, an estimate must be made of how much of the calculated source for any segment is actually external to the river. The model does not distinguish between internal and external sources of organic materials. The primary internal source of BOD is assumed to be algae, which act as a source by recycling carbon dioxide into the pool of organic materials in the river.Decaying algae show up as part of the total carbonaceous BOD.

A generally assumed value for the contribution of algae to BOD in a river is about 1 mg/liter of BOD_5 (11). This is a typicel value and admittedly not applicable to cases of extremely excessive amounts of algae, such as might be found in some spots in the Charles during the late summer months. For the present purpose, this value will be used to estimate the average internal component of the total source quantity calculated for each segment. This is done by using the model to calculate sources under average conditions of temperature and flow, with an assumed constant BOD_5 concentration of 1 mg/liter at each station. The average conditions are determined from the 35 sampling dates to be a temperature of 15°C and a flow at Waltham of 260 cfs.

Subtracting the algae component from the total source quantity for each segment gives an estimate of the external source. This can be attributed to the land area draining directly into each segment. The results of these calculations will be presented after consideration of a means of comparing the contributions to each segment.

In the case of separate systems, on a yearly basis, the average source rate S_s (lbs BOD_u/year) for any segment may be expressed as:

$$S_s = K_1 F c_r \tag{18}$$

F = total quantity of runoff contributed directly to the segment from the local drainage area (cu.ft./yr.)

c_r = average BOD_u concentration in runoff (mg/liter)

K_1 = units conversion factor = $6.24\times10^{-5} \dfrac{\text{lb/yr}}{\text{(cu.ft./yr)(mg/l)}}$

The total yearly quantity of runoff may be expressed as:

$$F = K_2 \phi i a_s \tag{19}$$

i = yearly rainfall (in./yr.)

a_s = local drainage area of river segment served by separate systems (acres)

ϕ = runoff coefficient, fraction of the rainfall falling on the local drainage area which reaches the collection system (dimensionless)

K_2 = units conversion factor = $3.63\times10^{3} \dfrac{\text{cu.ft./yr.}}{\text{acre (in/yr)}}$

Combining equations (18) and (19):

$$S_s = K_1 K_2 \phi i a_s c_r \tag{20}$$

This is an expression for the average yearly quantity of BOD contributed in runoff. If the runoff passes through a combined system which overflows into the river, the net effect could be represented approximately as an increase in the runoff concentration c_r to an average overflow concentration c_o. Another factor, f, is added to account for the runoff which is handled by the collection system and carried to the treatment plant. f is expressed as a fraction less than 1. Accordingly, for combined ssystems, the quantity of BOD contributed to any segment, S_c (lb/yr) may be expressed roughly as:

$$S_c = K_1 K_2 \phi a_c (1-f) c_o \tag{21}$$

In order to eliminate variation in the calculated sources due to water quantity and to focus on variation due to concentration, it is advantageous to normalize the sources on a per unit of local drainage area basis. In order to to do this, define a loading factor, W (lb/acre-yr), as the following:

$$W = S/a \tag{22}$$

For a segment served by completely separate sewers, the loading factor would be given by:

$$W_s = S_s/a_s = K_1 K_2 \phi i c_r \tag{23}$$

The corresponding expression for a combined system would be:

$$W_c = S_c/a_c = K_1 K_2 \phi i (1 - f) c_o \tag{24}$$

The runoff coefficient ϕ for a typical city may vary between

.3 and .7 (37), depending upon the percentage of pavement in the area. The total yearly rainfall, i , is a constant for all segments. The f factor for combined systems depends upon the capacity of the combined sewers. It is difficult to predict this value, though it is probably small because the storm-water capacities of most of the combined sewers in the area are quite low. Most of the difference between W_s and W_c would be due to the difference in concentrations c_r and c_o. As many of the segments have local drainage areas which are served by both combined and separate systems, the expression for the loading factor for any segment may be given by:

$$W = y W_s + (1 - y) W_c \qquad (25)$$

y = fraction of the local drainage area which is served by separate sewers

$1 - y$ = fraction served by combined sewers

Some variation in W_s and W_c from area to area might be expected, due to differences in runoff coefficients, average concentrations, and combined system characteristics. The next step is to calculate loading factors for all segments and to see how they vary with sewage system characteristics in each local drainage area.

The results of these calculations are shown in Table 4-5. The loading factors for BOD_u and BOD_5 are cited. The latter may be compared with values cited from the literature and contained in Table 2-6. Table 4-6 shows the characteristics of the sewer systems in the areas contributing to each model segment. The table contains a breakdown of each local drainage area into separate areas, combined areas, and separate areas discharging storm-water into combined conduits. Systems of the last category would be expected to contribute the same quantities of organic materials to the river as combined systems and thus are treated as effectively combined areas.

Table 4-5

Comparison of BOD Quantities Contributed to Each Segment

Segment:	A	B	C	D	E
Total BOD Source[a] (lbs./day)	1930	2610	3702	4152	9607
Internal Source[b] (lbs./day)	521	795	753	1844	3278
External Source[c] (lbs./day)	1409	1815	2949	2308	6329
Local Drainage[d] Area (acres)	4936	2332	2333	11203	4328
BOD_u Loading Factor[e] (lbs./acre-yr)	104	284	461	75	534
BOD_5 Loading Factor[f] (lbs./acre-yr)	85	243	378	62	446

a - average values for sources of ultimate carbonaceous BOD; calculated from data taken after the activation of the South Charles Relief Sewer; see Table 4-4
b - algae component, assumed contribution 1 mg/liter of BOD_5
c - difference between total and internal source
d - area draining directly into each model segment; see Table 4-3
e - calculated from external source quantities and local drainage areas
f - assuming BOD_5/BOD_u = .82 , as for sewage (11)

