Section 7 Modeling Studies

7.0 INTRODUCTION

The waste treatment planning process requires a knowledge of: the quantity and quality of the ground and surface fresh waters; the quality of the marine surface waters; the movements of ground and surface waters; the inter-relationships of ground and surface fresh waters with the marine waters—in terms of movement and quality; the contaminants that enter and impact each and all of these waters; and the structural and non-structural abatement alternatives most suited to cope with each set of problems in order to achieve the protection and/or rehabilitation of drinking and swimming waters to safe levels of water quality.

This requires an in-depth knowledge of existing conditions and an ability to predict future conditions under a variety of impacts. The study of existing conditions can be made by direct investigation. Water samples can be taken and analyzed, and controlled experiments can be conducted. This in fact has been done. However, in a region of more than 1200 square miles and as complex and dynamic as the Nassau-Suffolk Region it is prohibitive in terms of time and cost to totally rely on field studies. Furthermore, the only way that field studies can be used for predictive answers is to actually build a facility or to add contaminants and then measure the results. Fortunately a more effective and efficient set of tools is available to complement the field examinations; namely, the use of models.

One of the significant components of the Long Island 208 study centers on the development and use of appropriate mathematical models. The wastewater management plan for this area ultimately depends on the ability to quantify a preference for one set of structural and non-structural alternatives over another. In the process of doing so it is necessary to understand what the impact of these alternatives may be on ground and surface waters in terms of changes in movement and quality which result from the options used. The modeling efforts provide flexible tools to evaluate and predict such impacts without engaging in prohibitive tests on the actual water bodies themselves. That is, modeling is a surrogate for reality in which certain basic inter-relationships in the real world are expressed by mathematical statements. By manipulating these proxy statements one simulates, as it were, the events which actually take place. The models are therefore a method for organizing the complex interactions which occur between the water bodies and the stresses placed on them. This is a simplification of nature in that all interactions are excluded other than those which are of relevance to the study.

The use of mathematical statements to imitate reality has a long tradition in water management and other planning studies. Models enable researchers to carry out "what if" experiments on nature. For example, one can replace the actual waste loads which enter Long Island bays with esti-
rated loads which result from potential abatement options and then see what effect this has on subsequent bay conditions. Thus these changes can be simulated without having to physically build treatment facilities. That is, one can look into the future by tracking present conditions to determine what eventual effects they produce on water movement and quality. Finally one can play out enough such alterations in initial and boundary conditions to find the ‘best’ future scenario.

Section 7.1 of the report provides a straightforward but intuitive discussion of hydrodynamics. Fortunately, just a few basic facts concern the relation between water elevation (or ‘head’) and flow velocity suffice to derive all the fundamental equations used in the modeling of movement of ground and surface waters. As a result there is an essential conceptual unity between the apparently different modeling efforts used in the 208 study. The various models also use a common and simple idea, that of mass balance, in calculations of water quality and “water budget” considerations.

Section 7.2 contains a review of the surface water (including river) models, with some discussion of their limitations and uses. These models characterize the changes which occur over time (in a water body) to constituents such as salinity, nitrogen and dissolved oxygen. There is also a discussion of the models developed for achieving ecological balances under various conditions of constituent loadings.

Groundwater models are reviewed in Section 7.3. Two were developed; an analog and a digital one. In addition, water budget studies are explained.

Section 7.4 attempts to tie together the entire 208 modeling effort with a discussion of the results actually obtained and how they will be used in the development of a wastewater management plan.

The reader who does not wish to linger over some of the technical arguments can simply read the last section to get a perspective of how the models relate to the 208 study. We caution the reader, however, that this report cannot be a substitute for the detailed documentation provided by the respective consultants. The aim of this report is to explain the ideas which are necessary to an understanding of the structure of the models, their limitations, and integration into the overall 208 study. Explicit details concerning computational schemes, data sources, and eventual model validation are only hinted at, and the technically interested reader must consult the original sources on these questions. (See Bibliography.) Also excluded is a description of model outputs for different sites on the Island since these are also fully documented in reports issued by the consultants.

Every attempt was made to explain these models in understandable prose form. Differential equations—the language of models—were eliminated from the discussion. In those instances where technical jargon was deemed inescapable it was followed by a common language definition. A glossary listing is included at the end of the report.

7.1 HYDRODYNAMIC BACKGROUND

7.1.1 Introduction

The groundwater and surface water models used in the 208 study are based on the same hydrodynamic principles. It will be convenient, therefore, to review these basic ideas before they are applied to the models themselves. The discussion is somewhat simplified in order to provide a broad understanding of this material.

The derivations therefore will generally be intuitive to avoid technical complications or the overuse of mathematical formulae that add little or nothing to an overall appreciation of the essential ideas. Those readers not familiar with mathematical notation can safely omit the occasional allusion to equations. These are given as footnotes and hopefully do not intrude in too distracting a way.

7.1.2 Fluid Flow

Imagine a water body as being represented by a rectangular tank of elevation H and a fixed surface area $A_S$ connected to the outside by an inlet pipe or channel (see Figure 7-1). The tank volume is therefore $A_S H$. Water flows into or out of the tank through the channel with a velocity $u$. If the tube has a cross sectional area $A_C$ then the net flow of water is designated by $Q$. Using meters as a unit of length, $u$ can be expressed as meters per second. Therefore flow is given as cubic meters per second and the relation between the two is

$$Q = A_C u$$  \[\text{(7.1)}\]

It is apparent that if there is no mass flow into or out of the tank then the rate of change over time of the volume of water which it stores must be zero. This is simply a statement of conservation of mass. However if the net mass flow $Q$ is non-zero then the volumetric change per second must equal $Q$ itself. This is one of the two fundamental facts used in the models. Note that $H$ (also known as the ‘head’) varies up and down as $Q$ increases or decreases.¹

Suppose now that the channel is connected to another water column and that the storage tanks are labeled as 1, 2. If the elevation of water in

---

**FIGURE 7-1** Flow Into a Water Tank.
each of these is different than the resulting pressure differences at the two ends of the channel cause water to flow (See Figure 7–2). The rate at which this flow changes is in fact proportional to the difference in head. In particular if \( H_1 = H_2 \) then there is no flow. In actuality there is another force which acts to impede flow in the conduit. This is frictional resistance, a quantity which we denote by \( R \), caused by roughness of the channel walls. If we ignore this resistance, what we have is essentially a statement that water flows from a higher to a lower level and that the rate of flow is proportional to the difference in level. This is our second fundamental fact. In effect, it expresses Newton’s contention that every body has inertia and that the only way it moves is in response to some external force. When that force is zero, there is no motion.2

![Figure 7-2: Flow Between Water Tanks of Different Elevations.](image)

To adequately describe flow in the aquifers in the groundwater system it is necessary only to introduce a minor addition to the discussion above. In essence, the soil occupies some of the space which would normally contain water. It is as if we examined the tank in Figure 7–1 and, to represent soils, threw into the tank a number of balls to represent a porous medium. In such a case, the flow of water into the tank causes greater increases in the head since some of the space is already taken up. Similarly, in Figure 7–2 we should fill both tanks and, more importantly, the channel with balls. It is then evident that these will considerably restrict the flow of water from one tank to another (see Figure 7–3). In surface waters, where the flow is relatively free, water movement from one place to another in the system takes place over periods of hours which corresponds to a tidal cycle. However, in the groundwater system the flow is so restricted that it requires a much longer time. It should be kept in mind in reading later sections that this discrepancy in the time required for differences in head to be equalized is one of the most significant differences between surface and groundwater models.

Let us try to make the discussion more precise. The flow of fluid in a porous saturated medium is dominated by two features not present in the open surface case. First, there is porosity. This represents the fraction of bulk volume in a tank which consists of open pores accessible to fluid storage. Porosity can be defined by the number \( \varepsilon \), where

\[
\varepsilon = \frac{\text{volume of pores}}{\text{bulk volume}} \tag{7.2}
\]

It is apparent that \( \varepsilon \) is less than one. Although in open waters \( H \) is volume divided by surface area \( A_S \), in porous medium \( H \) equals the volume divided by \( \varepsilon A_S \) and therefore is larger. Second, the interconnected pores act as thin capillary tubes. This creates a condition known as viscosity. Viscosity, which we denote by \( \mu \), describes the fluid resistance toward flow past a boundary. Permeability, indicated by \( k \), describes the restriction upon flow imposed by the porous medium itself. These fluid and medium properties can be combined to describe the overall hydraulic conductivity, or ability of water to flow in the groundwater system.3

Two very similar pairs of relations are now derived; one for open water flow and the other for flow in a porous medium. The first set will be used in surface modeling and the other in groundwater modeling. Each pair of relations expresses the temporal change in hydraulic head of a water body in terms of velocity of flow into that body, as well as the temporal change in velocity between two parts of a water body in terms of the differences in head and the resistance of the medium. Thus the essential output of the hydrodynamic equations are a knowledge of how \( H \) and \( u \) vary over time in relation to each other.

The only major difference between the arguments used in the derivations above and those which would be used in a more exact mathematical approach is that the macroscopic view of tanks and conduits is replaced by a study of the instantaneous changes of infinitesimal slices of volume. This leads to what is known as partial differential equations of motion for \( H \) and \( u \) which are valid at any point of the water body rather than to an entire water column as above. Such equations are not used in this report.

Let the symbol \( \Delta \) refer to an incremental increase or decrease of some quantity. In the discussion of rates of change of \( H \) and \( u \) over some time \( \Delta T \) denote the interval of time in which things happen and let \( \Delta H \) and \( \Delta u \) refer to the corresponding jump in the value of \( H \) and \( u \). If \( \Delta T \) is the increment from some previous time to the present, then our two basic relations relate \( \Delta H \) to the previous value of \( u \) and \( \Delta u \) to the previous value of

![Figure 7-3: Flow in a Porous Medium. For a Given Volume of Water, Both \( H_1 \) and \( H_2 \) Are Greater Than in Open Water Since the Medium Occupies Some of the Bulk of the Tanks.](image)
the head difference $H_1 - H_2$. This is an important observation because it says that one can compute present values of $H$ and $u$ from previous values since $\Delta H$ and $\Delta u$, the incremental change, is now known. Following this argument back to some initial time we see that if head $H$ and velocity $u$ are both specified initially, all future values are predictable. Note also that in the process of obtaining these future values, $H$ and $u$ are both dependent on each other. Taken together they tell us about the movement of a water body over successive intervals of time. In essence this is the core of what is called hydrodynamics.

In order to illustrate what can happen over successive intervals of time (where $\Delta T$ represents a few hours, for example) let us consider the exchange of water between two tanks as shown in Figure 7-4. This could represent “snapshots” of water elevation in two parts of a bay due to oscillations which result from tidal flow. Once set in motion, the water’s momentum causes it to move back and forth. If the external influences of the open sea were suddenly to cease then the oscillations would eventually damp down (due to frictional resistance) and the system would come to rest. The second fundamental relation is but a restatement of this fact. Except for the external forces which intrude, this is exactly what would happen in practice.

In the discussion of surface and groundwater models later in this report the simple ideas expressed above will prove to be the key ingredients to understanding how the models work. There is also another idea which we need to treat here since it forms the basis for water quality modeling and for the water budget model. This is an extension of conservation of mass, as used above for water, to include all substances which enter into the water due to natural and man-made discharges. If these substances, such as nitrogen and phosphorus, do not decay over time then in the absence of any external influences which may add or subtract from the total amount, the total mass remains unchanged over time. This statement is completely analogous to the observation made earlier about the water in a bathtub and the way it is used is to remark that the only changes which can take place in a water body, other than external “sources” or “sinks,” is that the water constituent shifts in concentration from one place to another with the total amount remaining constant. Therefore if one adds up all decreases in the concentrations in different parts of a bay, these must equal the sum of increases in the remaining parts of the bay. This fact forms the basis of an accounting procedure known as “mass balances.” The concept of “water budget” is also very similar, as will be shown.

There is one last remark to be made in this connection. If a substance is inserted into a water body it will move about due to two forces. The first, called advection, is the displacement of the constituent from one place to another because the water in which it finds itself happens to be moving. If a dye is put into a bathtub and a wave is generated, then the dye will tend to migrate in the direction of the wave movement. The other force, called diffusion, is the propensity for a substance to move from a region of high concentration to a lower one at a rate proportional to the difference in concentration between two points (this is similar to water itself wanting to move from a high point to a lower one at a rate proportional to the difference in head). This results in a spreading out in all directions of the original mass until it reaches a state of total dispersion. Again, if you place a dye in a tub, even in a perfectly motionless tub, it will move away and thin out from where it was put in. One of the main goals of the surface and groundwater models is to capture the effects of advection and diffusion to describe water quality in a water body. To do this, hydrodynamic considerations are important since water movement determines advection.

To summarize: essentially two basic facts were reviewed which dominate the structure of all the models in the 208 project. Each states the premise that in the absence of any external forces a water body (or of a conservative substance within it) reaches a state of equilibrium in which total mass remains unchanged and in which velocity of flow is zero. When this is not true, the lack of equilibrium is proportional to the sum of the forces which act on the body or on the substances within it.
7.2 SURFACE WATER MODELS
  7.2.1 Water Quality

  7.2.1.1 Hydrodynamics in an Estuary. The Long Island waters are
contoured by a number of bays and estuaries. Pollutants which enter these waters
are largely dispersed through the flushing action of the tides. The purpose
of the Tetra Tech modeling effort is to determine the concentrations of the various
constituents which enter the waters both spatially, and over time, as a
result of this tidal action. In order to understand how this can be done, it will
be convenient to first describe the unsteady flows of the coastal waters and
then to relate this to the procedures used in computing water quality. The
first part therefore is based on the discussion of hydrodynamic relations
which were developed in 7.1.

  A number of plausible assumptions are made in the formulation of the
model in order to make the analysis more tractable. To begin with there is
the problem of spatial dimensionality. If one assumes the water body being
studied is completely mixed in the vertical direction (no differences in
salinity, for example) then it is possible to consider the body as a two
dimensional expanse of water in which all changes take place in the planar
directions. For bays and estuaries in which water depth is shallow compared
to the planar width this assumption is reasonable. However, even the two
dimensional description is generally too complicated. Therefore we further
reduce this problem into a set of much simpler one-dimensional descriptions.
How this is done will be shown in a moment. It is worth noting immediately
that what makes this reduction from a single hard problem into many simpler
ones useful is that it is typically more efficient to code a computer to carry
out a large number of straightforward computations than to have it perform a
single complex problem.

  The model divides the water body in question into a set of “nodes”
which are joined together in a network by a series of interconnecting “links.”
Each node represents a completely mixed “tank” of water having a specified
surface area $A_s$, an elevation $H$ (relative to some arbitrarily chosen datum)
and a volume $A_s H$. The nodes are where water is stored in the bay (from now
on “bay” refers to any coastal water body and may in fact be an estuary) and
the links represent pathways through which water is conveyed from node to
adjoining node. Each channel preserves information about flow $Q$ and velocity
$u$ of water movement as well as the coefficients of resistance to that flow; it
also has a specified length $L$ and a cross-sectional area $A_c$ which varies along
its longitudinal dimension, as well as a specified width. Figure 7–5 exhibits a
grid network of nodes and links overlayed on a typical bay while Figure 7–6
displays one possible representation of a node and of a channel. When there
are $N$ nodes in the network then it is useful to label them in some order.

  Since flow is along the longitudinal direction of each channel the character-
istics of motion in a channel are one-dimensional. The price paid for this
reduction in dimensions is that there are now a large number of channels to

FIGURE 7–5  Node and Channel Layout for the Manhasset Bay System.
Suppose there are m channels connected to the given node which is labeled as node s as shown in Figure 7–7. The channels link to other nodes which we can arbitrarily index by j where j runs between 1 and the number m. At any given time the average flow in each channel is written as \( Q_j \) (away from or towards node s). In addition at node s there may be some additional sources or sinks (that is, additions or losses) of flow which we write as \( Q_{s, \text{in}} \) and \( Q_{s, \text{out}} \). For example if node s is near the shoreline then \( Q_{s, \text{in}} \) could represent tributary inflows or sources of waste discharges from a treatment plant or some measure of runoff along the embankment while \( Q_{s, \text{out}} \) could denote water diversions and evaporation. Groundwater inflows and even rainfall can also be accounted for at each node. This allows us to extend our basic relation about head to say that at each node s, \( \Delta H \) is proportional to the velocity in addition to the net flow \( Q_{s, \text{in}} - Q_{s, \text{out}} \). The second fundamental relation of section 1 does not require modification except to make the dependence on the node arrangement more explicit (see Figure 7–7).

**FIGURE 7–6** Typical Section of an Estuarian Node-Link Network.

Consider simultaneously but, as said earlier, this is preferable from a computational point of view. Collectively, this network of one-dimensional flows constitutes a proxy for the original problem. It is apparent that in bays of irregular shoreline geometry, some care must be taken to account for changes in flow patterns caused by these irregularities by choosing a suitable placement of junctures for the links. If there are natural channels of flow then links of appropriate length and width are inserted to represent this. The grid is not uniform, being densely laid out where water quality or hydrodynamic conditions change rapidly and being more sparse in those portions of the bay where it is likely that there will be less action. The nodes themselves will vary in size (that is, in surface area) depending on the accuracy desired. Since a node represents a completely mixed water column, the extent to which the constituents of the water are actually homogeneously distributed is a determining factor in how big or small such nodes should be. Therefore careful judgement must be made in the preparation of a grid configuration prior to any computations. This is one of the places where the skill of the model maker is revealed.

Before continuing it must be noted that the channels are considered to extend from the center of one node to the center of the adjacent one. The nodes or water tanks are all contiguous and together cover the bay. Hence all exchanges of constituents within a node resulting from the flows into and out of the tank are assumed to occur at its center.

Let us now consider a typical node, having some fixed surface area. Over time the only thing that varies within it is the elevation or head of the water. Then as known from section 1 the rate of change of \( H \), denoted by \( \Delta H \) during the interval \( \Delta T \) is proportional to the velocity \( u \).

**FIGURE 7–7** Typical Flows as Seen in Part of an Estuary Node-Channel Layout.

**7.2.1.2 Solution Procedure.** The detailed numerical procedure employed to obtain solutions to the basic hydrodynamic equations is described in "Documentation Report for the Estuary Water Quality Models" (Tetra Tech, May, 1977). Herein, it is only important to summarize the elements required in order to carry out these computations. To begin with the grid network must first be specified. This means that the geometric layout must be given, and also the values of surface area \( A_s \) for each node and the length \( L \) and widths of each channel.

In addition boundary and initial conditions must be specified. The solution of the equations begins at some initial time and proceeds over a set of incremental steps whose duration \( \Delta T \) is usually of the order of an hour or more. All those nodes having a known inflow \( Q_{\text{in}} \) and outflow \( Q_{\text{out}} \) must have these values given at each time step for the entire computational horizon which is being considered for running of the model (the question of time span will be reconsidered in more detail later). These values may of course be constant in some problems. Generally speaking, the nodes so specified are located along the boundary of the bay. Also those nodes which interface with the
open sea (Long Island Sound and the Ocean) will need to have the elevations \( H \) given corresponding to the actual tidal oscillations which prevail there. These are similarly given at discrete time intervals over an entire tidal cycle.

The knowledge of head \( H_j \) at the boundary nodes is used to derive the elevations at all other nodes. In order to carry out the numerical procedure it is necessary to also specify in addition to the boundary conditions described above, a set of initial values for the velocities in each channel and a tidal elevation of other than seaward nodes (these may be taken initially to be zero). As the computations proceed these values repeatedly adjust and after running through several tidal cycles one begins to notice that successive values of velocity \( u \) and head \( H \) over two consecutive cycles no longer differ significantly. At this point equilibrium has been achieved in the model and the numerical procedure terminates. A typical plot of elevations at two nodes in an estuary would be as shown in Figure 7-8 where a shift in phase and a difference in amplitude can be seen. This is largely due to the drag induced by the channel resistance \( R \), especially during ebb tide when the estuary is shallow.