Table 4-6

Characteristics of Sewer Systems Draining into Each Model Segment

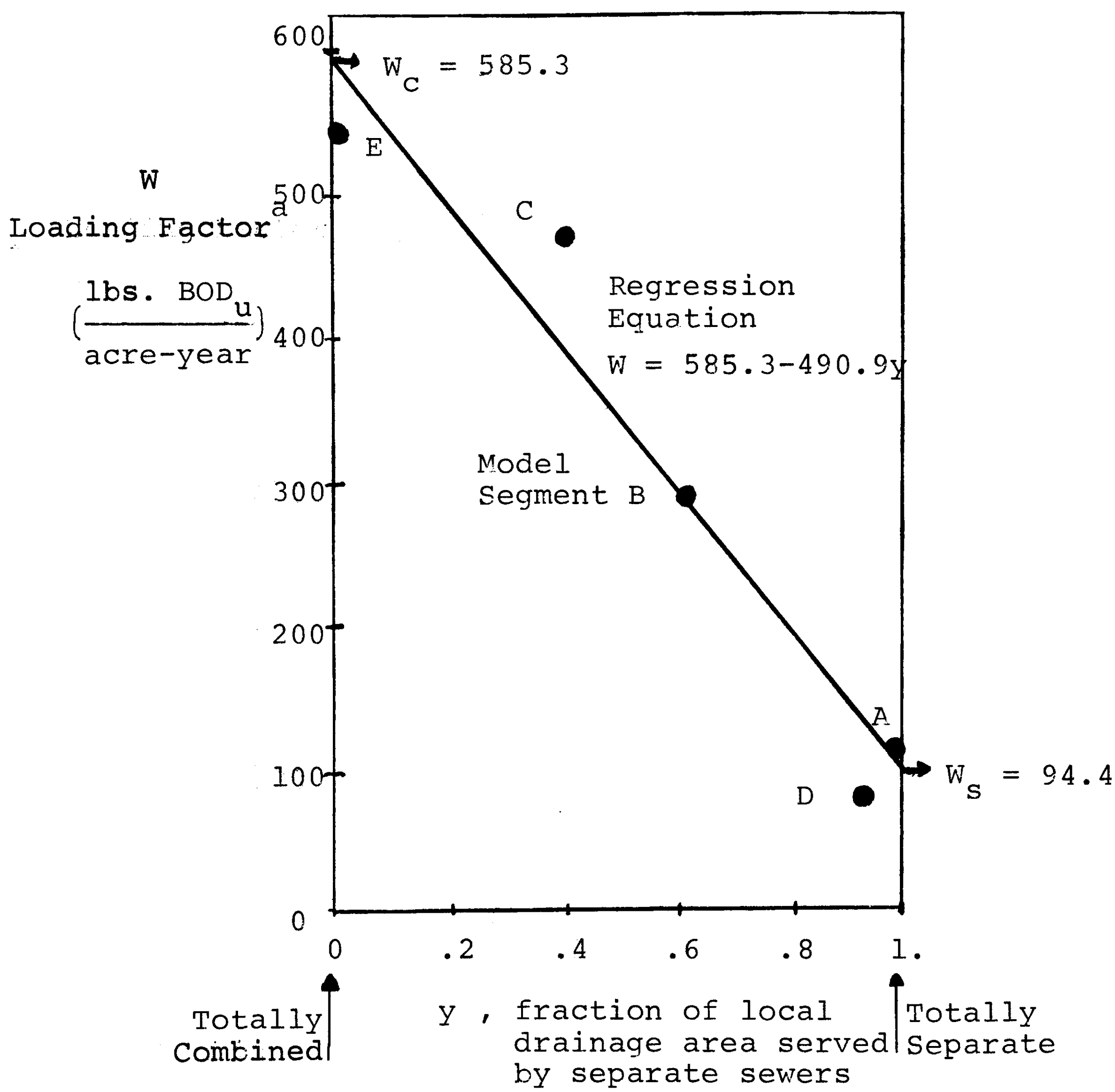
	A	B	C	D	E
Local Drainage Area (acres)	4936	2332	2333	11203	4328
Separate	4936	1460	955	10574	51
Combined	0	872	276	629	3697
Both[a]	0	0	1102	0	580
Percent Separate[b]	100%	62.6%	40.9%	94.4%	1.2%

a - separate sewered areas discharging storm-water into combined conduits
b - separate area/total area x 100%

Assuming that all of the combined sewer areas in the watershed might be characterized by one average BOD loading factor, W_c, and all of the separate areas by another, W_s, one would expect there to be a linear relationship between the loading factor for a given area and the percent of the area served by separate sewers, as given by equation (25). Figure 4-2 shows that the assumption holds true. The loading factor for each segment is plotted against the percent sewer separation. The linear relationship is readily seen. A regression analysis was done to determine the extent of the linear relationship between these two parameters. The correlation coefficient was found to be .97. The regression equation developed from these points is:

FIGURE 4-2

Effect of Sewer Separation on Quantities of BOD Contributed to the Charles River



a - total quantity of carbonaceous BOD contributed to model segment per acre of local drainage area per year

$$W = 585.3 - 490.0\,y$$
$$= 94.4\,y + 585.3\,(1 - y) \qquad (26)$$

On the average, combined sewer areas in the Charles River Basin watershed can be characterized by a BOD_u loading factor, W_c, of 585.3 lbs./acre-yr. Separate sewer areas can be characterized by a loading factor, W_s, of 94.4 lbs./acre-yr. In other words, combined areas contribute to the Charles about 6.2 times the quantity of organic materials contributed by separate systems per unit area per year.

Given average values of W_s and W_c, an estimate can be made of the average concentrations of BOD which one would expect to find in runoff and combined overflows, using equations (23) and (24). The runoff coefficients for areas of Boston and Cambridge vary between .3 and .5 (32); a typical value might be around .4. A value of f, the fraction of the total runoff which is handled by the combined system, is difficult to estimate. For most of the systems in the area, it is probably less than .1. This value will be used for the sake of estimation. Assuming, then, that the parameters in equations (23) and (24) are given by:

$$\phi = .4$$
$$i = 43 \text{ in/yr}$$
$$f = .1$$
$$W_s = 94.4 \text{ lbs/acre-yr}$$
$$W_c = 485.3 \text{ lbs/acre-yr}$$

$$c_r = W_s/K_1K_2\,\phi i = 24.2 \text{ mg/liter } BOD_u$$
$$= 20.0 \text{ mg/liter } BOD_5$$

$$c_o = W_c/K_1K_2 \, \phi i \, (1-f) = 139 \text{ mg/liter } BOD_u$$
$$= 113 \text{ mg/liter } BOD_5$$

These values for typical concentrations of BOD_5 in runoff and combined overflows may be compared with values presented in Tables 2-2 and 2-3 quoted from the literature.

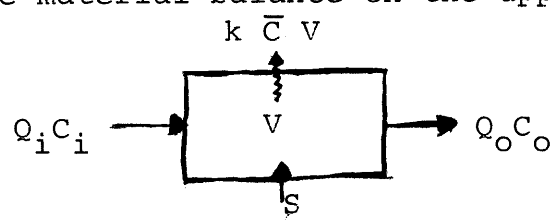
5. Discussion of Results

There are basically three aspects of the results which merit further consideration. First, the information obtained about the model itself will be discussed. Secondly, the demonstrated effects of the activation of the South Charles Relief Sewer will be considered. This will be followed finally by further application of the results in the formulation of a general pollutant material balance on the river and an evaluation of the possible effects of some of the proposed abatement plans.

5.1 Discussion of the Model

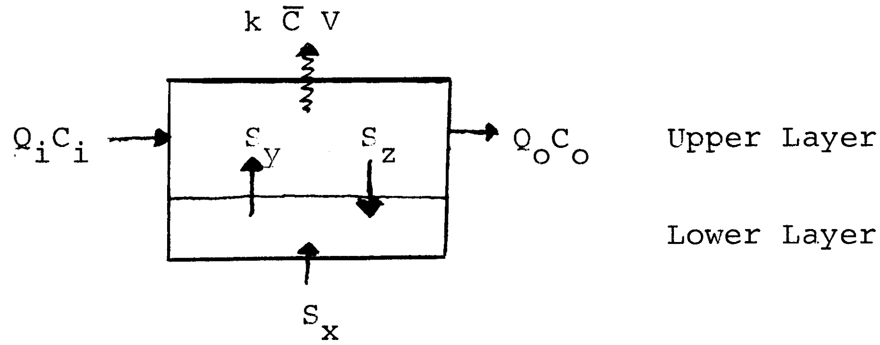
Some additional information about the model has been obtained as a result of its application. As discussed in Section 4.2, the calculated source quantities were negative on a number of occasions, particularly for segments A and D. This can be attributed to three factors: (a) statistical fluctuations due to sampling difficulties or lack of cross-sectional mixing on occasions, (b) non-steady-state conditions, or (c) a problem inherent in the model. One possible difficulty associated with the model is its failure to account for the settling of organic materials from the upper layer of the basin. In order to explain this, it would be advantageous to re-examine the fundamental material balance on the upper layer, and to formulate a balance on the lower layer.

The material balance on the upper layer is given by:



$$S + Q_iC_i = Q_oC_o + k\bar{C}V \qquad (27)$$

The lower layer is defined as the anaerobic water layer and the river bottom. Since overflows generally come up underneath the river, it is assumed that all of the sources must pass through the lower layer before reaching the upper layer. If the assumption is made that solid materials may settle out of the upper layer, the material balance on both layers is given by:



Upper Layer: $S_y - S_z + Q_iC_i = Q_oC_o + k\,\bar{C}\,V$ (28)

Lower Layer: $S_x + S_z = S_y + A_L$ (29)

S_x = BOD contributed to the river by overflows and storm drains (lbs/day)
S_y = BOD diffusing from the lower layer into the upper layer (lbs/day)
S_z = BOD settling from the upper layer into the lower layer (lbs/day)
A_L = BOD accumulating in the lower layer (lbs/day)

In this formulation, it can be seen that the net amount of material entering the upper layer from the lower is given by:

$$
\begin{aligned}
S &= S_y - S_z \\
&= S_x - A_L
\end{aligned}
\tag{30}
$$

Fusing the two material balances has shown that the source calculated by the model, S, is equal to the overflow and storm drain contributions, S_x, only in cases where the accumulation rate of solid BOD in the lower layer is zero.

On the average, one would expect the accumulation rate to be near zero, or only slightly positive because there does appear to be net accumulation of sediment on the bottom, as discussed in Section 2.2.1. However, on days when there are relatively excessive amounts of suspended solids in the upper layer, settling may occur at a sufficient rate to cause the model to calculate a negative source value. On other days, settling rates might be low, and diffusion from the lower layer into the upper layer may occur at such a rate as to cause a depletion, or negative accumulation of materials in the lower layer. On these days, the model would calculate a source that would be greater than the external overflow component, S_x. On the average, the sources calculated by the model are probably quite close to S_x. The primary consequence of the failure of the model to account for solids settling from the upper layer is a broadening of the distribution of calculated source values. The sources determined for segments A and D have the widest distributions. Both of these segments are particularly susceptible to solids settling because the flow velocity of the water decreases upon entering each of these segments.