![Figure 7-8 Phase Shift of Tidal Oscillation in an Estuary](image)

After equilibrium has been achieved in the computations one may wish to compare model results with actual field data obtained at monitoring stations in the bay. Tide gauges, for example, will give elevations which occur at various locations and these can be used for comparison. If the model results differ substantially from this data then the modeler engages in model calibration. The values of the resistance \( R \) along each channel are used for this purpose. By adjusting these values for each channel and re-running the model computations a greater or lesser degree of compliance to field results can be achieved although there is a reasonable limit to the amount of adjustments which are possible. Experienced modelers can usually do this without too many trials. If, however, compliance is still not satisfactory it may mean that the original grid of nodes and links does not supply enough detail. By enlarging the grid geometry by the inclusion of more nodes, it is possible to improve overall accuracy. However, there is a purely technical snag here which should be pointed out. For the computations to remain stable it is necessary to require that the time step \( \Delta T \) be smaller than some factor of the smallest channel length in the network. Therefore as grid size is made more dense in portions of the bay the computation work is increased because of the extra nodes and links, and the number of time steps of size \( \Delta T \) also decreases. There is therefore a practical limit to the amount of detail that can be expected from the network configuration itself.

After calibration it is usual to further check the model against a new set of field data (assuming these are available). This process is known as model verification, although in practice it is often difficult to distinguish the exercise of verification from that of calibration. If the model results fit the new data set reasonably well then this lends confidence in the use of the model as a predictive tool.

At this point one can use the hydrodynamic information so far obtained to derive an understanding of water quality. This is discussed in the next section.

7.2.1.3 Water Quality in An Estuary. When pollutants enter a bay, they are dispersed, largely as a result of the action of the tides. Some fraction is carried out to sea and the remainder is spread throughout the water body through mixing. In addition some of the constituents of the bay will decay over time. The relationship between the concentration of a constituent and the forces of tidal flushing and decay follows.

The model formulation assumes the same node/channel configuration of the bay as in the hydrodynamic equations. Several assumptions are made in addition to that of complete vertical mixing in each node. These will be described as we proceed. Vertical mixing is somewhat problematic for those nodes in the vicinity of an outfall or at the head of a fresh water tributary. Here the concentration of an effluent or, in the case of a tributary stream, of the salinity is non-uniformly dispersed and forms a plume. As a result there is some stratification of constituents in the water column. However these will be considered to be negligible effects in the model.

As a result of tidal oscillations, the concentrations which spread out from the boundaries of the bay are folded back and forth on each other. This dispersion is caused by two separate forces as seen in 7.1. The first, called advection, is the result of material being transported through the displacement of the water itself. The second, called diffusion, is the natural process by which any concentrated material tends to distribute itself uniformly throughout the water body.

Two kinds of constituents are handled by the Tetra Tech model—
conservative and nonconservative. The first group are the conservative
components of salinity, total nitrogen, and total phosphorus. 'Conservative'
refers to the fact that they do not decay in the water over time. The non-
conservative components (that is, those which decay due to bacterial decom-
position or which are otherwise removed due to chemical reaction and sedi-
mentation) are total and fecal coliforms, nitrogenous and carbonaceous bio-
chemical oxygen demanding materials (or simply BOD for short), and
dissolved oxygen (or DO). Temperature in the water can also be modeled as a
decaying substance but is not considered here. The nonconservative
substances are assumed to decay at a rate which at any instant is proportional
to their concentration at that time.

Let a constituent be measured by concentration c (kg. per cubic meter).
Then its total mass in a node of volume V is simply V times c. Since the
principle of conservation of mass says that the rate of change of a non-
decaying substance c in the bay is zero (total mass is conserved), therefore, if
one observes changes in mass from node to node it must be due either to
advection or diffusion or decay or to other special sources or sinks. That is

\[
\text{Rate of change of } c \text{ in a node} = \text{Advection} + \text{Diffusion} - \text{Decay} + \text{Other Sources} - \text{Other Sinks} \quad (7.3)
\]

Relation 7.3 is called a mass balance relation because it says that in
order to know the total change of a constituent in a node during an interval
of time \( \Delta T \) one must add up all amounts coming in and subtract all
amounts which leave. If these two amounts balance each other then the total
mass of the constituent remains unchanged and, conversely, if no change in
mass takes place then losses must balance the gains (principle of mass con-
servation). Of course there are N such relations to satisfy for each constituent
if there are N nodes in the bay.

The advection terms in equation 7.3 reflect simply mass transport of
the constituent from one node to another during tidal flushing. The concen-
tration in the nodes shown in Figure 7–2 will depend upon volume of water in
the tanks (here again a simplified view of the water body as a system of
tanks with connecting conduits is adopted). The diffusion terms in equation
7.3 reflect the tendency of the difference in concentrations between the two
banks in Figure 7–9 to equalize. When \( c_2 \) is greater than \( c_1 \), there is a pollu-
tant dispersion in the direction shown. This is similar to the spread of dye
dropped into a bathtub as discussed in Section 7.1. The rate of diffusion is
dependent on the level of concentrations between tanks. Where substantial
differences in concentration are found, rapid diffusion is present. The overall
rate of change of concentration is governed by an empirically determined
diffusion constant which is labeled \( k_D \). This constant is discussed
further below.

For a nonconservative substance the decay rate is proportional to the
quantity of the substance which may be present.7 When the decay is zero it
signals that the substance is conservative.

![FIGURE 7–9](image)

Suppose \( C_2 > C_1 \), but \( H_1 > H_2 \). Then Advection and Diffusive Flows Are in the Direc-
tions Shown.

Sources include waste discharge outfalls, storm runoff, groundwater
inflow, rainfall, etc. Sinks refer to losses such as benthic uptake, water
pumping, and so forth. Although it is not explicitly shown here equation 7.3
can be further extended to include other terms of constituent loss or
accretion when these are applicable. In particular the equation for dissolved
oxygen would include first order relations for \( \text{re-aeration and photosynthesis} \)
(positive accretion terms) and for \( \text{respiration (negative loss terms)} \).8 Both
photosynthesis and respiration depend on the concentration of algae. One
can also add terms for loss of constituent if there is benthic uptake since this
acts as a sink.

It is important to note that since constituents such as nitrogen and
phosphorus are nutrients for the various microorganisms and other marine
life (especially phytoplankton) which inhabit the bay, one could usefully
include in the balance equation an accounting of how nutrients are used up
and later returned to the estuary by this marine life. In the process oxygen
levels will vary considerably over a diurnal cycle (that is, over a 24 hour
period) since photosynthesis depends on the amount of sunlight which
penetrates the water body. The phytoplankton are prey for the various
zooplankton and the populations of both fluctuate over time. Therefore the
terms used above in which photosynthesis and algal concentrations are
assumed constant must be considered as rough approximations. Benthic
uptake is also assumed constant. Benthic uptake represents chemical and bio-
logical oxygen losses to the bay bottom. An extension of the water quality
model to include these additional marine interactions constitutes what is
known as an ecological model. This will be briefly discussed at the end of this
chapter.

In order to solve equation 7.3 one must know, in addition to decay
constants, diffusion constants, and the like (all empirically determined)
the values of the volume of each node and the flows which determine
advection. Moreover these must be known for each successive interval of
time. But of course the flows and volumes (which are given in terms of head)
are already known from the solution of the hydrodynamic relations. One can therefore appreciate now why the Tetra Tech modeling process is necessarily a two-step procedure: water quality mass balance equations require hydrodynamic inputs.

7.2.1.4 Calibration Procedure. When the model is to be calibrated, one has available the various constants used in equation 7.3. Perhaps the most significant is that of $k_d$, the diffusion constant. It is worth taking a closer look at it. Mixing in a bay or estuary is subject to location. At or near the ocean outlet, most mixing is due to velocity differences which advect the constituents in the water in varying directions and at varying rates. This induces a kind of turbulence which causes the constituents to disperse. At the same time, the back and forth flow caused by tidal oscillations also results in mixing. However, as one moves further into the bay, these effects tend to lessen especially in estuaries where tidal flow from the ocean interfaces stream flow. At the interface we have a mix of salt and fresh waters and dispersion occurs mainly as a result of the increased salinity gradients, by which is meant that the stratification of the estuary into layers, due to differences in water density, causes mixing to occur between water at different depths. Finally as the fresh water zone is reached, all of the above effects abate and non-adveective dispersion will be minimal. Although density differences are supposedly ignored in the assumption of vertically mixed water columns, the constant $k_D$ can capture this effect indirectly. Tetra Tech proposes therefore that $k_D$ be computed as

$$k_D = k_{D1} + k_{D2} \Delta s$$

where $\Delta s$ is the salinity differences between two nodes divided by channel length. $k_{D1}$ and $k_{D2}$ are chosen so that where tidally induced and advective mixing predominates $k_{D1}$ is larger than $k_{D2}$ but the contrary be true where salinity effects begin to predominate. This is illustrated in figure 7-10. By suitable adjustment of $k_D$ one may adequately account for the actual dispersive effects in the waterbody. Again, this is a question of judgement and skill on the part of the modeler.

There is one more observation to make about the solution of the balance equation. It may happen that relations which apply to one constituent require information about another one. In this case, one has a coupled system and it matters which set of equations is solved before the others. This is especially true in the case of BOD/DO interactions where the amount of DO depends on the amount of BOD there is. The solution to the equation for DO therefore requires that the solution to that for BOD be obtained first.

7.2.1.5 Dynamic Versus Steady State. The above model formulation can be run in a computer in several different modes, each of which has its own virtues and limitations. These are steady state, quasi-dynamic and dynamic. Tetra Tech refers to these as Aqual and Dytqal. The most complex and detailed is the dynamic, or time varying, model in which changes at each node are followed at each time step of duration $\Delta T$ over an entire tidal cycle or indeed longer. The time step can be as small as one wishes depending on the need of detail that is desired but of course computational expense usually limits $\Delta T$ to be of the order of one or several hours. If no essential changes take place in a bay over repeated tidal cycles then it suffices to run the model for one or two such periods. However, the major utility of the model is to account for short term and rapidly changing boundary conditions which may induce variations in water quality over a longer period of time. For example, stormwater inputs or intermittent and unsteady effluent discharges will require source flows and concentrations to be altered over a sequence of time steps at the boundary nodes. The resulting concentrations of the several constituents in the water may fluctuate considerably for a long stretch of time and over a period of several days or weeks may never actually settle down to equilibrium values. In addition the values of temperature (which affects the decay constant $k$) and photosynthesis and respiration rates (which vary over a diurnal cycle) may also be varied over the time steps allowing for a better accounting of nutrient interactions with marine life. Of course this is still only a proxy for a more comprehensive ecological model but in many cases it may suffice for an adequate prediction of water quality changes.

However dynamic models are expensive to run, as we said, and so one may on occasion more usefully employ a simpler version known as the tidally-averaged, or quasi-dynamic, model. This consists of computing average water quality conditions over an entire tidal cycle and then monitoring subsequent changes from cycle to cycle. This approach is especially apt when one wishes to follow changes which may occur in the bay over a long term period, from several weeks to several months. The results of this hydrodynamic model are first calculated as in the dynamic version. Then instead of using the resulting flows and elevations at each time step, the computed values are averaged over all increments used in the tidal cycle. At the same time any inflows and outflows of constituents, which vary over time, are also averaged over each successive cycle, and the concentrations which result from the mass balance equations then represent average water quality conditions over consecutive time periods, where the duration of each

FIGURE 7-10 Dispersion Rates Versus Distance.
period is a cycle. If one wishes, in fact, the averaging can even be done on a daily basis. Although this would lead to less accurate portrayals it does permit one to more economically track the changes in pollutant levels over longer time horizons. This model is dynamic in the sense that it does allow for gross temporal changes although it does sacrifice the detail of the more truly dynamic version discussed above. An even simpler approach is the steady state model. This too is tidally averaged but it further assumes that no significant changes occur from cycle to cycle. In essence this assumption implies that the bay has come to equilibrium with respect to the pollutants and other constituents which enter the water and that all transient effects due to past disturbances have died out. This kind of situation would prevail in the long run if ideally the initial influences on the bay (in terms of inflow, outflow, age concentrations, and the like) had dissipated and no further changes took place. Obviously this is a considerable simplification but it does allow one to make some rough comparisons between alternative wastewater abatement schemes or to estimate the approximate effect of the water body of different boundary conditions. Of course this approach would be inadmissible if one were to track a series of storm events.

Since time no longer plays a role in steady state modeling both the hydrodynamic and mass balance equations become considerably simpler to handle. This is because the left sides of the equations can now be made to vanish since the rates of change over time must be zero. Needless to say, if the water body is in fact subject only to slow changes in pollutant levels, and if it is relatively undisturbed as in some less populated coastal areas, the steady state modeling may be the appropriate and only sensible tool to use.

One should also note that after a sufficiently long time period has been simulated with dynamic or quasi-dynamic models, all changes that occur in successive time periods may become increasingly negligible. In this case the dynamic models eventually reach a condition of steady state. What the steady state model does is to regard the future as now.

A technical note should be interjected concerning steady state and quasi-dynamic models. Since both are tidally averaged, the dispersion of pollutants due to advective forces is less significant than in the dynamic model since only average values are used and the larger velocity changes at extremes of the tidal cycle remain underrepresented. In order to compensate for this, it is expedient to increase the value of the dispersion coefficient K*D in order to better approximate the actual mixing which occurs in a cycle. This procedure is followed by Tetra Tech.

One can summarize the discussion of the Tetra Tech modeling effort by noting that the water quality component is a set of equations which are simply adjoined to the equations for hydrodynamics. The latter describe how water flows into and out of the bay while the water quality model tells how something which is injected into that water will disperse because of the back and forth motion. The relation between the two is graphically illustrated by Figure 7–11 which is adopted from the Tetra Tech user’s guide.

7.2.2 Limitations and Uses of Estuary Model

The discussion has attempted to underscore the most significant assumptions made in model development. Each such assumption is a shortcut to reality, and it reflects an implicit belief that in spite of such simplifications all essential relationships which occur in nature will be preserved by the model. Depending on the context this may or may not be true. Credibility in a model rests on the extent to which the user is willing to accept such assumptions as plausible and the extent to which model calibration and verification demonstrates the ability of the model to portray actual conditions in the water body considered.

Some of the simplifications made are explicitly stated by modelers, but often there are others which appear only implicitly. For example, although complete vertical mixing is clearly part of model formulation for surface water quality, much less explicit is the fact that the equation ignores the effect of velocity differences over the length of the channel. Fortunately, this is not a very significant omission.

Many assumptions have been made beginning with the choice of a particular geometric configuration for the grid layout, the size of nodes, the choice of time step, the use of first order reaction terms for decay and photosynthesis of material (among others), the setting of dispersion K*D as a linear function of salinity differences and so on. Also, although some of the parameters used as constants in the models are better known than others, all are but approximations to actual values which may vary both spatially and over time. Similarly, boundary conditions on distributive inflow sources, dis-
charge rates of treatment plants, and tidal depths at the sea are only known approximately. Therefore the models cannot be expected to reproduce exactly the behavior of the water body. However, experience with the models has demonstrated that they can be usefully employed to obtain acceptable measures of actual water quality conditions and, therefore, to predict future conditions under a variety of changes.

Calibration and of course the eventual verification of a model does depend on having available a data source on actual water quality and hydrodynamic conditions over time at a variety of sampling stations. These are used to adjust or “tune” the model and then to compare model outputs to actuality. Such data is, however, hard to come by and is sometimes incomplete or of poor quality. For this reason adequate validation remains a difficult and time consuming task, although in the 208 project this does not appear to be a serious problem.

It may be argued that by increasing model detail, accuracy and confidence can both be improved. The most cogent reason for limiting such detail is however to achieve a level of complexity which is compatible with the available computational capabilities. Also, complexity for its own sake is illusory since the information needed to closely model certain interactions is often not known well enough. In the Long Island 208 the Tetra Tech modelers have avoided this pitfall.

In the context of the 208 project the Tetra Tech models are used to provide information on water quality as part of the general assessment of environmental conditions and structural alternatives. They play a crucial role in quantifying the impact of sewage and runoff on the coastal water. What is more, the models can compare alternative tactics for abating this impact through sewage treatment, recharge, and shifts in land use.

The basic input to the models is information generally provided on discharges into the rivers and bays in terms of flows from various sources (streams, point discharges, runoff, and so on) and in terms of the pollutant loads carried by these flows. As the reader now appreciates, this information constitutes much of the boundary data for the models.

The models can be run in three modes, as discussed earlier. The dynamic, or time varying mode, is especially useful in studying the short term transient effects of storm runoff and in computing the diurnal variations in dissolved oxygen which result from changes in algal activity. The information is provided over small time steps but this of course precludes model runs which exceed more than a few tidal cycles. For longer stretches of time, a more flexible mode is quasi-dynamic or tidally-averaged dynamic. Here one can follow slow changes in pollutant levels as boundary conditions gradually change over a period of weeks and months. Changes which occur over a tidal cycle are averaged out so that the fine detail of a more fully dynamic model is now lost. Finally, in the event that discharges to the water body are reasonably constant over a long period, or when one can assume that the water body has come to equilibrium with respect to the flows and loads which enter it, a steady state model is more appropriate. Here not only is information averaged over a tidal cycle, but it also no longer varies from cycle to cycle. Tetra Tech has found all the above modes useful in its work with some modes being more appropriate than others in certain bays. In all bays, however, steady state runs were provided in addition to some dynamic runs in select areas.

The actual use of a model depends not only on the time scale and level of detail required but also on whether it is to be used as a descriptive or predictive tool. As a descriptive tool, one uses the model to explain existing conditions in the water bodies. For example, to understand the extent to which tidal fluxes will account for larger concentrations of some constituent in one part of a bay over another. This kind of information is generally part of the process of model verification. Another use is as a predictive tool. Here one wishes to know what future changes will occur in the water body as a result of present conditions. For instance, what are the nitrogen concentrations in a bay tomorrow as a result of a storm surge today? Here one predicts future changes in a way that clearly is impossible by observation of the bay itself until after the storm surge has occurred. The value of a well designed model as an oracle largely accounts for the fascination such quantitative tools possess. Finally, the models can be used for purposes of optimization. Not only do we wish to predict future conditions but one would like to shape that future by manipulating the present. Thus, for example, by varying the outfall location, size, and level of treatment of a plant, one gets different eventual concentrations of BOD in portions of the bay. Of these many options on the treatment plant, one of them will clearly result in a lowest BOD level at some future time. If one disregards cost and other factors which may actually inhibit the selection of this option, one now has an optimal way of influencing the future of the bay. In the 208 study all the above model uses are present but in terms of developing a final waste management plan it is the predictive and optimization modes which will dominate as a tool.

7.2.3 River Models

Tetra Tech has developed appropriate hydrodynamic and water quality models for the Carlls and Peconic Rivers. This section is much shorter than preceding ones since the essential ideas are really special cases of what has gone before, and the equations used are very much simpler.

Once again one employs a node/channel schematization with the same assumption of complete vertical mixing. However, now each node is linked to only two adjacent nodes (as shown in Figure 7-12) in a simple linkage which follows the course of the river. Each channel links only two nodes which are labeled \( j = 1, \ldots, N \) starting at the river source. The channels are similarly labeled. The model structure will allow two upstream channels to converge at one node. Likewise the downstream flow from one node can diverge into two channels. These features will allow one to model river tributaries, rivers with branching structures, and two flow channel systems.
Once flows are known at each node, one refers to empirically determined *stage-flow relationships* (that is, observed values of river depth at different river locations for varying flow rates) to obtain estimates of river depth in a channel. The depth in the node is the average of the channel depths at each end of the node. River width is determined from the channel cross section. Width of a river node is the average of the channel widths. The channel cross sectional area is calculated as a trapezoid. The velocity \( u \) in a channel is then simply flow divided by area \( A_c \), as in the previously discussed estuary model.

The channel/node scheme should roughly correspond to changes along the river. Each stretch of river from one tributary juncture to the next should certainly include at least one node. Also, major changes in the river's configuration (such as bends) may suggest additional node detail.