It would be possible to add a term to the model to account for the settling of solid materials. This would require some information about the fraction of the BOD measured at each station which subject to sedimentation. This fraction may vary from day to day, though it might be sufficient to assume a constant value at each station. In combination with this, another term would have to be added to account for diffusion of organic compounds from the settles materials into the upper layer. This term, on the average, would balance the settling term. It would not include materials contributed from external BOD sources. Presumably, such a modification would produce more consistent results, though have little effect on the average values.

5.2 Effect of the South Charles Relief Sewer

The average source values calculated by the model for the periods before and after the relief sewer activation reflect the effects of the sewer on the abatement and redistribution of BOD sources in the basin. As show in Figure 2-3, the relief sewer runs along the south bank of the Charles, carrying overflows from the Charles River Valley Sewer to the B.U. Bridge. During wet weather when the line to the Ward Street pumping station reaches capacity, overflows are discharged into the river immediately upstream of the B.U. Bridge.

The total reduction in average source values for segments A,B, and C, all upstream of the B.U.Bridge, is 5316 lbs. of BOD_u per day, as shown in Table 4-4. Mr. William Butler of the F.W.Q.A (32) estimates that the activation of the South Charles Relief Sewer prevented the dry-weather discharge of 3400 lbs. of BOD_5 per day into this section of the river. This is equivalent to approximately 4150 lbs. of BOD_u per day, assuming $BOD_5/BOD_u = .82$ (11). This is the dry-weather component and agrees quite favorably with the results of the model, which, as previously discussed, may include both dry and wet-weather components.

The model indicates an increase of 1456 lbs. of BOD_u per day in segment D. The increase is not statistically significant, though there is a possible explanation. The activation of the relief sewer essentially concentrated all of the overflows from the South Charles System at the B.U. Bridge. Solid materials entering the basin at this point would presumably settle immediately downstream in segment D. Organic materials would then diffuse out of these settled solids into the upper layer. This would cause an apparent increase in the source quantities calculated for that segment.

The abatement of sources in segment E was calculated to be 1491 lbs. of BOD_u per day. This cannot be explained by the activation of the South Charles Relief Sewer. The M.D.C. activated another major sewer approximately a month before the new South Charles system was put into operation. This interceptor, as shown in Figure 2-3, carries dry-weather flow from the Stony Brook Valley Sewer and the West Side Interceptor to the Ward Street Headworks. This prevented the dry-weather flow of sewage into the M.D.C Marginal Conduit, which discharges into the basin at high tides. The calculated reduction in source quantities for segment E could be a result of this new interceptor.

5.3 Application of Results

The model itself is of little direct relevance to water quality problems in the Charles. It is nothing more than a tool used to estimate source quantities. The results of the model have been shown to be internally consistent and to reflect changes in the sewer systems. Perhaps the most significant result is the demonstrated relationship between sewage system characteristics and source quantities, as shown in Figure 4-2. This information can be used to provide a clear picture of the amounts and distribution of the pollution sources in the basin. Such a picture is invaluable in the interest of evaluating plans for pollution abatement.

It would first be of interest to describe further the distribution of the various types of sewer systems in the area. As shown in Figure 2-4, the systems are of three basic varieties: combined, serparate, and separate systems discharging storm-water into combined main interceptors. Areas of the last category will be treated as effectively combined areas. Table 5-1 contains a breakdown of the sewer areas in the watershed by city.

The model has provided the information to make a comparison of the BOD contributions of each city. As demonstrated in Section 4.3, the total BOD load may be estimated as:

$$S_T = A_C W_C + A_S W_S \qquad (31)$$

TABLE 5-1

Distribution of Combined and Separate Areas by City

Areas in acres[c]

City	Total Area in Watershed	Separate[d]	Combined	Both[a]	Percent Separate[b]
Boston	13389	8967	2599	823	67.0%
Newton	4094	2815	0	279	93.2%
Brookline	3072	2569	503	0	83.6%
Cambridge	2582	823	1372	387	31.9%
Watertown	1702	1702	0	0	100.0%
Somerville	193	0	0	193	0.0%
Belmont	100	100	0	0	100.0%
Total	25132	17976	5474	1682	71.5%

a - separate areas discharging storm-water into combined main interceptors; treated as effectively combined

b - completely separate area/ total watershed area x 100%

c - estimated from a map composed from individual sewer maps from references (33), (34), and (35).

d - also includes small contributions from non-sewered areas

S_T = total quantity of BOD_u contributed to the river (lbs/year)

A_c = combined sewer area (acres)

A_s = separate sewer area (acres)

W_c = combined area BOD_u loading factor = 585.3 lbs./acre-yr

W_s = separate area BOD_u loading factor = 94.4 lbs./acre-yr

Since combined overflow consists partially of runoff, it is possible to separate the loading factor for combined areas into two components:

$$W_c = W_c' + W_s \tag{32}$$

$$W_c' = 585.3 - 94.4 = 490.9 \text{ lbs/acre-yr}$$

W_c' represents only that portion of the organic materials in overflows contributed by sanitary sewage. The expression for the total source rate may be reformulated as:

$$S_T = A_c W_c' + A_T W_s \tag{33}$$

$$A_T = A_c + A_s \tag{34}$$

In storm-water runoff alone, any separate or combined sewer area, a, would contribute a fraction of the total BOD source given by:

$$\frac{aW_s}{S_T} = \frac{aW_s}{A_T W_s + A_c W_c'} = \frac{a}{A_T + \frac{A_c W_c'}{W_s}} \tag{35}$$

The corresponding expression for the sanitary sewage component of the total contribution from any combined area, a_c, is:

$$\frac{a_c W_c'}{S_T} = \frac{a_c}{\frac{A_T W_s}{W_c'} + A_c} \tag{36}$$

The model has provided the value:

$$\frac{W_c'}{W_s} = \frac{490.9}{94.4} = 5.2 \tag{37}$$

This provides the necessary information to estimate the percentage distribution of the BOD sources in the basin. There are two basic components: runoff, which originates in all areas, and sanitary sewage, which escapes in combined overflows. The percentages in Table 5-2 are a result of the application of the areas in Table 5-1 to equations (35), (36), and (37). The table shows that about 40.3% of the total quantity of BOD reaching the river originates in run-off. The remaining 59.7% can be attributed to sanitary sewage contributed in combined overflows. This means that complete separation of all of the sewers in the area would reduce the total loading by only 59.7% of its present value. An alternate way of expressing this distribution would be 28.8% attributed to separate systems and 71.2% attributed to combined systems. This distribution is obtained by adding the runoff and sanitary sewage components of combined over-flows to obtain the total contribution from combined sys-tems.

The result that 40.3% of the total quantity of BOD contributed to the basin originates in runoff is quite sur-prising. This figure was derived from the application of a simplified mathematical model which admittedly involved nu-merous assumptions. Skepticism about the validity of the model may lead to skepticism about the validity of the cal-

TABLE 5-2

Percentage Distribution of BOD Sources in the Basin

City	Percentage of Total Source[a] Contributed as: Storm-Water Runoff	Sanitary Sewage[b]
Boston	21.5%	36.9%
Newton	6.6%	2.3%
Brookline	4.9%	4.2%
Cambridge	4.1%	14.7%
Watertown	2.7%	-
Somerville	.3%	1.6%
Belmont	.2%	-
Total	40.3%	59.7%

a - total source rate of carbonaceous BOD_u = 14,810 lbs/day
b - sanitary sewage component of combined overflows

culated distribution of sources between sanitary sewage and urban runoff. It is possible to reinforce this result, however, by reference to the literature.

Section 2.3.1 contains a discussion of measurements made in various cities on the pollution potential or urban runoff and combined sewer overflows. Table 2-6 contains a summary of 5-day BOD loading factors for combined and separate systems, as determined by other investigations. These values were obtained by taking measurements on the sources themselves. In this table, the loading factor for separate systems, W_s, varies from 12 to 124, with an average of 56 lbs of BOD_5 per acre per year, equivalent to about 68 lbs of BOD_u per acre-year. The loading factor for combined systems, W_c, varies between 101 and 555, with an average value of 254 lbs of BOD_5 per acre-year, equivalent to 310 lbs of BOD_u per acre-year. These values may be applied to the distribution of combined and separate sewer areas in the present case. As in equation (32):

$$W_c' = W_c - W_s = 310 - 68 = 242 \text{ lbs/acre-yr} \quad (38)$$

$$\frac{W_c'}{W_s} = 3.6 \quad (39)$$

Using this value in equations (35) and (36), the distribution of sources comes out to be 51% attributed to runoff and 49% attributed to sanitary sewage. This shows that, on the basis of a comparison with literature values, the estimate of 40.3% is, if anything, low.

The estimate of the total quantity of BOD contributed to the basin provides a means of estimating quantities of other pollutants. This can be done using typical concentrations of various pollutants found in urban runoff and sanitary sewage. Table 5-3 shows typical concentrations of BOD_5, suspended solids, total nitrogen, total hydrolyzable phosphate, and total coliforms found in urban runoff and sanitary sewage. Assuming, in each case, that the ratio of the concentration of each material to the concentration of BOD is typical of sewage and runoff entering the Charles, the total quantity of each material reaching the basin can be estimated from the total quantity of BOD. The results of these calculations are presented in Table 5-4. Urban runoff alone is responsible for a significant portion of the total quantities of each material reaching the river in every case except total coliforms. The percentages due to runoff represent the sources quantities which would result if all of the combined areas were separated. According to these results, it is not clear that even complete sewer separation would solve the problems of the Charles.

The plans for pollution abatement in the Charles River Basin, as described in Section 2.3.2, are based on the assumption that the problems will be solved by elimination or treatment of combined overflows. The results presented above indicate that this may not necessarily be true. The success of the proposed abatement plans will depend upon how the problems are defined. From a bacteriological standpoint, it appears that the elimination of combined overflows will significantly improve the situation. The three main phases

TABLE 5-3

Concentrations of Various Pollutants Typically[a]
Found in Urban Runoff and Sanitary Sewage

Material	Concentrations in mg/liter: Sanitary Sewage	Urban Runoff
BOD_5	200	17
Suspended Solids	200	227
Total Nitrogen	30	3.1
Total Hydrolyzable Phosphate	25	1.1
Total Coliforms[b]	2.5×10^8 [c]	5.8×10^5

a - Weibel, et. al. (20)
b - expressed as total number per liter
c - Kittrell (11)

TABLE 5-4

Total Estimated Quantities of Various Pollutants
Reaching the Charles River Basin

Material	Quantities in lbs/day Contributed in: Sanitary Sewage[a]	Urban Runoff	Urban Runoff Percent of Total
BOD_u[b]	8841	5868	40.3%
BOD_5[c]	7250	4894	40.3%
Suspended Solids	7250	65336	90.0%
Total Nitrogen	1088	891	45.0%
Total Hydrolyzable Phosphate	906	314	25.7%
Total Coliforms[d]	4.14×10^{15}	7.58×10^{13}	1.8%

a - sanitary sewage component of combined overflows
b - carbonaceous only
c - assuming $(BOD_5/BOD_u) = .82$ (11)
d - expressed as total number per day

of the combined overflow abatement program are the Storm Detention and Chlorination Station, Cambridge sewer separation, and the construction of a pumping station to eliminate overflows from the M.D.C. Marginal Conduit. These programs, in combination, will influence the quantities of pollutants from all of the combined areas in the watershed. Table 5-5 shows the estimated effects each program will have on the total quantities of BOD and coliforms reaching the river. The total planned abatement amounts to a 46.7% reduction in the present BOD contributions and a 97% reduction in coliforms. It appears that the elimination of overflows from the M.D.C. Marginal Conduit will have the largest effect of any of the programs. There is considerable doubt as to whether the proposed pumping station will have a significant effect on overflows from this conduit. (1,32). Other alternatives for elimination of this problem should be examined. As shown in Table 5-5, the Deep Tunnel Plan, which proposes to eliminate the problem by removing stormwater from the combined areas, appears to be the most effective measure that could be taken.