Using this hydrodynamic information, the water quality component of the model computes concentrations of various constituents exactly as before except that now the question of dispersion by diffusion is far less significant than the result of the advective unidirectional flow. Of course, decay of nonconservative substances, benthic uptake and the like, all have their effect and are taken into account in balance equations which resemble those of the estuary models. It is not necessary to repeat this description here because it is a slight variant of the previously derived equations. It does appear, however, that Tetra Tech does incorporate the effect of heat losses and gains in the river across the air/water interface in a more pronounced way than in the bay models. Temperature, together with re-aeration, photosynthesis and respiration of algae, and BOD loading, are all factors which contribute to the dissolved oxygen deficit of the river. Each of the balance computations involve the volume of a node, but these are easily computed from the depth, width, and channel length.

If inflows and outflows along the river vary with time then the hydrodynamic computations would be carried out over a set of successive time increments of duration appropriate to the rate at which significant changes occur (hourly, daily, or even longer time spans). When all flows and loads are constant over time and when there is no change in constituent concentrations, one obtains the steady state modeling situation referred to in the discussion of surface models. However, in the event that one wishes to track variations over a diurnal cycle, it is possible to do so by choosing suitable small time steps in the water quality mass balance equations. This is referred to as the dynamic water quality model by Tetra Tech.

### 7.2.4 Phytoplankton Model

Some consideration is being given to an ecological assessment of how structural and nonstructural abatement alternatives impact on the bays. This work essentially extends the water quality considerations which were treated earlier. In the estuary model it was noted that dissolved oxygen is dependent
on concentrations of algae in the water, since these microorganisms both produce and utilize oxygen through the process of photosynthesis and respiration. This concentration is assumed to be a known quantity which is given as one of the model inputs. What this explicitly ignores, however, is that the algae concentration is itself dependent on other factors which include not only the level of nutrients in the water, such as the various nitrogen and phosphorous compounds which form part of the waste loads discharged into the bays, but also on the level of oxygen itself. In effect, one has a coupled system of mutually interacting influences which are accounted for in the Tetra Tech models only in an implicit way.

An ecological model attempts to capture some of the interactions which take place over time between nutrients and organisms in the water in order to give a better grasp of the diurnal fluctuations which do occur, and in order to predict the conditions which can lead to eutrophication. The process of eutrophication is characterized by an excessive growth of algae ("blooms"), an accumulation of organic debris, as well as low oxygen levels.

The model uses a set of mathematical relations which describe how phytoplankton (a class of microorganisms which include algae) are consumed by herbivorous animals (the zooplankton) and how the detritus of dead matter and excreta is recycled through bacterial decomposition to provide a new supply of nutrients for the phytoplankton. These relations are conceptually similar to the ones used in the water quality model in that each of them give the rate of change of one constituent over a time interval $\Delta T$ in terms of the values of other constituents. For example, the rate of growth of zooplankton is proportional to the population level of phytoplankton, with a growth rate which measures the efficiency of feeding (zooplankton graze more than they actually consume). In this way one in effect writes down a set of equations which simply extends and complements the ones given in the earlier model. Figure 7-14 below displays a schematic of the various interrelations which are dealt with in an ecological model. Note that oxygen is used in the process of bacterial decay of waste loads generated either by man induced wastes or through the natural processes of organic decay in the waters. The nutrients on which the phytoplankton feed are provided either through these same man-induced loads which enter the bays as sewage and runoff or through the natural recycling process which transforms decomposed material into a new supply of nutrients. Therefore, one deals here with a nearly closed system except for such external factors as waste discharges or temperature and sunlight (each of the last two affect both the rates of photosynthesis which generates oxygen) and of respiration (which absorbs oxygen). Incidentally, large temperature increases can also occur from man-made heat discharged from power plants.

The complete loop is generally self limiting in the sense that under "normal" conditions all interactions balance each other and therefore no single constituent can grow in an unlimited fashion. An unbalanced situation, triggered perhaps by a large input of organic wastes during a storm surge, can cause excessive algae growth, and a mathematical model allows one to roughly predict what the extreme conditions must be to cause such behavior. In this way another tool is added to the arsenal which is available to manage water quality in surface waters. One should caution that, as with all modeling, it is clearly impossible to capture all of the interactions which actually take place and that the equations can at best approximate reality. Moreover, many of the parameters of the model (such as grazing rates, growth rates as a function of sunlight penetrations, and so on) are imperfectly known. Nonetheless, with proper interpretation of the results useful insights can be obtained. The environmental consultant intends to utilize these ideas as part of the overall environmental assessment in combination with other techniques and approaches. The model, in fact, is essentially a modification of the classical mathematical relations which describe interactions between predator and prey. In this case the predators are zooplankton, and prey are phytoplankton which feed on a nutrient source. In the absence of predators the growth of the prey is limited by the population carrying capacity, denoted by a value $k_A$, which varies with the level of nutrients in the water. The predators grow at a rate which is proportional to $AB$, where $A$ is the concentration of prey and $B$ is the concentration of the predators themselves. (The units of $A$, $B$ are biomass per unit volume). However, these organisms also have a carrying capacity which limits their productivity and in the model an additional curtailment on growth is imposed which is based on the idea that predators reach a level of satiation in their feeding. The various feeding rates and the satiation constants are determined empirically.

FIGURE 7-14 The Ecological Cycle in Estuaries.
7.3 GROUNDWATER MODELING

7.3.1 Introduction

The groundwater modeling studies carried out for the 208 project by Princeton University and the United State Geologic Survey are oriented toward an understanding of groundwater flows and pollutant dispersion in the aquifers which underlie Long Island, as shown in Figure 7–15. Model development is similar to that described in Section 7.2 for surface waters but with two important distinctions. The geometry and physical characteristics of the soils which constitute the aquifers act to impede and redirect the fluid flow. Consequently, it becomes necessary to accurately represent the interaction between the water and the medium through which it flows. In addition, whereas vertical mixing in surface waters permits reduction to planar flow, here three-dimensional water movement is considered. In reality, the vertical flows are considerably smaller than horizontal and a simplified picture may be constructed in which an aquifer is represented as a series of horizontal planes, or layers, which are then linked together in the vertical direction. As in the case of surface waters, the hydrodynamics of water flow is first modeled and pollutant dispersion, or water quality modeling, is then based upon the hydrodynamic solution. The general character of both hydrodynamics and pollutant dispersion follows the discussion in chapter 1 of this report, though the analogy of the use of tanks interconnected by pipes is replaced with a more specific description of how the groundwater system is represented for model development.

In this description of groundwater flow, it is assumed that the aquifer, or medium, is saturated with water. The medium gives rise to several important features of fluid flow not present in the earlier discussion of surface waters.

Since the material of the medium takes up some of the space in a given total volume, the remaining open pores available for the storage of water are characterized by a porosity \( \epsilon \). In addition, empirical studies show that water flows through the medium as if it were moving through long capillary tubes. This creates a condition in which the medium is partially restrictive of the water flow, and this is described by permeability \( k \). Within the thin capillaries, viscosity \( \mu \) of the fluid establishes resistive forces which dominate the flow. The porous structure of aquifers is also highly anisotropic. That is, under specified pressure differences the velocity of groundwater flow will vary widely with direction, often by more than an order of magnitude. Flow velocities are greatest along more or less horizontal planes in the aquifers and least in the vertical direction. Finally, these primary physical parameters which govern groundwater hydrodynamics—porosity, permeability, viscosity, anisotropy—are all dependent upon the specific aquifer (since these have different soil types) and upon spatial distribution within an aquifer both horizontally and vertically. Consequently, while the model equations will have the same form from place to place within an aquifer, the values of the governing parameters will need to be specified in great detail.

Two fundamental relationships are required to describe groundwater flow which are analogous to the hydrodynamic equations for surface waters. First, the velocity of flow is driven by differences in pressure, or head, from one point to another in the aquifers. Second, hydrostatic head charges with time as the flow velocity transports fluid from one place to another in the aquifers.

Therefore, two equations are required to determine the two variables of head (pressure) and velocity. The basic form of the equations was discussed in Section 7.1 and, here, they are made specific to groundwater.

For convenience in model formulation and computation, consider an aquifer divided into layers of square nodes of finite volume interconnected by channels. For the present, the vertical coupling between layers is ignored, to be added as a source term later. In the simplified illustration of nodes coupled together in Figure 7–16, a change in head (water elevation or pressure) at node 1 results from the net flow of fluid into the node. Since the node surface area \( A_s \) remains constant, the volume change is due to change in head. Compared to surface waters, the porosity \( \epsilon \) (which is less than or equal to one) implies larger head changes for the same net inflow into the node. In addition to flow of water from one node to another due to differences in head, there may be various natural and/or man-made mechanisms for flow into or out of the node. Consider for example the aquifer layer which represents the water table, or top of the groundwater system. This layer is subject to distributed recharge from natural sources as well as direct wastewater recharge at specific locations. Recharge flow \( Q_{\text{in}} \) (cubic meters per second) at each node must be specified and these contribute to increasing head. Similarly, there are natural and induced sinks or losses. Wells utilized for public water supply and other withdrawal purposes act to decrease head at each node they impact. In addition, there are groundwater seepages and discharge to streams and underflow to bays. The pumping wells and natural discharges are introduced at each node by terms \( Q_{\text{out}} \) (cubic meters per second). Finally, there is the coupling from one layer in the model to another. Since such flows are driven by vertical pressure differences in the aquifers, they are characterized by terms analogous to the horizontal velocity. It should be emphasized that such terms reflect the anisotropic flow and the vertical velocity will tend to be small in relation to the horizontal flow velocity. It proves convenient to define a specific storage \( S \) as \( \epsilon A_s / A_L \), where \( L \) is the channel length between nodes, which is roughly the node horizontal surface area divided by node volume. Specific storage \( S \) is therefore approximately the height of the aquifer layer. The change in velocity with time is limited by hydraulic conductivity \( K \), a team which combines fluid viscosity with the permeability of the medium. This constant reflects the role of viscous forces upon fluid flow in porous soils.

This discussion of groundwater equations may be summarized by noting that the starting point for both Princeton and USGS models is in the
FIGURE 7-15
Crosssection of Long Island Groundwaters.
hydrodynamic equations of motion for head and velocity. There are then two basic choices in an approach to construction of a model of groundwater movement. One can adopt a strategy of taking the basic hydrodynamic equations and develop a direct computer solution. Such computer solutions have the advantage that information in the model may be adapted from one area to another so that the model is relatively transferable. This is how the Princeton group approached it. An alternative is to build some type of physical structure which behaves in the same way as the groundwater system. This is the approach taken by the USGS. It should also be emphasized that a computer oriented approach facilitates the inclusion of pollutant dispersion, which will be discussed in connection with the Princeton University modeling effort, whereas analog models generally do not.

Before we proceed to detailed discussion of the Pinder finite element method and USGS analog model structures, it will be helpful to get some idea of the magnitude and relation between head H and velocity u and the parameters of specific storage S and hydraulic conductivity K. It can be shown that the solution to the velocity equation has the form shown in Figure 7-17. In physical terms, the velocity responds to a pressure difference by increasing to a maximum steady state velocity $u_0$ which is known as the Darcy Velocity (first measured in experiments by Darcy in the late 1800's).\textsuperscript{11} Using hydraulic conductivity typical of Long Island Upper aquifers, and a hydrostatic head difference of 2 meters between nodes of spacing about 2000 meters apart (as in the USGS model), the steady state velocity $u_0$ is about .01 meters per day and flow rate is therefore quite small. Note that the time $t_0$ required for velocity to respond to pressure difference is about $K/g$ from Figure 7-17. Using the hydraulic conductivity above and gravitational acceleration 9.8 m/sec$^2$, the equilibrium time becomes 9 microseconds. As we see below, this represents very rapid achievement of the steady state Darcy velocity compared to the time over which hydrostatic head differences are compensated.

\begin{figure}[h]
\centering
\includegraphics[width=0.8\textwidth]{figure7-17}
\caption{The Darcy Velocity in a Porous Medium.}
\end{figure}

Using the steady state velocity, the equation for hydrostatic head has a solution shown in Figure 7-18. The heads approach a steady state condition $H_2 = H_1$ in which there is no pressure difference and consequently no remaining flow. As noted earlier, the specific storage is approximately an aquifer thickness of about 50 meters. Then using hydraulic conductivity (10m/day) and node spacing (2000 m) from above, the time required for the pressure to go to equilibrium in Figure 7-18 is about 30 years. Hydrostatic head equilibrium is achieved slowly because viscous flow in the porous medium limits hydraulic conductivity, and hence limits velocity to .01 m/day. This brief description of the general character of groundwater flows is sufficient to proceed to specific model formulations.

\begin{figure}[h]
\centering
\includegraphics[width=0.8\textwidth]{figure7-18}
\caption{Head Change in a Porous Medium.}
\end{figure}
7.3.2 Analog Model

The USGS model of Long Island groundwater is formulated by using an analogy between the hydrodynamic equations for steady flow and the voltage-current relationships in electrical circuits. In Figure 7–19, it is seen that water flow into a node, or tank, in the groundwater system leads to an increase in head. Analogously, the flow of current into a capacitor leads to an increase in voltage across a capacitor. Indeed, the two equations for these physical phenomena have exactly the same form. Consequently, to construct a physical representation of the groundwater system requires that a capacitor be chosen of the right size to represent storage at each node of the model. It might be noted that capacitors are devices which are each about the size of a U.S. twenty-five cent coin. Continuing with the development of the analogy, Figure 7–20 shows that the flow of water from one point to another in the groundwater system is driven by a difference in head. Analogously, the current through a resistor is driven by the difference in voltage between the two ends of the resistor. Consequently, for each channel which connects nodes together, one resistor which is a device about the size of a small piece of chalk is introduced. Overall, the model may be constructed as a set of capacitors which represent specific storage within each node and a set of resistors which represent hydraulic conductivity associated with a flow between the nodes, as shown in Figure 7–21.\textsuperscript{12}

The model includes a description of recharge and discharge at each node in the system. Recharge to the aquifers is represented by current generators which pump current directly into the capacitor which represents a node, much as is shown in Figure 7–19. Similarly, discharge or seepage to streams and bay waters, pumping and other sources of loss from a node, are presented by a current drain away from the node.

![Figure 7–19](image1.png) **Figure 7–19**  
**Analogy Between Water Flow and Electrical Current in USGS Model.**

![Figure 7–20](image2.png) **Figure 7–20**  
**Analogy Between Water Flow and Electrical Current in USGS Model.**

![Figure 7–21](image3.png) **Figure 7–21**  
**Analogy Between Water Flow and Electrical Current in USGS Model.**
The USGS analog model represents the upper glacial, Jameco and Magathy aquifers in a five layer structure. As seen in Figure 7–22, the grid system specifying the nodes extends into the New York City portions of the Long Island groundwater system. Groundwater conditions in the North Fork and South Fork are not included in the model. The horizontal node spacing is 1829 m, while the vertical depth of nodes ranges from zero to about 120 m. The five layer structure is required to accurately represent vertical change in hydraulic conductivity $K$ and specific storage $S$ which are the governing parameters in the hydrodynamic equations. More importantly, these two parameters vary widely in the horizontal plane. Therefore, it is necessary to specify $K$ and $S$ for each grid element in Figure 7–22 and to calculate a value for the resistor $R$ and capacitor $C$ which represent these parameters in the analog model. In practice, the specific storage is poorly known but appears relatively constant over reasonable areas, and this simplifies determination of capacitors. Conversely, hydraulic conductivity varies widely.

There are some 2000 horizontal grid elements in Figure 7–22 which are represented in the model, and there are five such layers. Consequently, there are about 10,000 nodes or capacitors. At each node, there may be a connection to as many as six other nodes using resistors. In addition, recharge, pumping wells, discharge to streams, and the like are all entered at specific nodes primarily in the upper layers, from which we see that the construction of a physical analog model can be quite laborious.

Given the types of data available on specific storage and hydraulic conductivity throughout the aquifers, the USGS model makes several plausible assumptions. Specific storage is taken as constant within a particular model layer; capacitors are uniform. The primary variation in hydraulic conductivity is its anisotropy. They assume a roughly 10:1 ratio between horizontal and vertical conductivity. Therefore, at each point in the system an average horizontal conductivity is established from field data and the vertical values are then scaled accordingly.

FIGURE 7–22 Grid Layout For USGS Model.
The effects of recharge and discharge are thought of as "stresses" upon the groundwater system. The model is assumed calibrated and verified if it replicates past variations in head when the historical pattern of stresses is used to drive the model. Where discrepancies arise between predicted and observed head, the vertical hydraulic conductivity is a primary calibration parameter because it is the least certain of the data inputs (remember this requires physical change in the values of resistors). Following such calibration, the model is assumed valid for future prediction of response to new stresses upon the system.

### 7.3.3 Finite Element Model

The Princeton University group (Pinder for hydrology models, Cleary for the groundwater quality model) has prepared groundwater models which have been utilized in hydrology studies on the South Fork and main portions of the Island and in examination of water quality surrounding the Babylon landfill site. It should be noted immediately that the hydrodynamic equations for the USGS analog model and the Pinder computer-based models are the same; only the solution technique differs. However, within the analog model it is virtually impossible to introduce pollutant dispersion, whereas the computer formulation lends itself to an extension into groundwater quality modeling.

The use of the hydrodynamic models, and pollutant dispersion models within the overall framework of the 208 project will be quite different. For area-wide studies of the impact of pumping and recharge upon water table, the USGS analog and Pinder computer-based models provide comparable information. On the other hand, it was noted earlier that groundwater flows are quite slow and it requires decades for water to move over distances of a mile or so. As a result, the dispersion of pollutants within the groundwater system is quite localized. The Cleary groundwater quality model best serves to define the details of contamination around localized sources, such as the Babylon landfill.

The Princeton University group has prepared four specific models for analysis of groundwater conditions on Long Island, as noted in Table 7–1. The two South Fork models are of course specific to this particular area of the Island where water availability and quality have become major concerns. The regional South Fork model describes head, or water table, throughout the area and includes explicit representation of the saltwater-freshwater interface as it shifts due to withdrawals for water supply and irrigation needs. The local well South Fork model specifically considers saltwater intrusion into a major pumping well where the regional model results indicate that such problems might arise. The Long Island groundwater model is identical to the USGS analog model (with the exception of mathematical solution technique as noted earlier). Both models essentially describe change in head or water table for various stresses such as pumping, recharge, changing stream flow, and the like upon the groundwater system. Because these Long Island models

<table>
<thead>
<tr>
<th>Title</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>South Fork-Regional (Pinder)</td>
<td>Spatial distribution of height of water table. Movement of saltwater-freshwater interface.</td>
</tr>
<tr>
<td>South Fork-Local Well (Pinder)</td>
<td>Saltwater intrusion into a pumping well.</td>
</tr>
<tr>
<td>Long Island-Regional (Pinder)</td>
<td>Spatial distribution of height of water table (equivalent to USGS but different mathematical formulation).</td>
</tr>
<tr>
<td>Groundwater Quality (Cleary)</td>
<td>Contaminant plume (pollutant concentration from a landfill or other source).</td>
</tr>
</tbody>
</table>

are designed for large-scale analyses, they do not include any detailed characterization of saltwater/freshwater interface for the main island and, as a result, are not appropriate for saltwater intrusion studies. Finally, the Cleary groundwater model is a very localized hydrodynamic model somewhat similar to the South Fork local well model, augmented by pollutant dispersion which will be described briefly below. Cleary takes an approach which involves direct solution of the coupled hydrodynamic-pollutant concentration equations so that certain non-linear interactions between groundwater flow velocities and contaminant diffusion are fully represented in the model and permits study of a very wide range of contamination problems.

In the description of pollutant dispersion, there is one important distinction between the surface water system and groundwater. Nonconservation pollutants are those which tend to decay or change over time. This includes, for example, fecal coliforms, biological oxygen demanding substances (BOD), and the like. Such contaminants have rates of decay which are typically hours or days. In consideration of tidal flows, the movement of water takes place over periods of hours as does pollutant decay, and it is important to track both the movement and the decay of such pollutants. However, in the groundwater system, the residence time before there is any substantial movement of water is decades. Consequently, nonconservative pollutants introduced into the groundwater system undergo effective decay at the point of entry, and they will not be found at any substantial distance from the source. As a result, the pollutant dispersion mechanism need be considered only for conservative pollutants. With this distinction, the description of pollutant dispersion is otherwise quite similar to that in surface waters.
The general character of pollutant dispersion in the groundwater system is shown in Figure 7–23. The change in concentration of a contaminant introduced into the groundwater at, say, a recharge well, occurs because of two general forces. First, advection describes the overall flow of water from one point to another in the aquifer. If the motion were entirely uniform, a "slug" of contaminant would remain intact. In reality, we have noted earlier that flow is anisotropic. The small downward components of flow lead to some vertical spreading of the pollutant and, hence, reduction in level of concentration. Superimposed upon this advection is diffusion, the tendency of any concentrated pollutant to spread out and fill the entire volume in which it is situated, as illustrated in Figure 7–23. In the absence of any groundwater flow (no advection) diffusion itself will lead to lower concentration of pollutant. For example, a drop of dye placed in a bathtub gradually diffuses to fill the entire tub with a low uniform level of dye, as pointed out earlier. As noted in Figure 7–23, the superposition of advection and diffusion leads to development of a containment plume in the aquifer. Such a plume extends several miles south of the Babylon landfill site on Long Island. Remember that groundwater flow is quite slow and that it takes decades for such plumes to form.

These may be adjusted during calibration to achieve an adequate match between head predicted by the model and actual field data, but other factors such as stream flows might also be factors. Thus, one cannot always be certain that the right parameters have been set during calibration. Once again this largely depends on the experience of the modeler and in the Long Island 208 this is not a real concern.

The USGS analog model and Princeton Long Island models serve the same purpose. They each compute the height of the water table or head at roughly 1 mile spatial increments everywhere between the New York City line and Riverhead in the Bi-County region. In addition to specific storage and hydraulic conductivity parameters, the models require groundwater seepage to streams, natural recharge and other stresses including pumping well locations and volumes, injection wells for recharge and the like. The information needs for such regional models, involving up to 10,000 nodes, are extensive. Fortunately, such a set of data has been prepared for the Long Island region. The models may be run in a dynamic mode using time steps of a year since decades are required for large differences in head to appear. The analog model requires considerable effort to establish electrical circuit arrangements for stresses and/or physical parameters and this places some limits on its ability to compute a wide range of alternatives for wastewater management. The Princeton model provides a computational advantage over the analog model simply because it is computer based. In the Princeton version it is easier to change the values used in the model and even the geometry of the grid system itself. The model is therefore more flexible in that the time required for modifications is short. This means that it is more readily transferable from one place to another since it does not entail the re-wiring of a physical analog.

The South Fork model is a parallel to the Long Island hydrology models. Because of the geology of the South Fork and the potential of saltwater intrusion problems, the model is structured as a two-layer aquifer system (rather than five for the Island) and includes a detailed description of the saltwater-freshwater interface. It is utilized to assess overall impacts of pumping in much the same way as the models above and is subject to the same limitations. Again, the model provides a dynamic history and/or future of head changes and how the saltwater/freshwater interface shifts in response to pumping stresses.

The Local Well model is essentially a small piece of the regional model in which a fine grid configuration may be used to follow saltwater intrusion into a well. In this case, it relies upon an accurate knowledge of the physical constants of the aquifers as well as some detailed history of saltwater and freshwater movement in the local area of the well considered. Such measurements are often difficult to assemble. Consequently, these highly localized models are often calibrated and verified for the specific well under study, for which an extensive data base may exist, but the model and its results may not be easily transferable to other situations.

The Cleary groundwater quality model is another highly localized
model which is an adaptation of a small section of the hydrology model with additional relations to describe dispersion of pollutants within an aquifer. As such it is a water quality model. In addition to data limitations associated with the groundwater hydrology, the model requires sufficient local data to establish a pollutant diffusion coefficient in order to track the spread of the contaminants.

Overall, the groundwater models fall into two categories. The hydrology models are designed to supply information of the spatial distribution of the water table and its change in the future. The localized well and water quality models are useful in the specific sites for which they are prepared and are not directly applicable on a more extensive basis.

7.3.5 Water Budget Model

The water budget model prepared by the Cooperative Extension Service (CES) for the 208 project has two primary purposes: first, to estimate recharge of precipitation and/or irrigation to the groundwater and, second, to provide a transport model for nitrogen loadings to both ground and surface water, in the Long Island area. In this section, the transport mechanisms contributing to the overall water cycle and the extension of this transport model to consider nitrogen loads is described.

We wish to draw attention immediately to the fact that this work does not constitute a groundwater model in the sense of the USGS and Princeton efforts. Recharge here is to the upper aquifers where the soil is relatively unsaturated and does not deal with movements of water in the deeper aquifer layers. The basic premise of the water budget model is that most of the predominant processes involving uptake of water and/or nitrogen occur within the root zone for vegetation on the Island. One consequence of this observation is the fact that whereas lateral movement of groundwater is more significant than vertical flow in the models of sections 7–1 and 7–2, in the present discussion it is the vertical infiltration of the unsaturated subsurface medium which is important. Lateral flows are, as we will see, considered insignificant.

A schematic of water flows is shown in Figure 7–24. The root zone itself is some one to three feet deep depending upon vegetation type. The whole of the Long Island Bi-County region is divided into such cells with surface areas of 2.25 miles, coincident with the planning maps for demographic and land use development. Twelve types of land use development may be introduced on the surface of any cell. The model states simply that recharge is the net of precipitation and/or irrigation inflow over runoff, evapotranspiration uptake by plants, and other outflows from the cell.

In each cell a mass balance relation is employed which asserts that total water into a cell must equal total amount leaving it. This assertion is conceptually no different from the mass balance relations utilized in the water quality models discussed earlier. For a given cell one has the following relation which defines 'water budget':

\[
\text{Recharge} = \text{Precipitation} + \text{Irrigation} - \text{Runoff} - \text{Evapotranspiration} \tag{7.4}
\]

One should also add to this equation a term for net horizontal flow of water into or out of neighboring cells, but this is considered negligible in relation to vertical flow since the soils are generally considered to be loose, sandy and unsaturated. The term evapotranspiration in relation (7.4) refers to evaporation plus transpiration of water through the root zone of plants.

The generation of recharge to the groundwater system may be viewed as a two step process. First, precipitation and/or irrigation cause the entry of water into the system, as illustrated in Figure 7–25. Secondly, vegetation takes up some water for respiration. These evapotranspiration losses are also shown in Figure 7–25.

At the surface, water either infiltrates the ground or creates runoff. The split between these two is characterized by a runoff coefficient determined for various types of soils and surfaces. Impervious surfaces associated with land use development are assumed to block all infiltration. There is an effective "active storage capacity," which describes its overall ability of the soil to capture and retain water. This storage capacity is the product of depth of the root zone and an available moisture capacity.

In general, the moisture content of the soil varies from a "field capacity" in which all available pore space in the soil has been occupied by water down to a minimum in which the little remaining water is locked into the soil structure (see Figure 3.12). When soil moisture drops, evapotranspiration by plants becomes limited by available moisture. At the minimum level, called the "wilting point," the little remaining water is unavailable to plants. Conversely, near the field capacity, soils are near saturation and excess water is generally present over and above that required to support plant respiration at its maximum. As the moisture content drops below the satu-
rated condition, plants still have a plentiful supply. However, at a level denoted as critical moisture, plant respiration requires larger amounts of water than available from the soil. Moisture content of the soils becomes the limiting factor in plant respiration. The wilting point is that where no free moisture remains and plant life ceases.

Within the model it is necessary to characterize four major parameters noted above:

precipitation/irrigation—these are the primary water inputs to the system.

runoff coefficient/impervious area—the coefficient describes runoff in areas of vegetation, whereas impervious area gives 100% runoff. However, because of the wide distribution of recharge basins, both natural and man-made, runoff is assumed to occur only in those grid cells adjacent to a water body, e.g. only within about 2 miles of shorelines or streams.

active soil storage (root zone depth multiplied by the available moisture capacity)—this parameter describes the ability of the root zone area to store water.

critical moisture—this parameter characterizes the limiting moisture content for evapotranspiration.

To establish values for these parameters, the CES group relied upon an extensive review of existing data and literature supplemented by field testing, particularly in the area of evapotranspiration effects. The model is calibrated for iteration over steps of one month to obtain annual water cycles. Since precipitation rates, and hence soil moisture content, vary widely within any month, it is important to test the accuracy with which one month averages describe actual recharge. In general, errors in using monthly rather than daily water balances are less than five to ten percent. In addition, following calibration of the parameters above, estimates of recharge and runoff from the model are in good agreement with past direct measurements in selected areas on the Long Island region.

The model is oriented toward the immediate impact of land use development upon recharge to the groundwater system. In its present form it provides a picture of the way in which continuing and alternative land use development limit eventual recharge.

The model above may be supplemented by the addition of nitrogen loadings. Primarily, nitrogen loads reflect the use of fertilizers in residential and commercial areas for lawn maintenance as well as in agricultural areas. It becomes necessary in this case to augment the evapotranspiration relationships with a description of nitrogen uptake in the plant life. With this addition, the model tracks the infiltration and/or runoff of nitrogen, its uptake in the root zone, and the balance which flows on through the system as recharge.
7.4 INTEGRATED USE OF MODELS

The Long Island area is one of wide variety in terms of marine and hydrogeologic water systems. The study area shown in Figure 7-27 has some 1000 miles of shoreline, wetland areas, barrier beaches, shallow bays, and a number of other important elements of the coastal environment. The groundwater system is equally complex, consisting of shallow and deep aquifers, intervening clay layers, underflow to bays, seepage to lakes and streams, and is subject to the problem of salt water intrusion near shorelines. Surface features which contribute recharge to groundwater and runoff to estuaries range from built up urban environments to natural meadowlands. Within this kind of framework the use of models plays an important role in addressing the difficult questions of regional strategy for wastewater management, particularly in our ability to quantify the impacts of different land use schemes and wastewater alternatives. Data gathering and assessment for model development, as well as the models themselves, also prove valuable in clarifying the present condition of surface and groundwaters in the area and thus serve to provide a backdrop against which the future may be judged.
The way in which results from the different modeling efforts are used in the evaluation of various alternatives for the wastewater management is a complex process. Moreover, it is important to distinguish the use of the models in their role as evaluators of alternatives from that of guiding the selection of alternatives which are appropriate to specific areas. As observed above, much of the data used in model development and calibration serves as a backdrop for the selection of alternatives. In addition, some of the estimates of future wastewater loads may be used in conjunction with the models to define the general character of the region's wastewater future. The model computations, in this case, serve as a backdrop of future conditions, within which to choose viable management strategies. While it is appropriate to discuss such overall regional strategies, specific alternatives will most often concern selected study areas. Consequently, we provide here a short discussion of the models and the ways in which information from these is integrated. It should be pointed out that this integration generally takes place in the context of meetings of groups of individuals, rather than through any type of formal or computer link between the models themselves. The illustrative case study is the Riverhead/Peconic Bay (Figure 7-28). Similar considerations of course apply elsewhere.

FIGURE 7-28 Peconic River, Peconic Estuary, Flanders Bay with Waste-Water Management Structural Alternatives.
The engineering consultants have recommended a number of alternatives which include the use of regional, sub-regional, and local sewage treatment plants or diversions to the Riverhead sewage treatment plant, which may require expansion of that plant. There have also been expressed concerns with Peconic River stream flow conditions so that it might be appropriate to consider stream augmentation and/or recharge options which assist in maintaining stream flows, the water table, or water quality, respectively. To test the various alternatives, including location of outfall pipes, stream augmentation, recharge, and the like, requires the use of different models for different water bodies. The impacts of discharge to rivers can have secondary effects upon the bay systems, for example through increased effluent loading to the bays. Likewise, in the Riverhead/Pecanic area, recharge options may reduce nutrient and organic loading to streams which could have adverse effects upon flora, fauna, and stream water quality. In the face of these complex interactions, it often becomes necessary to combine the information from the several models to properly evaluate these types of wastewater management alternatives.

Figure 7-28 illustrates some of the possible structural options in the Riverhead/Pecanic region. The water balance model (Cornell Extension Service) is utilized to establish an accounting of precipitation, evapotranspiration, recharge, and runoff components in the total water cycle as shown in Figure 7-29 for the Riverhead area. Of course, paved surface, recharge basins, and other characteristics of the future land use lead to changes in runoff, while removal of vegetation decreases evapotranspiration water loss. The model is so constructed that it can describe the impact of land use development upon runoff, which is a component which enters into estuary and stream models, and recharge, the flow of water which enters the groundwater system. Since it has been estimated that runoff nutrient levels can be a substantial part of the total load to surface waters in areas such as Riverhead, the water balance model can be used to estimate runoff itself as a contaminant to bays, and for recharge to the water table. It should be noted, however, that this model describes only aggregate water flow over large areas and is able to only include conservative contaminants as part of the water cycle, primarily nitrogen and phosphorus. Some of the information which the model does not address is made available through estimates provided by the engineering consultants.

With regard to sewage treatment plants, some method of disposal of effluent is required either to surface waters or groundwater. In the event of discharge to estuarine waters, the Tetra Tech models prove useful in the assessment of the impact upon water quality in such bays. Outfall discharges may add to the water flow and contaminant load. For example, if one considers a specific outfall site in the bay, the Tetra Tech model provides profiles of constituent loading at various locations (as shown by the nodes along the bay channel in Figure 7-30, such as for total nitrogen N and phosphorus P). This is shown in Figure 7-31. The curves correspond to total concentrations

![Figure 7-29 Example of CES Computations in Peconic Bay: Precipitation–Evapotranspiration–Runoff–Recharge.](image)

in milligrams per liter for a typical model run. An alternative strategy might be to consider the diversion of wastewater flows to existing or new plants as suggested in Figure 7-28. The options for wastewater discharge from such plants may include stream augmentation. In this case, the Tetra Tech river models are utilized to describe water quality along the length of a river as well as the impact of the discharge upon the water flow itself.
Discharge to the groundwater system has two effects. First, there are local increases in the water table elevation and in addition, the local groundwater quality may be either improved or degraded depending upon prior water quality. Where only a description of impact upon water table and water flow from one point to another in the aquifers is necessary, the USGS and Princeton University models may be applied. For example the Pinder model for North Haven would typify groundwater "hills" such as those shown in Figure 7–32 which would surround an injection well at the point of recharge. In the evaluation of alternatives involving recharge, it might also prove useful to utilize the Cleary groundwater quality model for assessing the flow of contaminants away from the injection well though data sufficient to effectively utilize the model is not readily available for all areas of the Island.
The flow of data and information for the evaluation of wastewater management alternatives is exhibited in a different form in Figure 7–33. Here we indicate the possible role of land use development as a basic driving force for the region.

![Diagram](image)

**FIGURE 7–33** Evaluation of Management Alternatives.

It should be noted that the solid lines indicated in Figure 7–33 denote flow of information within the 20B project itself and not formal links between the various computer models. Ideally, one would like to tie together the models into an overall comprehensive assessment. However, there would be a number of difficulties in carrying this out. The models were all developed by separate organizations using different programmers, programs, computer languages and computer systems. Consequently, it would be a formidable task to pull these together into a comprehensive whole. In a practical sense, however, it is not really necessary to pull all the models into one package. In the assessment of various alternatives, such as a sewage
treatment plant option, its impact upon the water system in the area is assessed using only the specific models which apply. For example, if only a bay outfall is to be considered as the discharge option, the Tetra Tech estuary model alone would be used to assess this. There are, in addition, a wide range of qualitative and quantitative considerations which are drawn into the evaluation of wastewater management alternatives at this point, but which lie outside the scope or abilities of the models. Outfalls to open surface waters could have impacts on stream flows. The groundwater models are not constructed to deal effectively with such effects in individual streams. Moreover, the river model itself does not accurately describe the impact of outfalls on upstream water levels. Environmental impacts upon shorelines, drainage structures, flooding, and ecology of bay and river systems also fall outside the range of the models. However, the models do provide the quantitative background of water quantity and quality required to address these questions. Where impacts of alternatives are considered undesirable, one may cycle back by reconsidering the option, introduce some changes in the character of the alternative, and then run the new option back through the models to assess whether its impacts might now be considered more favorable.

After having surveyed in this report the several modeling components of the 208 it is useful to give a brief summary of what each model does in terms of output, where that output is directed in terms of Long Island waste management, and the inputs required to make the model operational. This is shown in a series of flow diagrams, given in Figures 7–34, 7–35 and 7–36. This introductory view of the 208 models may hopefully encourage the serious reader to consult some of the more technical documents provided by the consultants.

**FIGURE 7–34 Water Budget Modeling.**

**FIGURE 7–35 Groundwater Modeling.**

**FIGURE 7–36 Surface Water Modeling.**
BIBLIOGRAPHY


1 Let $\Delta H$ denote the incremental change of $H$ during a time interval of duration $\Delta T$ seconds. Then the volumetric change during $\Delta T$ is given by $\Delta H A_s$ and the principle of mass conservation then says that

$$\Delta H A_s = q \Delta T$$  \hspace{1cm} (1)

where $q \Delta T$ is the net flow during $\Delta T$. Using (1) equation (2) is often written as

$$\frac{\Delta H}{\Delta T} = \frac{A_s u}{A_s}$$  \hspace{1cm} (2)

As we will see later a simple variant of (2) also holds for groundwater flows.

2 Let $\rho$ denote the water density (kg per cubic meter) so that $\rho A_c L$ is the total mass in kilograms of the water in the tube. The momentum of the moving fluid is the product of mass and velocity and is given by $\rho A_c L u$. Newton's second law of motion says that the rate of change of momentum over time must equal the sum of the forces which act on the moving body. In the absence of any external forces on the fluid, the rate of change of momentum is zero, which is the principle of conservation of momentum. One such force, as we already noted, must be due to the pressure difference exerted at the two ends of the channel. If we add to this the resistive force due to friction caused by the roughness of the channel then, since $u$ is the only quantity which can vary in the expression for momentum, the rate of change may be stated as

$$\rho A_c \frac{\Delta u}{\Delta T} = \text{Pressure Force} + \text{Resistive Force}$$  \hspace{1cm} (3)

in which $\Delta u$ is the incremental change in velocity during an interval of duration $\Delta T$.

It can be shown that the pressure force is given by

$$p \rho A_c (H_1 - H_2)$$

where $g$ is a number known as the gravitational constant (9.8 meters per second squared). Note that if $H_1$ is greater than $H_2$ then the force is positive and it acts so as to give a flow from tank 1 toward tank 2. When $H_2$ is greater than $H_1$ the opposite is true and the flow is reversed. When $H_1 = H_2$ the the net force is zero and there is no exchange of water between tanks.

Resistive force is found from an empirically derived relationship which states that resistance is proportional to the square of the velocity. That is

$$\text{Resistive Force} = - \rho A_c R L u |u|$$

where $R$ is a constant which measures the roughness of the channel walls. As the ratio of the area $A_c$ to the perimeter of the cross section of the channel increases the resistance $R$ decreases. The particular form of the above expression was chosen so that the force will always operate in a direction opposite to the direction of flow (if we had written $u^2$ instead of $u |u|$ this would no longer be true.)

By combining the above remarks together into relation (3) and by eliminating $\rho A_c$ from both sides of the equation one obtains

$$\frac{\Delta u}{\Delta T} = g \frac{(H_1 - H_2)}{L} - R u |u|$$  \hspace{1cm} (4)

3 Hydraulic conductivity is defined by a number $K$ which equals $\mu q g / k$. Once again $g$ is the gravitational constant. As $K$ increases, resistance decreases.