TABLE 5-5

Influence of Proposed Abatement Plans on BOD and Total Coliform Contributions to the Basin

Plan	Combined Area Treated (acres)	Percent Reduction of Present Source Values: BOD[a]	Percent Reduction of Present Source Values: Total Coliforms[b]
Storm Detention and Chlorination Station[c]			
(1) South Charles System	1273	2.5%	16.7%
(2) Brookline[d]	629	1.2%	8.3%
(3) No. Charles Relief Sewer	977	2.0%	12.7%
Full Operation	2879	5.7%	37.7%
Cambridge Sewer Separation[e]	975	8.1%	13.6%
M.D.C. Marginal Conduit Pumping Station[f]	3302	32.9%	45.7%
Total Planned Abatement	7156	46.7%	97.0%
Complete Sewer Separation	7156	59.7%	98.2%
Deep Tunnel Plan[g]	7156	71.2%	98.7%

a - present value = 14,810 lbs/day carbonaceous BOD_u.

b - present value = 4.2×10^{15} total coliforms per day.

c - assuming 20% of influent BOD removal and 95% of influent total coliform kill.

d - including some combined areas in Boston west of the Fens.

e - only areas not served by proposed North Charles Relief Sewer.

f - assuming complete elimination of overflows from the M.D.C. Marginal Conduit.

g - assuming removal of all storm-water from combined areas in the watershed.

6. Conclusions

The following conclusions can be formed from evidence presented in this study:

1. The condition of the Charles River Basin can be traced to two factors: its highly urbanized watershed and its low dilution capacity.

2. The basin accepts wastes in two primary forms: storm-water runoff and sanitary sewage. Approximately 40% of the total quantity of organic materials contributed to the basin can be traced to runoff and 60% can be traced to sanitary sewage.

3. The storm and sanitary sewage collection facilities in the area cannot be blamed exclusively for the river's condition. Careless littering and inefficient urban housekeeping define the pollution potential of urban runoff and must share the blame for the river's condition.

4. There are basically two ways of controlling urban storm-water pollution:
   (a) by removing the storm-water, as recommended by the Deep Tunnel Plan (35);
   (b) by cleaning the city to prevent harmful materials from entering runoff.

   There are problems associated with each method. The first, while perhaps the most effective, appears to be prohibitively expensive. The second is limited by factors which are inherent in the city and are therefore difficult to control. These include littering, spillage, and dustfall. Efficient street cleaning and garbage collecting can help to minimize these problems.

7. Recommendations

The following recommendations are made:

1. There are a number of unanswered questions about the behavior of the river itself and about the nature of its pollution. These questions relate to:
    - (a) the proliferation and overall effects of algae, especially in relation to the oxygen balance of the basin;
    - (b) the response of dissolved oxygen levels to heavy inputs of organic materials occurring during storms;
    - (c) the changes in the salt wedge which may occur with the seasons;
    - (d) the effects of any toxic compounds which may be found in trace or greater quantities in the basin.

2. Since storm-water runoff is an important pollution source in the basin, a significant portion of the problem can be traced to private citizens, in their littering and other forms of carelessness. These aspects of the problem are unnecessary relative to those resulting from the sewage collection systems. Concerned people may make a significant contribution in this area by focusing on problems of the following sort:
    - (a) tracking down sources of specifically harmful or displeasing materials, such as waste oil;
    - (b) undertaking or provoking clean-up campaigns within the city, particularly in places where garbage may accumulate and contribute harmful materials to runoff;
    - (c) continuing to encourage the city governments to develop more efficient and more frequent street cleaning and waste collection procedures;

(d) discouraging additional pavement, which would only add to the problem;

(e) communicating to the public the need for their concern and for their conscious awareness of how their actions may directly contribute to the condition of the river.

3. Of the combined systems contributing to the Charles, the M.D.C. Marginal Conduit appears to be the most potent source of harmful materials. Since there appears to be considerable doubt as to the effectiveness of the plan, alternatives to the proposed pumping station should be sought and examined in the interest of eliminating overflows from this system. The solution should also take into account the interest in improving Boston Harbor water quality.

4. Ultimately, two programs will be necessary in order to significantly enhance the recreational and aesthetic value of the basin:

   (a) elimination or treatment of combined sewer overflows;

   (b) efficient solid waste management to reduce the pollutional threat of urban runoff.

   The Charles can be viewed as a place in which many of the city's harmful effects on the environment are concentrated. Control of pollution in the basin will only come through a waste management program which takes into account the air, land, and water resources of the area.

5. In the meantime, alternative ways of increasing the recreational value of the basin should be examined. The Army Corps of Engineers has proposed a system of bikeways which would extend along the Charles from

the Galen Street Bridge in Waltham to the Charles River Dam and along Muddy River from Jamaica Pond to the Harvard Bridge (39). The benefits afforded by such a system are obvious and numerous. Interested parties should push to turn this proposal into a reality.