In view of the above discussion the basic relations 2 and 4 need to be modified slightly. Consider 2 first. Since $\epsilon$ is now the fraction of volume which can store water, then $\Delta \epsilon A_c H$ is the effective change in volume of the water in the tank due to a change $H$ in head. Otherwise the same conservation of mass principle still applies as before and so equation 2 becomes

$$\frac{\Delta H}{\Delta T} = \frac{A_s u}{A_s}$$  \hspace{1cm} (5)

When $\epsilon = 1$ (as in open water) this relation of course reduces to the previous one. Note also that when $\epsilon$ is smaller than 1 then, since there is now less volume in which to store water, the change $\Delta H$ is larger than for open waters. Relation 4 for open water is similarly modified in the case of a porous medium. Once again $A_c$ is replaced by $\epsilon A_c$ as the effective cross-sectional area which is acted on by the pressure force. Also the resistive force is now due to the viscosity of the water in the permeable medium as we discussed above and is empirically found to be proportional to the velocity $u$. Specifically one can show that

$$\text{Resistive Force} = - \rho \epsilon A_c L u$$

This allows us to write an analog of relation (4) as

$$\frac{\Delta u}{\Delta T} = g \frac{(H_1 - H_2)}{L} - \frac{\rho \epsilon u}{k}$$  \hspace{1cm} (6)

Note that although in open water resistance to flow is proportional to the square of the velocity, in the porous medium it is simply proportional to the velocity itself.

4 The analogy can be carried further. Heat flows from a point of high temperature to a lower, also at a rate proportional to the temperature difference. Finally, in an analogy that will be referred to again in the section on groundwater models, current flows in a circuit at a rate proportional to voltage difference between two points. The difference in concentrations relative to the distance between two points of water levels, osmotic, or voltage, describe what are called "gradients." When these gradients or differences are zero there is no further exchange between the two points in the body and the system is said to be in equilibrium. One more point: all the flows described here are also subject to the level of conductivity (or permeability, or what have you) in the channel which connects the points. The smaller the "conductivity," the greater is the resistance to flow.
Referring to equation 2 this can be stated mathematically for each node $i$,

\[
\frac{\Delta H}{\Delta t} = \sum_{j=1}^{m} A_{is} u_j + Q_{in} - Q_{out}
\]

(7)

where all flow conditions on the right side of (7) are determined at some previous time and $\Delta H$ is the net incremental change in $H$ during a time step $\Delta t$ which takes us to the present moment. The symbol $\sum$ is of course the usual shorthand for taking a sum of terms from $j = 1$ through $j = m$.

Equation 4 must now be interpreted in terms of a head difference between two specified nodes labeled as $j_1$ and $j_2$ and $u$ then flows between these two nodes.

Consider BOD, for example. Suppose $k_B$ is a constant which denotes the rate at which BOD decays over time. If $C$ is the quantity of BOD in a given volume $V$, at some earlier instant, then $k_B C \Delta t$ is the amount of BOD lost to decay during the next time interval of duration $\Delta t$. If we subtract this from the previous value of $C$ we obtain the current value of BOD. That is

\[ V \Delta C = -k_B C \Delta t \]

The reason for subtracting is that the new value of BOD will always be less than its previous value. Of course this argument assumes that all other factors which may affect BOD are absent during the time interval. That is, no sources or sinks and no advection or diffusion. The relation for BOD is simply a special case of equation 7.3. In general the full relation for BOD will include all these other terms in addition to the one given here for decay.

Re-aeration refers to oxygen gains which occur at the surface of the water due to contact with air, while photosynthesis is the process whereby microorganisms in the water release oxygen as a result of the manufacture of carbohydrates from water and carbon dioxide. Finally respiration is the opposite of photosynthesis and refers to the biological activity in which organic carbon is oxidized. As a result oxygen is consumed.

This leads to the equation

\[
\varepsilon A_s \frac{\Delta H}{\Delta t} = u A_C + Q_{in} - Q_{out}
\]

(8)

where $u$ is velocity in the channel of the area $A_C$. This extends the equation to include source and sink terms and is analogous to equation (7) for surface models.

Consequently, using the expression (8) above we see that

\[
\frac{\Delta H}{\Delta t} = \frac{u + Q_{in}}{L} - \frac{Q_{out}}{L} - \frac{C_{A_s L}}{L}
\]

where the left side is approximately the percentage change in head which is driven by the velocity and the various recharge, pumping and seepage flows for the node. Note that since $C_{A_s L}$ is the node volume for water storage, $Q_{in} / A_s L$ is a percentage volumetric flow. The overall change in head is a result of all incoming flows from other nodes within the layer considered, or to the layers above and below, and contributions from recharge or discharge, at the node.

The expression for change in velocity (or momentum) with time requires no modification. As shown in Figure 7–15, the velocity is driven by the difference in head, or water elevation between two adjacent nodes. However, the soil acts to restrict flow of groundwater and the rate of change of velocity is

\[
\frac{\Delta u}{\Delta t} = \frac{g (H_2 - H_1)}{L} - \frac{1}{K} u
\]

(9)

for the two coupled nodes. If we generalize to a multi-layer aquifer system, three separate equations of type (9) must be written for velocities $u_x, u_y$, and $u_z$ along $x$, $y$, and $z$ axis, respectively. However, this will not be critical to our discussion of the model below.

Given by $u_0 = K (H_2 - H_1) / L$

The choice of resistor and capacitor values is important. For convenience, capacitors should be small enough that the currents required to drive the physical model are appropriate for small operational amplifiers and small signal equipment. Similar conditions hold for resistors in order that impedance match to external sources not be a problem. Also, the time required to resolve pressure differences in the real groundwater system is something like thirty years, as we know, and in the model this is characterized by the time constant for resistor/capacitor circuits. However, we prefer this not be the thirty year equilibrium for the aquifer, but some convenient laboratory time. The USGS model is formulated so that two years real time is about 1/1900 sec. in model time.

The Princeton models are finite element models as opposed to finite differences represented in the USGS model. Finite difference refers to sectioning the aquifer layer into nodes (points) and channels (lines) so that one has a set of difference equations involving head at each node. Finite element refers, roughly, to sectioning the aquifer layer into finite-sized cells (of any shape). The head is described by a mathematical function having spatial dependence and the particular value for water elevation at each finite element is found simply by numerical evaluation of the function.

The above numbers, although generally true, overlook the fact that the distribution of pollutants observed in groundwater systems is due to several mechanisms, of which physical dispersion is only one. A pollutant which may behave conservatively (not decay with time or react with the permeable media) in one aquifer may behave non-conservatively in another. For example a pollutant which absorbs strongly in silt and clays would be attenuated in shorter distances than in a system which is mainly sand and gravel. Moreover, the ability of groundwater systems to assimilate certain pollutants may be limited upon the rate at which pollutants are introduced (e.g., simple wastewater soil treatment systems clog with excessively high organic loads.)

Those shown in the figure are somewhat hypothetical and do not conform with the options being prepared by the engineering consultants.
**GLOSSARY**

Because this report is largely intended for the general reader certain unfamiliar technical terms have been defined in the text at the place at which they first occur. These are listed below together with the page location where the definition can be found. Certain more commonly used expressions are not included.

<table>
<thead>
<tr>
<th>Term</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Advection</td>
<td>246, 249</td>
</tr>
<tr>
<td>Anisotropy</td>
<td>256</td>
</tr>
<tr>
<td>Analog Model</td>
<td>259</td>
</tr>
<tr>
<td>Benthic</td>
<td>250</td>
</tr>
<tr>
<td>BOD</td>
<td>250</td>
</tr>
<tr>
<td>Calibration</td>
<td>249</td>
</tr>
<tr>
<td>Conservative Substance</td>
<td>250</td>
</tr>
<tr>
<td>Conservation of Mass</td>
<td>244</td>
</tr>
<tr>
<td>Darcy Velocity</td>
<td>258</td>
</tr>
<tr>
<td>DO (dissolved oxygen)</td>
<td>250</td>
</tr>
<tr>
<td>Diffusion</td>
<td>246, 249</td>
</tr>
<tr>
<td>Dynamic Model</td>
<td>251</td>
</tr>
<tr>
<td>Ecological Model</td>
<td>255</td>
</tr>
<tr>
<td>Evapotranspiration</td>
<td>263</td>
</tr>
<tr>
<td>Eutrophication</td>
<td>255</td>
</tr>
<tr>
<td>Finite Element</td>
<td>261</td>
</tr>
<tr>
<td>Hydrodynamics</td>
<td>246</td>
</tr>
<tr>
<td>Hydraulic Conductivity</td>
<td>245</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Term</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass Balance</td>
<td>250</td>
</tr>
<tr>
<td>Non-conservative Substance</td>
<td>250</td>
</tr>
<tr>
<td>Permeability</td>
<td>245</td>
</tr>
<tr>
<td>Phytoplankton</td>
<td>255</td>
</tr>
<tr>
<td>Photosynthesis</td>
<td>250</td>
</tr>
<tr>
<td>Porosity</td>
<td>245</td>
</tr>
<tr>
<td>Plume</td>
<td>262</td>
</tr>
<tr>
<td>Quasi-Dynamic Model</td>
<td>251</td>
</tr>
<tr>
<td>Re-aeration</td>
<td>250</td>
</tr>
<tr>
<td>Respiration</td>
<td>250</td>
</tr>
<tr>
<td>Sink</td>
<td>248</td>
</tr>
<tr>
<td>Specific Storage</td>
<td>258</td>
</tr>
<tr>
<td>Steady State Model</td>
<td>252</td>
</tr>
<tr>
<td>Stage-Flow Relationships</td>
<td>254</td>
</tr>
<tr>
<td>Verification</td>
<td>249</td>
</tr>
<tr>
<td>Viscosity</td>
<td>245</td>
</tr>
<tr>
<td>Water Budget</td>
<td>246</td>
</tr>
<tr>
<td>Zooplankton</td>
<td>255</td>
</tr>
</tbody>
</table>
Section 8 Management Alternatives

8.0 INTRODUCTION
Performing the 208 areawide study for Long Island involved a variety of investigations that were necessary for the development of sufficient data upon which future planning will be based. These fact finding studies were undertaken by a number of government personnel and private consultants specializing in various aspects of environmental management.

One of the key objectives of the study is the development of a planning process that will lead to the implementation of controls and abatements from nonpoint and point sources of pollution. This interim report covers in general terms the structural and non-structural options that can be employed on Long Island for these purposes. The principal sources of information reported are past studies and those included in the present 208 undertaking. An attempt has been made to incorporate only that material which is most useful to the development of the final plan. As such, the interim report provides an overview for planning purposes. This report is not meant to be inclusive, but rather to identify the major classes of techniques and technology for the interested reader. The actual selection of specific equipment and facilities is beyond the scale of the 208 program. No preferences are stated or implied. Normative judgements are made in the Plan Summary. This section is presented in two parts.

The first section addresses nonpoint pollution sources and predominately stresses non-structural solutions applicable under three major headings, i.e., stormwater runoff control, conservation and watershed management, and land use controls. Legal, administrative and design criteria are not mentioned. They are covered in the Plan Summary and in Section Six.

The second section addresses point source pollution control and here the emphasis is on structural solutions. Conventional treatment technology is generally covered, and includes primary, secondary and tertiary treatment. A limited discussion of advanced removal techniques for phosphorus, nitrogen and dissolved inorganics is included. The balance of the section mentions alternatives to conventional technology, ranging from sprinkler irrigation and meadow/marsh/pond systems to onsite facilities, such as waterless composting toilets, aerobic biological units and subsurface denitrification septic systems.

8.1 NONSTRUCTURAL APPROACHES
8.1.1 Introduction
Nonpoint source pollutants cause many of the most common and familiar pollution problems, including polluted well water; polluted and clogged rivers and streams; an overgrowth of weeds and algae, bacterial contamination of shellfish, plus sediment in lakes and ponds; and loss of wildlife. A description of these problems will demonstrate why attention must be given to a control program that limits discharges from nonpoint sources.

8.1.1.1 Pollution of Ground and Surface Waters. When the groundwater that supplies municipal and private wells becomes polluted, the water can no longer be used for drinking without costly treatment — treatment that may be impractical in the case of private wells. Groundwater becomes polluted when bacteria, chemicals or salts find their way into aquifers, the water-bearing underground rock formations.

A primary source of pollution of groundwater can be from cesspools
and septic tanks. They were often placed where the soil cannot do its proper filtering job because it is either too impervious (like clay) or too wet. Similarly, leaks that develop in sewer pipes — and they often do — will contribute to groundwater pollution.

Improperly located, designed and managed landfills, chemical or petroleum storage facilities and mining operations also may contribute to both chemical ground and surface water pollution. The concentration of pollutants works its way through soil and reaches the aquifer. And especially in coastal areas, freshwater wells may turn brackish as the water that is removed is replaced by the ocean's salt water.

Pollution of streams and the accumulation of silt or other sediment is a problem in some areas. When organic matter is washed into streams, there is an increased demand for oxygen (biochemical oxygen demand or BOD), which reduces the oxygen required for fish and other aquatic life, sometimes to a point where the water can no longer support life. It also may make the water unfit for swimming or other recreational uses. A stream that is filled with sediment can no longer support aquatic life, supply reliable amounts of water for other boating and other recreational opportunities.

While much stream and groundwater pollution comes from inadequate- ly treated point source waste discharges, a substantial amount comes from the following nonpoint sources: urban storm runoff, agricultural runoff, construction sites, hydrographic modification and solid waste sites.

Urban storm runoff carries with it debris; concentrations of chemicals and fertilizers that have accumulated on the earth's surface during dry weather; leaves, twigs and other organic matter; grease and spilled petroleum products from roads, parking lots and gas stations; animal droppings; and during winter thaws, salt used to melt ice on streets and sidewalks. The extent of pollution from urban runoff is partly due to the large areas of impervious surface which increase runoff and do not allow contaminants to be absorbed into the soil where they can be filtered.

Agricultural runoff from croplands and pastures carries sediment, fertilizer, pesticides, herbicides and animal waste into the water. Construction sites, where the land has been stripped of soil-holding vegetation, unless developed according to an approved erosion control plan will erode and contribute locally heavy sediment.

Hydrographic modification, i.e., changing the character of the stream itself by such measures as channelizing, may lead to increased flooding in unprotected downstream areas or modifying of fish and wildlife habitats.

Solid waste sites, including landfills and dumps, petroleum storage areas or mining operations, if poorly designed and operated, may contaminate both surface and underground water supplies with toxic chemicals and/or bacteria.

8.1.1.2 Pollution of Marine Waters. Weeds and algae have a number of unpleasant effects. Besides disrupting the ecological balance of the water body and reducing fish production, excessive weed and algae growth interferes with swimming and boating, and is generally unpleasant to look at.

Algae smells as it decays. Excessive sediment can significantly reduce the depth of lakes and streams, eventually filling them in. Adding too many organic nutrients to the water accelerates the eutrophication or aging process; the growth and decay of weeds and algae is speeded up, using up the oxygen supply and killing off fish and other water life.

The nonpoint sources that contribute to this problem are similar to those contributing to stream and groundwater pollution: urban and agricultural runoff which include fertilizer and animal wastes, semi-wild duck populations on inland ponds and individual cesspools and septic systems.

Added to this problem has been the loss of some of the Island's wetlands. Wetlands are like nature's treatment plants. They act as retention basins and filtering systems, removing and storing many of the nutrients in agricultural and urban runoff before they reach open waters. By interfering with the efficient natural system, we have had to create new, expensive and often less successful systems to replace what nature provided free. Fortunately, the remaining wetlands in Nassau and Suffolk Counties are now protected by the provisions of the Tidal and Freshwater Wetlands Acts of 1973 and 1975.

One of the major concerns that must be addressed satisfactorily in waste treatment planning is that of nonpoint source pollution. In drafting the 1972 Amendments to the Water Pollution Act, Congress recognized that structural controls over sources of nonpoint pollution are often undesirable and inappropriate. Structural controls are limited in coping with sediment, pesticide, micro-organisms and nutrient runoffs.

Proponents of land use planning and control strategies claim that the structural approach has been inadequate on four counts. First, control technologies have not been able, at the current level of funding, to keep pace with the growth in waste loads. Second, control technologies are not available for all sources. For instance, sediments, nitrates and phosphates resulting from land runoff are not practically susceptible to treatment at this time. Third, even the most effective technologies seldom achieve 100 percent removal, which may be necessary for sustaining environmentally sound growth in some urban/industrial centers. (At this time, even 90 percent removal effectiveness is still an outstanding achievement.) Fourth, the exponentially increasing costs associated with higher removal rates may impose economic burdens upon governments and private firms which are beyond their current financial capacity.

The most practical and economic approach to such pollution is preventive rather than remedial, i.e., preventing erosion and runoff by improvements in land use management and agricultural practices.

In general, nonpoint source pollution control relies on "best management practices" (BMP). For example, good agricultural conservation practices can reduce sediment yields from 50 to 90 percent. Preventive measures can

include limits, prohibition, substitution and changes in the method of use of
various contaminants to achieve control over the magnitude of specific
sources. Figure 8-1 depicts the relationships between source and type of
water pollution.

<table>
<thead>
<tr>
<th>TYPE</th>
<th>Domestic Waste</th>
<th>Industrial Waste</th>
<th>Agricultural Drainage</th>
<th>Construction Runoff</th>
<th>Urban Storm Water Runoff</th>
<th>Mine Drainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Organic Waste</td>
<td>●</td>
<td>●</td>
<td>●</td>
<td>○</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Nutrients</td>
<td>○</td>
<td>●</td>
<td>●</td>
<td>○</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sediment</td>
<td>●</td>
<td>●</td>
<td>○</td>
<td>○</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thermal Waste</td>
<td>○</td>
<td>●</td>
<td>○</td>
<td>○</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Toxicants</td>
<td>○</td>
<td>●</td>
<td>○</td>
<td>○</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other Types*</td>
<td>○</td>
<td>○</td>
<td>○</td>
<td>○</td>
<td>○</td>
<td>○</td>
</tr>
</tbody>
</table>

* Includes bacteria, oils, scums, etc.

Major or primary factor
Possible major factor — varies with conditions

FIGURE 8-1 Relations Between Source and Type of Water Pollution.

8.1.1.3 Land Use Practices. Most of these problems are caused by rela-
tively few practices, most of which have to do with the way we use the land.
The choice is to use land wisely by either limiting the production of pollu-
tants or to keep pollutants from entering the waters and affecting its quality.
Or land can be used poorly, and lead to serious and expensive consequences.

A list of poor land use practices, those that lead to lowered ground and
surface water quality, includes the following:

Using unnecessarily large areas of pavement which increases urban
runoff.

Cleaning streets infrequently, with the possible result that the “first
flush” after a storm in an urban area carries large quantities of chemi-
cals, debris and organic materials.

Overusing lawn fertilizer or road salt, or piling leaves in the gutters
of urban streets.

Overusing agricultural fertilizers, allowing animals direct access to
streams, and other agricultural practices.

Failing to take measures to reduce soil erosion during construction,
or during agricultural and forestry operations.

Failure to take measures to reduce or prevent sand mine leachates
and petroleum and chemical spills from reaching ground and surface
waters.

Locating housing with septic systems on unsuitable soils.

Filling and using wetlands for urban development.

Locating and designing waste disposal sites so that they leach (filter
down) into water sources.

Location of industrial facilities in prime recharge areas.

Dog curbing laws, sales of ducks as Easter pets, and encouraging
semi-wild duck populations on inland ponds by artificial feeding.

Efforts to clean up and prevent water pollution from nonpoint sources
are aimed at halting or remedying these kinds of practices. The solutions
involve regulation, monitoring, improved maintenance practices, voluntary
citizen compliance, structural improvement, improved construction practices
and improved land use planning implementation. By large, nonpoint
sources that can best be controlled by regulatory and administrative means
and by public education, e.g., industrial and animal wastes, landfills, agricul-
tural chemicals, etc., are not stressed in this report. These issues are covered
in Section Six, the Plan Summary and Section J of the Areawide Waste
Treatment Management Plan. The balance of the non-point portion of
this report discusses three broad approaches to nonpoint control — stormwater
runoff, water conservation and watershed management, and land use strate-
gies.

8.1.2 Stormwater Runoff

Every parcel of land is part of a larger watershed. Ideally, a stormwater
runoff management solution for any development project should be based on,
and support, a plan for its entire drainage basin. This is not a revolutionary
idea, but only recently have data collection and data handling technology
made this economically possible in a meaningful way. Even in the absence of
such basin-wide plans, new approaches to residential land planning, which
have been evolving since about 1995, have made it possible to apply more
creative approaches to stormwater management within a project. With their
application, the new effects of incremental urbanization can avoid most
negative impacts and may produce benefits, enhancing opportunities for future implementation of an overall basin-wide drainage plan. See Figures 8–2 and 8–3.

The major residential boom in the post 1945 era relied on the total subdivision of land into individual lots, often with the complete stripping of natural site features and their replacement by an “efficient” design. However, some proposed residential developments clustered dwellings and created common open space, seeking to preserve and enhance natural site

**Mechanical Measures**

1. **Land Grading**: Grade only those areas necessary for immediate construction. Minimize cut and fill; avoid heavy grading.

2. **Bench Terraces**: Constructed across the slope of the land to break long slopes and slow the flow of runoff.

3. **Subsurface Drains**: Sometimes required at base of fill slopes to remove excess ground water.

4. **Diversions**: Ridges and channels used to divert runoff away from erodible slopes, particularly useful along highway embankments.

5. **Dikes**: Useful around large parking lots to collect runoff for gradual release through grassed outlets or subsurface drains.

6. **Sediment Basins**: A permanent or temporary dam to detain runoff and trap sediment.

7. **Filter Berms**: Gravel or straw bale dikes used to filter stormwater runoff prior to discharge.

8. **Vertical Drainage**: Techniques to infiltrate excess runoff water to rapidly permeable subsoil to reduce excess runoff water.

**Vegetative Measures**

1. **Vegetative Protection**: Conserve maximum amount of ground cover particularly along stream corridors.

2. **Temporary and Permanent Seeding**: Seeding will add stability to soils which are not needed either permanently or temporarily for construction. Grasses, legumes, trees, shrubs, vines and ground covers can be used.

3. **Mulch**: Straw mulch can be used to protect constructed slopes and other areas regraded at an unfavorable time for seeding.

4. **Stream Channel Stabilization**: Erodable or erosion-prone channels and stream banks can be stabilized by use of vegetation, rip-rap and mechanical measures.

attributes. These various innovative concepts of land planning, which have now become grouped under the common title Planned Unit Development, present opportunities for stormwater management consistent with the emerging new philosophy advocated by this report. Traditional subdivision design practices will also benefit from the new stormwater management approaches, but not always to the same degree.

8.1.2.1 The Basic Concepts. The water falling on a given site should, in an ideal design solution, be absorbed or retained onsite such that development

![Figure 8-2](image)

**FIGURE 8–2** Partial List of Erosion and Sediment Control Measures for Construction Sites.

278

![Figure 8-3](image)

**FIGURE 8–3** Sample Erosion and Sedimentation Control Plan.
would not significantly alter the quantity and peak rate of runoff leaving the site from that produced from undeveloped sites.

Just as the importance of water quality is being increasingly recognized, a major new emphasis needs to be placed on the identification and application of "natural" engineering techniques to preserve and enhance the natural features of a site, and to maximize economic-environmental benefit. "Natural" engineering techniques are those which capitalize on, and are consistent with, natural resources and processes. Engineering design can be used to improve the effectiveness of natural systems, rather than negate, replace or ignore them.

Among the new trends in basic philosophy that should be pursued are:

(a) Concurrent recognition of the convenience drainage and overflow or flood conveyance elements of existing and proposed drainage systems, the use of on-site detention storage to reduce downstream peak flows, the use of land treatment systems to manage stormwater, and a recognition that temporary bonding at various points in the system, including on the individual lot, is a potential design solution rather than a problem in many situations.

(b) A continuing recognition that there is a balance of responsibilities and obligations for collection, storage and treatment of stormwater to be shared by individual property owners and the community as a whole.

(c) A new recognition that stormwater is a component of the total water resources of an area which should not be casually discarded but rather should be used to replenish that resource. Stormwater problems signal either misuse of a resource or unwise land occupancy.

(d) A growing emphasis on the recognition that every site or situation presents a unique array of physical resources, occupancy requirements, land use conditions and environmental values. Variations of such factors within a community generally will require variations in design standards for the optimal achievement of runoff management objectives.

The above key concerns, while not all-inclusive, embody a basic philosophy that should receive consideration. Although this portion focuses primarily on residential design practices, these concepts should be considered and applied to entire drainage basins in which any development may proceed. The responsible solution for individual developments will be more difficult to achieve in the absence of basin-wide plans, particularly where current practices are based on traditional drainage concepts. For example, if current practices allow an upstream development to use traditional drainage approaches that increase runoff, a development relying on new concepts might be unable to accommodate the amount of excess runoff thereby generated without additional significant costs. The approaches suggested herein should allow development to proceed on individual projects in the absence of a basin-wide plan, since the strategy for the retention and attenuation of peak runoff and total runoff to values not significantly different from predevelopment levels would normally be compatible with any future plan that might evolve for a watershed.

8.1.2.2 A Definition of "Stormwater Runoff System." The term "stormwater runoff system" used herein is, first of all, composed of both natural and man-made elements. In the past, designers have often failed to capitalize upon natural elements and have at times ignored them when a constructed element was installed. These components include not only those which contain and convey stormwater, but also those which absorb, store and otherwise use stormwater rather than dispose of it.

Within a single system, there are components that are designed primarily to obtain convenience at the smallest scale of the system, e.g., the individual site of intersection, during minor or frequent storms. During an infrequent or major storm, the capacities of many of the convenience-oriented components will be exceeded and flow capacity must be provided by other components designed to provide safety and minimize damage throughout the system, from the individual site to the discharge point of the drainage basin to downstream areas. It must be recognized and emphasized that a total stormwater runoff system cannot be expected to prevent inconvenience and minor property damage during a major storm event. A design that would eliminate all such stress would be fundamentally unreasonable and economically infeasible. Expected damages from a major runoff event would include minor erosion and scour, damage to lawns and vegetation, and damage to unwisely located structures, but flooding or undermining of buildings or essential facilities should not occur.

Thus a stormwater runoff system should be considered as a single system having three purposes: (1) the control of stormwater runoff to minimize damage to property and prevent physical injury and loss of life which may result from major storm events having a frequency of occurrence of less than once in 50 years; (2) the control of stormwater runoff to minimize inconvenience and disruption of activity from more frequent storm events (greater probability of occurrence than once in 50 years); and (3) maximizing the infiltration of stormwater.

There are a wide range of analysis techniques available for guiding the design of stormwater runoff systems. The choice of technique must be suited to the size and complexity of the area, the degree of safety and convenience sought and the cost factors involved. Regardless of the techniques selected to guide the design, the following factors must be considered:

1. Rainfall
   a. historic
   b. predictable future
   c. bases for design

2. Drainage Area Characteristics
   a. at the site
   b. downstream,
   c. upstream
   d. basin-wide

3. Land Use Characteristics
   a. present
b. future-short term
c. full development

4. Design Options
   a. on site detention/storage
   b. overland flow
   c. channel capacity; volume/storage
   d. storage, detention, routing
   e. natural drainage system

5. Risk Analysis
   a. to life
   b. to property
      i. on the site
      ii. downstream
   iii. upstream

6. Costs
   a. initial
   b. amortization
   c. operation
   d. maintenance
   e. replacement
   f. inconvenience
   g. flood damage

8.1.2.3 Storage Considerations and Criteria. One of the primary factors to consider in stormwater runoff management is storage. The availability or absence of facilities for temporary or permanent runoff storage is an important element in design. Storage should not, however, be considered a cure-all for stormwater runoff management. In many instances, the storage capacity required to assure both maximum safety and convenience will not be economically feasible, but may still be desirable.

Providing stormwater storage can reduce peak runoff rates; aids recharge to groundwater; provides attenuation mechanism if stormwater is to be treated; and reduces potential of downstream flooding, stream erosion and sedimentation.

Storage occurs naturally to some degree in most Long Island watersheds. Natural storage is provided by surface depressions and interception by vegetation. Greater storage is possible where the depressions and swales in the drainage area have highly pervious recharge areas. Much natural storage is usually lost through development. This volume can be replaced using swales, recharge techniques, vegetal practices, and by utilizing special inlets that meter the outflow from planned ponding areas. Where detention storage is used, overland conveyance must be designed with sufficient capacity to assure no downstream damage from major storm events. Large scale temporary retention storage should be used to replace storage loss due to the increase of impervious surfaces associated with development.

Rooftop and parking lot ponding are two methods of stormwater retention. In addition, recharge facilities and dry ponds may be utilized to control large amounts of stormwater runoff.

8.1.2.4 Degrees of Storage. Different degrees of storage should be considered in residential design. The lowest degree is the natural storage provided by surface depressions and by foliage and ground cover interception of rainfall. To take advantage of this storage, natural ground cover should be maintained. Small volumes of temporary storage can be provided for in the design of swales, pipes and channels. Outlets from temporary storage can be designed to attenuate peak outflow, and safely discharge stormwater runoff thus assuring protection of adjacent and downstream properties from flooding, erosion or sedimentation damage. Temporary storage facilities include rooftop and parking lot ponding, recharge sedimentation basins and normally dry ponds.

The comparative amounts of storage that may be achieved using different combinations of facilities will vary. The designer of storage should determine that the cost of storage provisions will not exceed benefits accrued and that the designs will be economical to maintain. The residential storage system should be coordinated with watershed and regional storage plans for flood control, water supply and recreation.

8.1.2.5 Other Storage Considerations. In creating ponds or lakes, the following considerations are worthy of mention.

1. Access to shorelines may be effectively limited to desired locations by planting thorny decorative shrubs.

2. Lake bottoms within ten feet of the shore should be so graded that water depth normally will not exceed eighteen inches, to simplify immediate rescue of small children.

3. Extensive areas of shallow water, especially in upper reaches of the lake, should be avoided to prevent undesirable weed growth.

4. Dense plantings of shrubs that will act as barriers to automobiles are appropriate where vehicles might otherwise run into the lake, especially at night.

5. Paved walkways roughly paralleling the shoreline, low-level night lighting, fixed benches, floored rain shelters and sensitive landscaping can add considerably to the charm of a lake or pond setting, and to the desirability of the surrounding neighborhood. Massive plantings of seasonally colorful shrubs, such as azaleas, redbud, dogwood or Japanese maple, can help publicize an area and create particular pride of ownership throughout the neighborhood.

8.1.2.6 Streets and Curbs. The primary purpose of residential streets is to provide vehicular access to homes and community facilities. Vehicles using the streets will vary from routine automobile traffic to larger delivery and service trucks and emergency police and fire vehicles. Streets also have several secondary functions. One is to provide routes for pedestrian and bicycle traffic; another, more relevant, is to collect and convey stormwater runoff.
Planning a drainage system should be done simultaneously with street layout and gradient planning, and careful consideration should be given to the following:

1. The functions of streets as parts of the stormwater management system.
2. Street slopes in relation to stormwater capacity and flow velocity in gutters and/or street swales.
3. The location and sizing of street culverts. Culverts may be sized to create temporary upstream storage if there is proper consideration of earth bank stability and potential overflow effects during major flood conditions.
4. Location of streets in relation to natural streams, storage ponds and open channel components of the system.
5. Location and capacity of inlet points to pipes in relation to gutter slopes, the spread of water across streets and the flow of water across intersections.
6. Coordination of street grades with lot drainage. Positive slope away from all sides of the house must be accomplished. Lot drainage becomes difficult when there is less than one and one-half to two percent (usually fourteen to 24 inches) fall from the earth grade at the center rear of the house to the street curb at the lowest front corner of the lot.
7. Use gutters, downspouts and dry wells on all newly constructed houses to reduce volume of runoff water in roads.

8.1.3 Water Conservation Practices

Water conservation is a means for reducing the necessary capacity of facilities needed to collect, convey and treat both domestic and industrial waste; for reducing per capita usage of water; and for the proper return of water to the aquifer.

The domestic use of various devices available to curtail water use include faucet aerators, flow control shower heads, automatic flush valve toilets, shallow trap water closets and the English dual cycle water closet. Figure 8–4 lists water saving devices ranked by cost-effectiveness.

Industrial, commercial and agricultural reuse of water is another option. For instance, in-plant recirculation of water would reduce total water requirements and also decrease waste discharges.

Planners of wastewater systems should be cognizant of possibilities for reuse of effluent, generally after some type of treatment, and alert both local governments and water users to such possibilities. Treated sewage effluent, for instance, can be utilized for irrigation purposes. However, special attention should be paid to land use impacts by such reuse practices (i.e., heavy metal residues in food, reduction in soil permeability, etc.).

Water conservation practices could be encouraged by incorporating them within building codes, state and local regulations, and the initiation of pricing policies that encourage less wastage of potable waters – particularly when used for non-drinking purposes.

<table>
<thead>
<tr>
<th>Hardware Device</th>
<th>Water Savings GPCD (1)</th>
<th>Estimated Installation Costs</th>
<th>Water-Savings Cost-Effectiveness Total $ GPCD (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Aerator for Lavatory and Kitchen Sink</td>
<td>0.5</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>2. Dual Cycle Water Closet</td>
<td>17.5</td>
<td>100</td>
<td>30</td>
</tr>
<tr>
<td>3. Limiting Flow Valves for Shower</td>
<td>6</td>
<td>35</td>
<td>15</td>
</tr>
<tr>
<td>4. Batch-type Flush Valves (2) for Water Closet</td>
<td>15.5</td>
<td>120</td>
<td>38</td>
</tr>
<tr>
<td>5. Vacuum Flush Toilet (for 100 homes)</td>
<td>22.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6. Recycle Toilets</td>
<td>24.7</td>
<td>300</td>
<td>25</td>
</tr>
<tr>
<td>7. Batch-type Flush Valve (1) for Water Closet</td>
<td>7.5</td>
<td>75</td>
<td>30</td>
</tr>
<tr>
<td>8. Shallow Trap Water Closet</td>
<td>7.5</td>
<td>80</td>
<td>30</td>
</tr>
<tr>
<td>9. Urinal with Batch-type Flush Valve</td>
<td>7</td>
<td>150</td>
<td>25</td>
</tr>
<tr>
<td>10. Washing Machine with Lever Control</td>
<td>1.2</td>
<td>35</td>
<td>0</td>
</tr>
<tr>
<td>11. Vacuum Flush Toilet (for Single Homes)</td>
<td>22.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>12. Limiting Flow Valves for Lavatory</td>
<td>0.5</td>
<td>45</td>
<td>23</td>
</tr>
</tbody>
</table>

(1) GPCD, gallons per capita per day
* Source: Wenk (1976)

FIGURE 8–4 Water Conservation – Plumbing Devices Ranked by Water Saving Cost Effectiveness.

8.1.4 Watershed Management

A properly implemented watershed management program is essentially a preventive and conservation approach primarily meant to prevent deterioration of groundwater supplies in undeveloped areas. It is a useful adjunct to other waste management options in the following ways:

1. It makes possible a broader range of community development patterns in non-watershed areas.
2. It helps minimize erosion and associated runoff problems.
3. It assures the availability of potable waters.
4. It can provide the locations for eventual recharge.
5. It is an important land use alternative in the attainment of Suffolk County’s long standing open space protection program.
6. It may provide a significant cost-effective benefit over sewering approaches.
8.1.5 Land Use Practices

Land use strategies loom as a promising complement to treatment technology. The way the use of land is managed is pivotal to the entire pollution problem. Without environmentally sensitive land use planning and control, investments in wastewater treatment facilities can easily be squandered. Well conceived and administered land use controls will allow reasonable levels of growth with minimal resource degradation and with a more favorable cost-effectiveness rate.

8.1.5.1 The Land Use Planning Process. Land use planning is a decision process as to how land should best be allocated and used based on current conditions and anticipated future events, i.e., as expansion, abandonment and renewal proceed through time. In simpler terms, it is an effort to anticipate future events and desires so that better decisions can be made today. The planning process has traditionally been oriented toward improving the man-made physical environment (i.e., the location, character and quality of housing, commercial and industrial activities, transportation, utilities and community facilities).

The interrelationships of land use and water quality are numerous and quite complex. Figure 8–5 is a schematic diagram of these relationships. Summarily stated, population and economic growth, private market forces, land use regulations, public services and facilities, and the natural features result in a particular land use pattern and land consumption rate. This pattern, with its underlying determinants, can cause water quality problems:

Damaging, and perhaps poorly distributed, point source waste discharges (i.e., domestic, industrial and power plant discharges).

Damaging non-point loads generated by urban and rural land uses as well as construction activities.

8.1.5.2 Overview of Land Use Strategies and Techniques. In developing land use strategies for water quality management programs, there are several goals toward which such efforts should be aimed (See Figure 8–6):

Reduce and balance point discharges with the water quality standards and established wasteload allocations for receiving waters. The planner should seek to distribute waste-generating development in a manner which does not overburden the receiving water body at points of waste discharge.

Reduce and balance harmful non-point discharges. The planner should set forth plans and programs which minimize both the production of non-point pollution and the entry of such wastes into water bodies.

Conserve the natural features and natural systems which protect water quality and quantity. The planner should plan and control

FIGURE 8–5 Schematic Diagram of the Land Use/Water Quality Relationship.
land use in ways which allow the natural terrain and ground cover to perform its runoff control, groundwater recharge and waste absorption functions.

Planning strategies oriented toward these general goals fall within three general categories which range in scope from areawide to site specific:

8.1.5.3 Regional Strategies. On the regional level, water quality management planners have responsibility for considering a wide range of options in meeting water quality standards and effluent limits. The achievement of this objective requires the sensitive consideration of growth objectives, alternative arrangements of the area’s physical structures, composition of settlement areas and the location and character of open space and rural land uses. Four land use strategies which have potential for improving water quality at the regional level are:

a. Modify growth rates.
b. Modify growth distribution.
c. Conserve environmentally sensitive areas and open space.
d. Control the siting of critical uses. See Figure 8–7.

8.1.5.4 Land Management Strategies. Not all land use water quality problems can be dealt with at the planning or planning-administration levels. Many water quality problems stem from poor land management in both rural and urban areas. The management of land is of great significance to the control of non-point pollution. The strategies which relate to land management include:

a. Control construction-related erosion.
b. Utilize agricultural best management practices to reduce storm runoff and control erosion.
c. Manage flood plain and shoreline uses.
d. Control resource extraction activities.

8.1.5.5 Site Development Strategies. Careful attention to the water quality impacts of individual projects can have important results, if not individually, at least cumulatively. At the site planning level, three strategies may prove fruitful, depending upon local conditions and institutional factors:

a. Modify site location practices.
b. Modify project size and/or mix.
c. Improve site planning and development.
d. Institute development site erosion control plan.

Techniques for implementing these strategies are diverse. In addition to traditional, well tested regulatory procedures, innovative policymakers and planners are devising an increasing array of controls in an attempt to weld environmental considerations to land planning and development endeavors. These implementation techniques include:

- Regulations
- Incentives and disincentives
The land use strategies outlined here are not brand new, visionary ideas. They are being considered and frequently implemented in various places throughout the country. The mechanisms to implement these strategies are even more familiar. It is more universal acceptance and adoption of these strategies that is now needed. Localities pursuing strategies to modify growth distribution, control location of critical uses, minimize erosion and sedimentation, and all the other strategies will reap benefits of improved water quality and, collateral preservation of other natural and human values.

In the matrix shown in Figure 8-9, the strategies for land use are evaluated as to long and short-range potential for reducing water pollution and as to ease of implementation. Each cell was rated from one to five, with the number assigned by assessment of the strategy in terms of (a) potential effectiveness under ideal conditions, and (b) realistic feasibility for implementation, given existing and anticipated political and economic constraints. The short-term payoffs of the strategies, it will be noted, have lower scores than the long-term payoffs. Lead time involved in implementing strategies, both legally and fiscally, generally means that full-scale implementation seldom comes about in the short range. More importantly, since land use strategies seldom begin with a pure water base, impact of existing and non-point sources will be felt for years to come, as tomorrow's treatment laborers catch up with yesterday's mistake. However, adoption of appropriate land use strategies will not only minimize water quality degradation from new development, but water quality could be expected to improve as poor land use patterns and practices are phased out.

In the short term, it is concluded that the greatest likelihood of water quality improvement would be obtained by pursuing control of critical use siting, erosion control and improved site planning and development. In the more distant future, additional benefits will be possible through more sophisticated planning for growth distribution and through protection of environmentally sensitive areas, including flood plains, shorelines and watershed conservation areas. See Figure 8-10.