APPENDIX

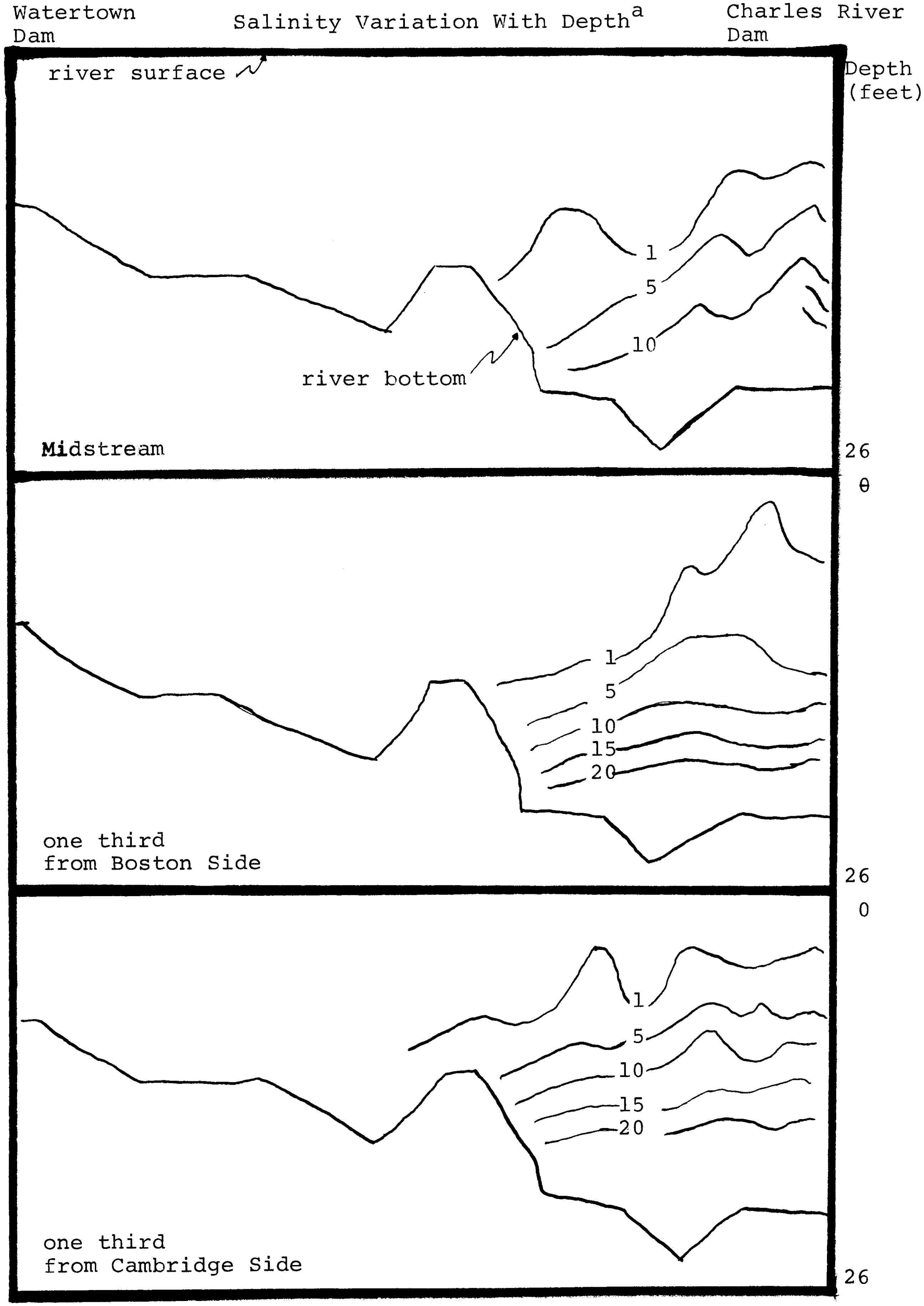
A. Salinity and Temperature Variation with Depth; Results of June 1968 survey by the F.W.Q.A. (12)