The payoff index also indicates that the most effective strategies are those which can be implemented most easily. For example, modifying the growth rate has a low hypothetical payoff, not because the strategy is not valid, but because widespread public acceptance and implementation are difficult to achieve. The matrix also summarizes tools for implementing the land use strategies. The tools were also evaluated as to political and economic feasibility as well as technical efficacy. Public controls such as zoning requirements, permits and other regulations can generally be enacted with little strain on the local budget or little widespread objection. Review and advisory services are not as effective simply because compliance cannot be enforced. The matrix also illustrates that all tools cannot be used for every strategy, but that there are several means available to implement each land use strategy to improve water quality.
8.2 STRUCTURAL APPROACHES

8.2.1 Water Pollution Control

The Alternative Systems Diagram for Water Pollution Control, Figure 8–11, comprises: a network of significant wastewater sources (e.g., domestic wastewater); significant waste parameters (e.g., biochemical oxygen demand, or BOD); wastewater treatment alternatives (e.g., activated sludge); and effluent disposal alternatives (e.g., groundwater recharge). Alternatives for disposal for the concentrated wastewater solids are also included.

It should be noted that the diagram is not all-inclusive of every wastewater source, discharge parameter or treatment process. For example, there are other constituents of wastewater besides BOD, nitrogen, phosphorus and dissolved solids. However, these are considered the most significant with respect to the selection of a suitable treatment system and mode of effluent disposal.

The impact of water pollution control systems upon coastal environments may be considered to be made up of three separately identifiable aspects. First and foremost, the impact of the effluent and the residual contaminants it carries must be considered. Secondly, the construction and

<table>
<thead>
<tr>
<th>Strategy</th>
<th>Implementation Techniques</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modify Growth Rate</td>
<td>Policy Recommendations and Decisions by Governmental Leaders</td>
<td>State and Local Studies in Virginia, Michigan, Fairfax County, Virginia, and Florida</td>
</tr>
<tr>
<td></td>
<td>Zoning</td>
<td>Population Ceiling</td>
</tr>
<tr>
<td></td>
<td>Public Service Policy</td>
<td>Imposed in Lake Tahoe, Proposed in Boca Raton, Florida</td>
</tr>
<tr>
<td>Modify Growth Distribution and</td>
<td>New Town Assistance Programs</td>
<td>Special New Town Incentives or Assistance: Texas, Arizona, California, Ohio</td>
</tr>
<tr>
<td>Density</td>
<td>Locational Decisions for Major Public Facilities such as Sewage Lines and Treatment Plants and Location of Major Highways</td>
<td>Extensive Agricultural, Low Density Zoning, State or Local Level, Hawaii; New Jersey Communities — Ramapo, N. Y. Growth Plan</td>
</tr>
<tr>
<td></td>
<td>Zoning</td>
<td>Nassau-Suffolk Regional Development Plan</td>
</tr>
<tr>
<td></td>
<td>Local Moratoria on Sewage Taps or Extensions</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Regional Planning and Adoption of Policy and Controls by Member Jurisdictions</td>
<td></td>
</tr>
</tbody>
</table>

Control Location of Critical Uses

- Collaborative Regional Planning & Funding
- Zoning
- A-95 Review; Environmental Impact Statements
- Federal or State Regulation of Certain Uses

Examples

- Plan for Year 2000, D. C. Area
- Sewer Moratoria in Florida & Washington, D. C.
- Prince Georges Co. Staging Policy
- National Parks and Forests, Local or State Parks
- Acquisition and Easement Programs: Boulder, Colo., New Jersey, Florida, Ohio
- Maine Site Location Law
- Suffolk County Farmlands Acquisition Program
- San Francisco Bay Conservation & Development Commission
- Shoreline or Flood Plain Zones — Wisconsin, Minnesota, College Township, Pa., Buffalo County, Wisc., Delaware
- Suffolk County Greenbelts
- Favorable Taxes on Undeveloped Land: Va., Cal., Fla., Conn.
- Proposed Abolition of Tax Benefits That Now Encourage Development in Certain Resort Areas
- Special Exception or Permit Required — State or Local Level: Maine Site Location Act, Local Zoning Ordinances, New York State Environmental Quality Review Act
- Proposed: Power Plant Siting Act, Sediment Control Act, Toxic Waste Disposal Control Act
FIGURE 8–9 Assessment of Payoffs and Implementation Requirements.

operation of the physical plant itself will impact the environment, and must be accounted for. Last is the impact, although secondary and indirect, of the final disposition of the wastewater solids which were concentrated and removed from the wastewater stream. The Alternative Systems Diagram delineates the possible alternatives and combinations of alternatives for treating the wastewater and disposing of the effluent, as well as the residual solids.

8.2.2 Conventional Treatment Technology

The current accepted method of wastewater treatment is that illustrated in Figure 8–12. This system is referred to by a variety of titles such as: conventional treatment, biological treatment and primary plus secondary treatment. This treatment has been specifically developed to remove suspended solids, biodegradable organics and microorganisms from wastewater.
### FIGURE 8-10 Environmentally Sensitive Areas.

Suspended solids produce sludge banks in rivers, biodegradable organics lower the oxygen resources of lakes and rivers and microorganisms are the source of most water-borne disease. In the past, it was only thought necessary to substantially remove these three classes of pollutants from wastewater prior to discharge to avoid adverse environmental effects. This is no longer generally true because of the newer awareness of other serious contaminants—the danger of which was not previously realized.

Although in many locations the conventional system no longer is sufficient of itself, it is the base upon which some newer, more effective treatment systems are constructed. Therefore, a short description of this system and the degree of treatment which it can provide is given.

Wastewater is first passed through preliminary treatment of screening and grit removal. Preliminary treatment is utilized to protect pumps and pipes downstream from being harmed by large articles and abrasives which are often found in sewage. Next, primary sedimentation is provided to remove relatively large organic solids. The following step is biological oxidation in which a large quantity of microorganisms is contacted with the sewage in an aerobic environment. In this step, the microbes convert the soluble and colloidal organics producing settleable masses of microbes plus carbon dioxide and water. Two systems which are used for biological oxidation are the trickling filter and activated sludge. The microbes which are active in biological oxidation settle by gravity from the flow in the secondary sedimentation tank since they cannot be discharged with the effluent. Some are recycled to the entrance of the biological oxidation process in order to maintain an adequate population in this unit, the remainder are sent to the sludge handling section. After passage through the secondary sedimentation tank the flow is disinfected, usually by the application of chlorine, and discharged. No attempt is made to sterilize the effluent since all that is required is destruction of pathogens (disease producing bacteria). Most pathogens are more susceptible to the effects of chemical disinfectants than the non-pathogens. It is important to note that the microbes which function in the biological oxidation process are non-pathogens.

When considering the cost and effectiveness of waste treatment technology, sludge disposal is often overlooked. This is unfortunate as up to half the cost of the conventional treatment train can be charged to sludge handling and disposal. Sludge is a thick suspension of organic solids which is drawn from the bottom of the primary and secondary sedimentation tanks. Disposal usually involves two steps, the first of which is dewatering. Dewatering is generally necessary because the cost of the second step "ultimate disposal" is proportional to the volume of the sludge. Unfortunately, sludge is usually difficult to dewater unless chemicals are added and/or it is subjected to anaerobic biological treatment. The liquid removed during the dewatering step is referred to as supernatant and is usually recycled to the initial portion of the treatment plant, i.e., path from 6 to 1 to 2 in Figure 8-12.

Among the methods employed for ultimate disposal are landfill, land spreading, incineration, wet oxidation and processing for recovery of useful byproducts. At present, research in ultimate disposal is aimed at reduction of costs of environmentally acceptable ultimate disposal methods.

Table 8-1 illustrates the performance which can be expected of a typical well-operated conventional treatment plant. The results have been presented both as percent removal and typical effluent characteristics. This type of plant cannot provide significant removal of phosphorus, nitrogen or salts. Suspended solids, organic removal and microorganism removal are significant.

#### 8.2.3 Biological-Physical Treatment

The inability of gravity sedimentation in the final clarifiers to remove small, light particles places a limit on the capability of conventional treatment
to reduce suspended solids and BOD in the treatment of wastewater. The usual range of this limit is indicated by the figures presented for typical effluent characteristics in Table 8-1. Integration of a better liquid-solid separation device into the treatment scheme is thus one method of up-grading conventional treatment. Figure 8-13 illustrates the simplest type of advanced waste treatment, the biological-physical treatment system. This system is identical to the conventional treatment system with the exception of the provision of an additional solids removal step after secondary sedimentation. Three types of systems have been widely used to upgrade the performance of conventional systems; microstrainers, deep bed filtration and chemical treatment.

8.2.3.1 Microstrainers. The simplest system is the microstrainer, illustrated in Figure 8-14. Microstrainers are rotating drums on which woven filter fabrics, usually of stainless steel, are mounted. Incoming wastewater flows into the drums along the axis and then passes through the filter cloth which forms the drum surface. Filter rates are six to ten gallons per minute per square foot with head differential at about six inches of water. Control of headloss is obtained by washing the solids from the screen as that portion of the screen rotates to the top of the device. The backflow liquid discharges to a trough in the interior of the drum from which it is recycled to the sedimentation basin. Ultraviolet light mounted above the screen is used to prevent biological growth from blinding the screen.

Table 8-1

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Effluent mg/l</th>
<th>% Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suspended Solids</td>
<td>20–30</td>
<td>80–90</td>
</tr>
<tr>
<td>BOD</td>
<td>15–25</td>
<td>80–90</td>
</tr>
<tr>
<td>COD (Chemical Oxygen Demand)</td>
<td>30–60</td>
<td>70–80</td>
</tr>
<tr>
<td>Ammonia-N</td>
<td>15–25</td>
<td>0–10</td>
</tr>
<tr>
<td>Phosphorus</td>
<td>6–10</td>
<td>0–40</td>
</tr>
<tr>
<td>Coliform</td>
<td>1 per ml</td>
<td>99.999</td>
</tr>
</tbody>
</table>
Microstraining is fundamentally a screening process, thus the performance is a function of screen size. Table 8—2 illustrates this effect. It can be seen that substantial improvement in effluent characteristics can be obtained by this technique. BOD removal closely parallels suspended solids removal as most of the BOD exerted by a conventional plant effluent is due to biological floc which escaped removal in the secondary tank. An extensive study of microstrainers was conducted in Chicago; based on this study a fifteen million gallons per day plant was designed and is now being installed.

**Table 8—2**

<table>
<thead>
<tr>
<th>Location</th>
<th>Screen Size Microns</th>
<th>Suspended Solids % Removal</th>
<th>BOD % Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Luton, Eng.</td>
<td>35</td>
<td>55</td>
<td>30</td>
</tr>
<tr>
<td>Bracknell, Eng.</td>
<td>35</td>
<td>66</td>
<td>32</td>
</tr>
<tr>
<td>Chicago, Ill.</td>
<td>23</td>
<td>71</td>
<td>74</td>
</tr>
<tr>
<td>Brampton, Ont.</td>
<td>23</td>
<td>57</td>
<td>54</td>
</tr>
</tbody>
</table>

**8.2.3.2 Deep Bed Filtration.** A somewhat more costly but more reliable upgrading technique is the use of deep bed filtration. A typical deep-bed filter installation is shown in Figure 8–15. In this technique, the wastewater is run at two to ten gallons per minute per square foot through several feet of a granular material wherein the suspended solids are deposited. Most of the removal takes place on the surfaces of the grains rather than in the bed pores. The accumulation of solids in the bed eventually produces an excessive pressure drop. At this point, the filtration run is terminated, and the bed is cleaned. Cleaning always involves an upward flow of water at a rate sufficient to expand the media, thus allowing for removal of the entrapped floc. In addition, an air scour and/or a water jet surface wash may be employed to remove solids which adhere to the grains. The nature of wastewater floc is such that it is almost mandatory for air scour or surface wash to supplement normal water backwash in order to achieve adequate cleaning.

Filter media ranges in size from ten mesh to 80 mesh and until recently only a single type of media has been employed. After backwash, however, a single medium filter is graded with the smallest size particles at the surface. This restricts filtration to the top layers of the bed. To counteract this effect, dual-media or multi-media filter beds are now coming into use. These employ granular materials of such sizes and specific gravities that after backwash the coarser material is always at the top of the filter. Typical combinations are coarse coal and fine sand or coarse coal, medium sand and fine garnet. These new combinations allow much longer filter runs at higher flow rates than single medium systems.

Typical performance of deep-bed filter systems for treatment of secondary effluent is given in Table 8—3. Removals of pollutants are only slightly better than with microstrainers, but deep-bed filters react well to shock loads.

**FIGURE 8–15**  Typical Rapid Sand Filter.

**Table 8–3**

<table>
<thead>
<tr>
<th>Location</th>
<th>Flow Rate gal/min/sq. ft.</th>
<th>Suspended Solids % Removal</th>
<th>BOD % Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Luton, Eng.</td>
<td>1.5—4</td>
<td>72—91</td>
<td>52—70</td>
</tr>
<tr>
<td>Chicago, Illinois</td>
<td>2—6</td>
<td>70</td>
<td>80</td>
</tr>
<tr>
<td>Los Angeles, California</td>
<td>-</td>
<td>46</td>
<td>57</td>
</tr>
<tr>
<td>Tudor, England</td>
<td>-</td>
<td>85</td>
<td>—</td>
</tr>
</tbody>
</table>

**8.2.3.3 Chemical Treatment.** Suspended solids and BOD removal across a microstrainer or deep-bed filter will rarely exceed 80 percent because these processes cannot remove colloidal matter from wastewater. Chemical coagulation is the only feasible method for removal of colloids. In this process, chemicals added to the wastewater entrap the colloids into a floc which can then be removed by sedimentation or filtration. A large variety of chemical agents have been found useful as coagulants, these include salts of iron, salts of aluminum, lime and organic polymers. In addition to removing colloids, chemical treatment will coagulate the suspended solids in sewage forming large, dense flocs which are amenable to removal by sedimentation.

Chemical treatment itself only tends to entrap or coagulate colloids and suspended solids. Removal of these coagulated impurities requires liquid-
solid separation techniques. Consequently, a system which employs chemical treatment: for upgrading can be quite simple or quite complex depending on the degree of removal and reliability desired.

The simplest system would employ chemical addition and flocculation between the biological oxidation unit and the secondary tank. A more comprehensive system would employ the system above with a deep-bed media filter following secondary sedimentation. The most complex system, which is illustrated in Figure 8–16, provides chemical addition, flocculation, sedimentation, and deep-bed filtration after the secondary sedimentation step. It will be noted that a microstrainer is not included in any of these flow schemes because it does not perform well with a chemical floc.

![Typical Flow Diagram of Clarification System](image)

**FIGURE 8–16** Typical Flow Diagram of Clarification System.

### 8.2.4 Tertiary Treatment

The term tertiary treatment causes some confusion in that it has a variety of meanings. In the general sense, it represents any stage of treatment or any treatment step applied after secondary treatment. This definition encompasses most of the unit operations and processes discussed in this chapter. In a more restricted sense, it represents a specific combination of treatment procedures applied after secondary treatment. In this restricted sense, tertiary treatment is the combination of chemical coagulation, flocculation, sedimentation, deep-bed filtration, and activated carbon adsorption. Nitrogen removal procedures are optional in this treatment train. Figure 8–17 illustrates primary, secondary, and tertiary treatment (restricted sense) combined together into a system which provides biological, physical, and chemical treatment.

The ultimate goal of the combination of all these treatment procedures is to produce renovated wastewater, i.e., wastewater which can be reused.

![Biological-Chemical-Physical Treatment System—Primary, Secondary and Tertiary](image)

**FIGURE 8–17** Biological-Chemical-Physical Treatment System—Primary, Secondary and Tertiary.

### 8.2.5 Phosphorus Removal

The key role of phosphorus in the process of eutrophication (aging of lakes and impoundments) has been known for many years. However, until recently, eutrophication of the Island's waterways was not a significant problem. Consequently, control of the phosphorus level in streams and lakes was not considered an important pollution control problem. Since the end of World War II, however, the rate of eutrophication has increased to the point where it is a major water quality problem. Although other nutrients play a role in eutrophication, recent articles have indicated that much of the recent increase in eutrophication rate is linked to significant increases in phosphorus discharge to rivers and lakes. Virtually all of the increase in phosphorus discharge is due to the activities of man and can be termed cultural eutrophication.

The major sources of phosphorus contributing to eutrophication are domestic sewage and agricultural runoff. Domestic sewage is the primary source in critical areas. Phosphorus gains entrance to sewage from human body wastes (primarily urine) and through the use of condensed inorganic phosphate compounds from detergents. Each of these sources accounts for about half of the phosphorus in domestic sewage. Treatment of domestic sewage to remove a significant portion of the phosphorus contributed by human wastes and detergents would have a significant effect on eutrophication rate.
8.2.5.1 Phosphorus Removal in Conventional Treatment. Removal of any pollutant from wastewater requires that it be converted to either an insoluble gas or an insoluble solid. Because none of the chemically stable forms of phosphorus is a gas at normal temperature and pressure, removal from wastewater is dependent on formation on an insoluble solid. Less than ten percent of the phosphorus discharged to municipal sewage systems is insoluble and none of the conventional treatment techniques is particularly effective in insolubilizing this nutrient. Thus, phosphorus removal in conventional treatment systems is relatively poor. Primary treatment can remove only the ten percent of the phosphorus which is initially insoluble. During secondary treatment, phosphorus removal is achieved by synthesis into the biomass followed by sedimentation and sludge wasting. However, municipal sewage contains a considerable excess of phosphorus over that required for biomass synthesis during complete utilization of the organic carbon present; thus, removals are generally limited to twenty to forty percent. Studies indicate that biological systems have the capacity for much higher removals through the mechanism of "luxury uptake." However, attempts to implement this phenomenon in actual plants have not been successful.

8.2.5.2 Phosphorus Removal by Chemical Precipitation. Fortunately, phosphorus forms essentially completely insoluble precipitates with a number of substances, thus high levels of removal can be obtained when appropriate doses of the proper chemicals are applied. A large variety of chemicals can be utilized for this purpose but economic factors dictate the use of salts of iron, salts of aluminum or lime. In selecting the chemical for use at any particular site, the factors listed in Table 8–4 should be taken into account.

Table 8–4

<table>
<thead>
<tr>
<th>FACTORS AFFECTING CHOICE OF CHEMICAL FOR PHOSPHORUS REMOVAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Influent Phosphorus Level</td>
</tr>
<tr>
<td>Wastewater Suspended Solids and Alkalinity</td>
</tr>
<tr>
<td>Chemical Cost Including Transportation</td>
</tr>
<tr>
<td>Reliability of Chemical Supply</td>
</tr>
<tr>
<td>Sludge Handling Facilities</td>
</tr>
<tr>
<td>Ultimate Disposal Methods</td>
</tr>
<tr>
<td>Compatibility with Other Treatment Processes in Plant</td>
</tr>
<tr>
<td>Potential Adverse Environmental Effects</td>
</tr>
</tbody>
</table>

Phosphorus removal is achieved by precipitation followed by liquid solids separation. For the most part, the usual liquid solids separation equipment in a treatment plant can be utilized for phosphorus removal. This results in a considerable savings in capital as well as integration of phosphorus removal into conventional treatment plant operation. In addition, it has been found that the use of chemical precipitants in conventional treatment can markedly upgrade performance of a treatment plant. This results from coagulation of organic suspended and colloidal solids by the chemicals added to precipitate phosphorus.

Figure 8–18 illustrates a conventional treatment plant with the three general sections in which phosphorus removal can be carried out. Chemicals can be added either just before the primary tank with removal taking place in the primary; in the secondary (biological) section of the plant with removal in the secondary sedimentation tank; or in a tertiary stage as was discussed in the section on suspended solids removal.