B. Oxygen Consumption and Transfer in Stagnant River Water.

C. Bibliography.

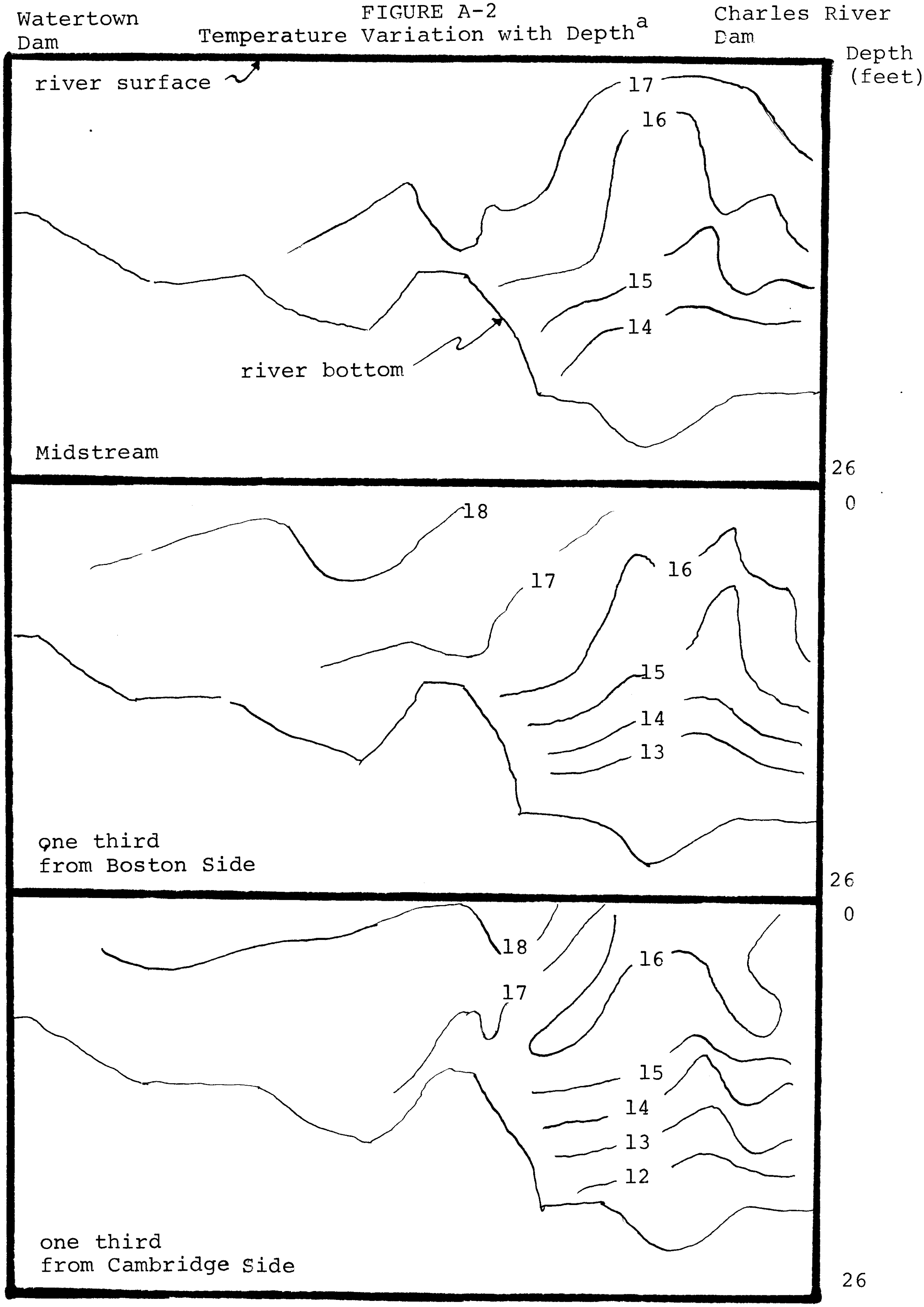
FIGURE A-1

Salinity Variation With Depth[a]



a - salinity expressed in parts per thousand; pure sea water = 30 ppt; measured by the F.W.Q.A. in June of 1968 (12)

FIGURE A-2
Temperature Variation with Depth[a]



a - temperature expressed in degrees C; measured by the F.W.Q.A. in June of 1968 (12)

APPENDIX B

Oxygen Consumption and Transfer in Stagnant River Water

The objective is to determine whether molecular diffusion occurs at a rate sufficient to keep up with oxygen consumption resulting from biodegradation of organic materials in stagnant river water.

Define the following terms:

D = diffusion coefficient of oxygen in stagnant water = 2×10^{-5} cm^2/sec. (21)

k = BOD rate constant 20^{o} C = .10 day^{-1} base 10 (11)
= .23 day^{-1} base e

L = concentration of ultimate carbonaceous BOD
≃ 5 mg/liter in the Charles
= 1 mg/liter for this estimate

h = distance from surface, or oxygen-staurated region (cm)

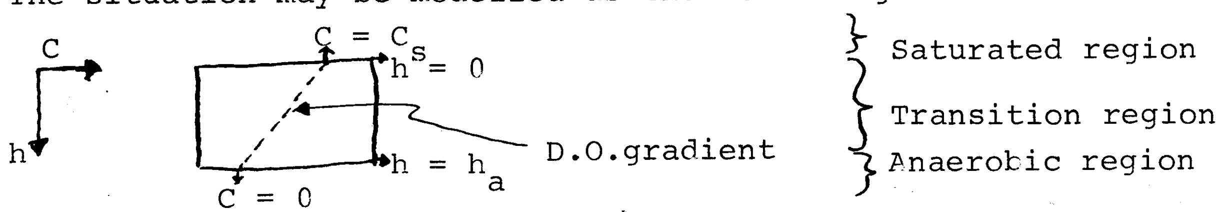
h_a = value of h at which anaerobic conditions begin (cm)

C = concentration of dissolved oxygen (mg/liter)

C_s = saturation concentration of dissolved oxygen (mg/liter)

A = surface area (cm^2)

The situation may be modelled as the following:



Assuming a linear D.O.Gradient with depth, the flux of oxygen into the volume of water may be represented as:

$$\text{Flux}_{in} = - D\ A\ \frac{dC}{dh} = D\ A\ \frac{C_s}{h_a}$$

The rate of oxygen consumption within the aerobic volume is given by:

$$\text{Consumption} = k\ L\ A\ h_a$$

Under steady-state conditions, equating flux_{in} with consumption:

$$D\ A\ \frac{C_s}{h_a} = k\ L\ A\ h_a$$

Solving for h_a, the depth at which anaerobic conditions begin:

$$h_a^2 = \frac{D\ C_s}{k\ L} = \frac{(2 \times 10^{-5} \frac{cm^2}{sec})(9\ mg/l)}{(.23\ day^{-1})(1 mg/l)} \quad (8.65 \times 10^5\ sec/day)$$

$$= 67.3\ cm^2$$

$$= 8.2\ cm$$

This means that even at oxygen comsumption rates one fifth those found in Charles River water,stable anaerobic regions will develop in areas where molecular diffusion of oxygen is the only method oxygen transfer. Anaerobic zones will develop in stagnant water less than 10 cm from flowing, oxygen-saturated regions.

APPENDIX C

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