![Figure 8–18](image)

**FIGURE 8–18** Locations for Chemical Control of Phosphorus.

Table 8–5 illustrates typical results obtained with phosphorus removal in the primary, secondary or tertiary. As can be seen, good removals are obtained in all sections; however, the lowest levels of phosphorus remaining are achieved in the tertiary addition. One reason for this is that a filter is usually included in the tertiary plant, thus better removal of fine precipitites is achieved. In addition, when the flow reaches this section of the plant all the complex phosphorus forms which are more difficult to precipitate have been hydrolyzed to orthophosphate which is the easiest to precipitate.

In plants where removal in the primary phase was practiced, a major effect to note is the significant increase in BOD and suspended solids removal achieved over the removal usually obtained in the primary tank. This may be important in helping to meet water quality standards for BOD and suspended solids if the treatment plant is overloaded.

In plants where the chemical is added in the secondary section, it has been observed that much more stable operation of the activated sludge is obtained than before chemical addition. The effect of the chemical is to weigh the sludge down, preventing its loss when a filamentous or dispersed growth predominates. Even in plants which have historically exhibited excellent performance chemical addition has improved performance by
helping maintain a higher concentration of activated sludge in the aeration tank. It has been found best to add the chemical between the biological reactor and the final sedimentation tank rather than at the head end of the secondary tank.

<table>
<thead>
<tr>
<th>Place</th>
<th>Chemical</th>
<th>P Removal</th>
<th>Effluent</th>
<th>BOD Removal</th>
<th>S.S. Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>PRIMARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grayling, Michigan</td>
<td>FeCl₂</td>
<td>72</td>
<td>4.4</td>
<td>78</td>
<td>58</td>
</tr>
<tr>
<td>Washington, D.C.</td>
<td>Lime</td>
<td>95</td>
<td>0.45</td>
<td>82</td>
<td>88</td>
</tr>
<tr>
<td>Mentor, Ohio</td>
<td>Pickle Liquor</td>
<td>83.5</td>
<td></td>
<td>59</td>
<td>74</td>
</tr>
<tr>
<td>SECONDARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pomona, California</td>
<td>Alum</td>
<td>80–93</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manassas, Virginia</td>
<td>Alum</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Richardson, Texas</td>
<td></td>
<td>0.5–0.75</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TERTIARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nassau County, N.Y.</td>
<td>Alum</td>
<td>0.08</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lake Tahoe, Calif.</td>
<td>Lime</td>
<td>0.14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lebanon, Ohio</td>
<td>Lime</td>
<td>0.1–0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8–5
PHOSPHORUS REMOVAL EXPERIENCE

<table>
<thead>
<tr>
<th>Component</th>
<th>Raw (mg/l)</th>
<th>Treated (mg/l)</th>
<th>% Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Organic Matter</td>
<td>250</td>
<td>25</td>
<td>90</td>
</tr>
<tr>
<td>Organic Oxygen Demand (BOD)</td>
<td>375</td>
<td>37</td>
<td>90</td>
</tr>
<tr>
<td>Ammonia-N</td>
<td>25</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Nitrogen Oxygen Demand</td>
<td>112</td>
<td>90</td>
<td>21</td>
</tr>
<tr>
<td>Total BOD</td>
<td>487</td>
<td>127</td>
<td>74</td>
</tr>
</tbody>
</table>

8.2.6 Nitrogen Control
Nitrogen can exist in the aquatic environment in any one of four forms: organic-N, ammonia-N, nitrite-N and nitrate-N. In sewage it is found primarily in the first two forms. In nature, biologically mediated reactions convert organic-N to ammonia-N which in turn is biologically oxidized to nitrite-N and nitrate-N.

Two major water quality objectives of nitrogen control are to prevent excessive build-up of nitrates in drinking water and depletion of the dissolved oxygen resources of streams by the biological oxidation of ammonia-N to nitrate-N. Table 8–6 shows the significance of the nitrogen oxygen demand (NOD) of a wastewater by comparison of the oxygen demand of raw wastewater to a well-treated secondary effluent. These data illustrate that 90 percent removal of organic oxygen demand (BOD) only results in a 74 percent removal of the total biological oxygen demand (TBOD). Nitrogen control is also important because ammonia-N exerts a chlorine demand which reduces disinfection efficiency, is toxic to fish and other aquatic life and stimulates corrosion of copper plumbing.

The least expensive method of preventing these adverse actions is to carry out the biological oxidation to nitrate under controlled conditions in the treatment plant. Control of ammonia-N is possible by altering the operation of conventional activated sludge to provide for a significant degree of nitrification. The modifications involve increases in aeration rate, contact time and sludge age, and maintenance of the pH at 8.0–8.4. The latter is quite important in view of the tendency of the nitrification reaction to lower the pH. These reactions are quite temperature sensitive showing significant rate decreases as the temperature drops below 18° Centigrade. As the temperature decreases, contact time and sludge age must be increased to compensate for the reduced reaction rates. Of all the factors listed above, sludge age is the most significant as it can be used to control the population of nitrifiers maintained in the system.

At low temperatures and when a significant diurnal flow variation exists, it has been found difficult to maintain both organic carbon oxidation and nitrification in a single reactor. In such situations, a two-stage system employing organic carbon oxidation in the first stage followed by a nitrification stage has shown significant superiority of performance. Separate sedimentation and sludge recycle is employed with each stage. This system exhibits much greater stability of performance because conditions in each stage can be established to favor the specific types of organisms desired in that stage.

Nitrification of sewage may not be sufficient for nitrogen control in some instances because an excess of nitrate in water will disqualify it from use as a potable water supply and because nitrogen in any form can serve as an algal nutrient. Phosphorus usually is the controlling nutrient in fresh waters whereas nitrogen appears to control within some estuarine environments. Areas where nitrogen is the controlling nutrient include San Francisco Bay, Lake Tahoe, the Potomac Estuary and Long Island. Removal of various forms of nitrogen from wastewater can be achieved by both biological and physical-chemical means.

8.2.6.1 Biological Denitrification. Under anaerobic conditions facultative heterotrophs will utilize nitrate ion as a hydrogen acceptor for the degradation of organic matter.

The end product of the nitrogen reduction is nitrogen gas which is essentially insoluble in water. Thus, a combination of nitrification followed by denitrification can achieve nitrogen removal from wastewater.
A three-stage biological system has been developed to provide nitrogen removal. The first stage is a high rate short aeration time (approximately two hours) biological reactor for organic carbon oxidation, and hydrolysis of organic nitrogen to ammonia. The second stage provides approximately three hours of detention and achieves essentially complete nitrification. The third stage is for denitrification of nitrate to nitrogen gas. An organic source must be added to the third stage to force the denitrification reaction to take place. A variety of substances has been evaluated for this use and methanol has been found to be far superior to all others. It is relatively inexpensive, reacts rapidly and provides only a minimum of energy for growth of new organisms. A diagram illustrating this system is given in Figure 8-19. Three alternate contactors are illustrated in the denitrification stage. Contactor I is a suspended growth contactor similar to that used in the first two stages. Solids control is somewhat difficult in this system, and it is often followed by a deep-bed filter. The other two systems supply a contact medium on which a heavy biological growth can develop. These latter systems require significantly less contact time than the three hours required in the suspended growth reactor, and they can accept higher hydraulic rates of application without fear of a washout of organisms. The coarse medium (one to two inches gravel) upflow system, requires a contact time of one to two hours and may require a supplemental filter. The fine medium down-flow system utilizes particles as small as three millimeters in beds ten to twenty feet deep. Flow rate is that of a high-rate filter seven gallons per square foot with a bed contact of ten to twenty minutes. Backwash of the fine medium units is required once or twice per day to relieve clogging. In all of the denitrification systems discussed, essentially complete denitrification has been obtained in pilot-scale experiments.

Most experience has been with the three-sludge suspended growth system. In Suffolk County, the most used system for small treatment plants has been extended aeration and gravity sand filter. Results from a 100,000 gallons per day pilot plant in Washington, D.C. are presented in Table 8-7. This table includes data on phosphorus removal, as addition of alum to the first stage was practiced both to maintain solids control and achieve phosphorus removal.

8.2.7 Breakpoint Chlorination

Chlorine reacts with ammonia-N to yield a variety of amines and inorganic compounds by a variety of pathways. The reaction requires a theoretical dose of 7.6 grams chlorine per gram of ammonia-N. The reaction occurs in a stepwise fashion; various chloramines initially form which in turn rapidly react with additional chlorine to yield nitrogen gas.

8.2.8 Ammonia Removal by Air Stripping

Soluble ammonia can be air stripped from wastewater by converting the ammonium ion to the unionized form which is the soluble gas NH₃. This is accomplished by lime treatment to precipitate phosphorus and raise the pH of the wastewater to about eleven. Wastewater is brought to the top of a cooling tower and distributed over the column packing. Forced air is drawn through the packing media to extract the gaseous ammonia from the wastewater droplets. Problems with cold weather operation due to low air temperatures and scale formation on the tower packing media have yet to be overcome. However, ammonia stripping can be an attractive method of nitrogen removal because of its relative low cost, especially when chemical cost to raise the pH is also applied for phosphorus removal.

8.2.9 Organic Carbon Removal

Organic substances are removed by most of the processes discussed in the previous sections of this paper. However, some of the organics cannot be removed by coagulation and sedimentation, nor are they amenable to biological oxidation. These have been termed refractory organics. Activities in the advanced waste treatment program were aimed at removal of refractory organics, and two methods with economic feasibility were developed: activated carbon adsorption and ozonation.

Recent studies have illustrated that these processes are just as applicable to all types of organics present in wastewater; consequently, they will be employed in the future as general purpose procedures for removal of organics in wastewater.
8.2.9.1 Activated Carbon Adsorption. The ability of activated carbon to remove soluble organics from wastewater is a result of the similarity of the surface chemistry of the activated carbon to that of the organic molecules. The ability of substances to adsorb organics is widespread; the characteristic of activated carbon which makes it unique is that it has a much higher adsorption capacity than other materials. The high capacity is the result of an extensive internal microporous structure formed during the activation process.

A generalized activated carbon treatment process is illustrated in Figure 8–20. The carbon and wastewater are contacted for a sufficient period of time for adsorption to take place, then they are separated. Eventually, the capacity of the activated carbon is exhausted, and it is removed from the contact vessel to a regeneration step. During regeneration some of the carbon is lost or consumed so make-up must be added.

![Diagram](image)

**FIGURE 8–20 Regeneration and Reuse of Activated Carbon.**

Commercial grades of activated carbon are either granular (sizes eight by 30 mesh, or twelve by 40 mesh) or powdered (greater than 300 mesh). Both forms have been found essentially equivalent in their ability to remove organics, but each type requires a different method of contacting and regeneration.

Granular carbon is contacted with sewage in columns through which the sewage flows. The columns may be either pressure vessels or gravity contactors; the former provide better operational flexibility, the latter are more economical. Flow may be upward through an expanded bed, upward through a packed bed, or downward through a packed bed. Packed bed operation provides filtration as well as adsorption but requires frequent backwash. Upflow systems allow for periodic removal of a portion of the carbon at the base, thereby providing countercurrent contact in a single vessel. However, good flow distribution is more difficult to achieve upflow than downflow. Columns can be connected in series as with the clinoptilolite system to achieve countercurrent contact. Alternately, a parallel flow arrangement can be used to achieve countercurrent contact. With the parallel flow arrangement, the starting time in service is staggered so that at any time one column is near exhaustion, one or more other columns are relatively fresh and all others are at some in-between stage. The flow from each column is blended to produce the final effluent.

Regardless of the contacting details, the fundamental system design parameter is contact time. The specific contact time to be used is a function of the effluent quality desired and the wastewater characteristics. In general, it has been found that as contact time increases, the rate at which organic removal takes place decreases. A point of essentially zero removal generally is located between 30 minutes and 60 minutes contact time. Contact time is always rated on an empty bed basis.
Regeneration of granular carbon is conducted in a multiple hearth furnace, a device which has had a long history of successful use for incineration of sewage sludge. Figure 8–21 illustrates a typical granular carbon regeneration scheme. Spent carbon is removed from the contactor by eduction, stored, partially dewatered, and then fed to the furnace. As the carbon moves from hearth to hearth it is dried, baked and finally reactivated. During the reactivation stage, the temperature is in the range of 1500° to 1700° Fahrenheit. After regeneration, the carbon is quenched and returned to a contactor. The furnace is rated at 80 pounds to 110 pounds carbon per square foot per day. Air pollution control equipment, such as an after burner and wet scrubber or cyclone, are usually required to purify the exhaust. Experience with this regeneration system at essentially full scale indicates carbon regeneration loss of five percent to ten percent per regeneration cycle. The losses are due to physical attrition in the regeneration loop and burnoff in the furnace.

**FIGURE 8–21 Activated Carbon Regeneration System.**

In systems utilizing powdered carbon, contact is brought about by mixing of the carbon and the wastewater in a reactor-clarifier contactor. This apparatus is illustrated in Figure 8–22. The carbon and wastewater are flash mixed and flocculated in the center well of the apparatus. The mixture then flows into the outer section where gravity separation of the carbon and wastewater takes place. A pool of carbon in a thick slurry form is maintained at the bottom of this contactor.

**FIGURE 8–22 Solids Contact Clarifier with Sludge Blanket Filtration.**

In this type of contactor the carbon comes to equilibrium with the organic concentration in the effluent wastewater, whereas it comes to equilibrium with the organic concentration in the incoming liquid in a granular contactor. The organic adsorption capacity of activated carbon is proportional to the concentration of organics with which it comes to equilibrium, thus it is more imperative that countercurrent contact be achieved in powdered carbon systems than in granular systems. Figure 8–23 illustrates schematically a two-stage counter flow contacting system which is utilized with powdered carbon treatment. The carbon is thickened (not shown) between stages to reduce pumping and the effluent is filtered to remove residual carbon fines. Figure 8–24 illustrates the most advanced system for powdered carbon regeneration. The slurry containing spent carbon is thickened, dewatered in a centrifuge or on a vacuum filter and injected into a fluidized bed furnace. The regenerated carbon is reslurried and pumped back into the system. In the furnace the temperature is maintained at the same level as in the multiple hearth furnace. Contact time of a carbon particle in the furnace averages several seconds. Shorter time results in only partial regeneration, long time results in particle incineration. Thus, tight control must be exercised with this regeneration scheme. To date small pilot studies have indicated the
the demand. However, granular carbon exhibits higher practical adsorption capacity, does not require complex dewatering procedures prior to regeneration and most important has been successfully regenerated at full scale. Economics demand that regeneration be incorporated into all but the smallest scale carbon adsorption plants. Thus, an engineer designing a treatment plant using carbon adsorption would most likely choose granular carbon.

Extensive processing experience has been obtained with the use of activated carbon for treatment of wastewater after biological treatment (tertiary application). Data from a number of facilities are summarized in Table 8–8. It can be seen that organic removal in the range of 70 percent to 80 percent was obtained. This is typical of the capability of activated carbon for organic removal from sewage. Data on adsorption capacity indicates a considerable spread, 0.25 to 0.87 pounds COD per pound activated carbon. This considerable spread occurs because carbon capacity is a function of desired effluent quality and influent organic concentration. Most significant, however, is that in all cases the capacity was significantly higher than that projected by an adsorption isotherm test. Biological activity on the carbon is the primary explanation of the significantly higher capacity obtained. It is theorized that partial regeneration in situ is accomplished by biological action. Naturally, this has had a favorable impact on the economics of carbon adsorption.

<table>
<thead>
<tr>
<th>Location</th>
<th>Carbon</th>
<th>Organics, mg/l</th>
<th>% Removal</th>
<th>Organics Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pomona</td>
<td>Granular</td>
<td>43</td>
<td>10</td>
<td>77</td>
</tr>
<tr>
<td>Colorado Springs</td>
<td>Granular</td>
<td>43</td>
<td>13</td>
<td>70</td>
</tr>
<tr>
<td>S. Lake Tahoe</td>
<td>Granular</td>
<td>12</td>
<td>3</td>
<td>75</td>
</tr>
<tr>
<td>Lebanon</td>
<td>Powdered</td>
<td>20</td>
<td>4</td>
<td>80</td>
</tr>
<tr>
<td>Tucson</td>
<td>Powdered</td>
<td>27</td>
<td>7.5</td>
<td>72.5</td>
</tr>
</tbody>
</table>

The process design for carbon treatment has had to be adjusted to take account of biological action. For example, only a normal upflow backwash was provided in early granular carbon contactor design. This was found to be inadequate to insure cleaning of the carbon column so air scour and surface wash are frequently added. In addition, more frequent backwash (at least once per day) must be planned for in design. Under heavy load, carbon columns become septic and hydrogen sulfide is produced. Aeration of the feed may be required or extra-chlorination to oxidize the effluent sulfide may be needed. Under heavy load, expanded bed operation becomes superior to packed beds because clogging is less of a problem and aeration is easier to accomplish.

The feasibility of this regeneration scheme with losses averaging fifteen percent. A larger version of this furnace is now being evaluated at Salt Lake City.

Both granular carbon and powdered carbon contact are excellent processes for organics removal from wastewater. Powdered carbon processes have advantages over granular carbon in that powdered carbon is cheaper than granular carbon (ten cents vs. 30 cents per pound); powdered carbon plants require a much smaller carbon inventory and can be dosed to meet
Powdered carbon contact design has also been affected by the realization that biological action may occur. It has recently been illustrated that when extensive biological action is encouraged, only one carbon contacting stage is required rather than two to achieve high adsorption capacity.

### 8.2.9.2 Physical-Chemical Treatment

Physical-chemical treatment of raw wastewater or more accurately chemical clarification—carbon adsorption was first evaluated as a treatment technique over three decades ago. It was found to produce a good quality effluent but at a cost greater than conventional processing. The recent up-grading of water quality standards has brought this treatment scheme closer to economic viability because it can produce an effluent superior to conventional processing. In addition, the developments discussed above in carbon adsorption techniques have tended to lower the cost of physical-chemical treatment. Finally, removals achieved by chemical clarification have been higher than anticipated which has also had a favorable effect on physical-chemical treatment costs.

Figure 8–25 is a general diagram of the clarification carbon treatment system. Following the standard type of preliminary treatment, the waste is dosed with chemical sufficient to achieve the desired level of suspended solids and/or organic and/or phosphorus removal required. The same chemicals are used as for phosphorus removal in the primary. Significant removals of organics, suspended solids, and phosphorus are obtained by chemical clarification as illustrated in Table 8–9. Especially significant is the organic removal which is higher than would be expected from complete removal of suspended solids. This extra removal is due to coagulation and possible chemical adsorption of colloidal and soluble organics. Up to 50 percent of the "soluble" organics in sewage may be removable by chemical coagulation.

<table>
<thead>
<tr>
<th>Plant</th>
<th>Chemical</th>
<th>Organic Removal %</th>
<th>S.S. Removal %</th>
<th>P Removal %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ewing-Lawrence</td>
<td>170 mg/l FeCl₃</td>
<td>80</td>
<td>95</td>
<td>90</td>
</tr>
<tr>
<td>New Rochelle (IZM)</td>
<td>Lime pH 11.5</td>
<td>80</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Westgate, Virginia</td>
<td>125 mg/l FeCl₃</td>
<td>70</td>
<td>75</td>
<td>80</td>
</tr>
<tr>
<td>Salt Lake City</td>
<td>80–100 mg/l FeCl₃</td>
<td>75</td>
<td>70</td>
<td>75</td>
</tr>
<tr>
<td>Blue Plains</td>
<td>Lime pH 11.5</td>
<td>80</td>
<td>90</td>
<td>95</td>
</tr>
</tbody>
</table>

After clarification is complete, the wastewater is passed to the carbon adsorption step for completion of the removal of soluble organics. Table 8–10 gives data from a number of plants which indicate that very good quality effluents can be obtained as well as high carbon adsorption capacity. As indicated previously, this is the result of biological action on the carbon. In physical-chemical treatment systems, the carbon is subjected to a heavier load than in tertiary plants, thus extra care in design must be taken to insure that the disadvantageous features of biological action on carbon can be mitigated.

<table>
<thead>
<tr>
<th>Plant</th>
<th>Effluent Organics mg/l</th>
<th>Carbon lbs. TOC</th>
<th>Capacity lbs. COD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blue Plains (G)</td>
<td>TOC = 6</td>
<td>0.15</td>
<td>0.41</td>
</tr>
<tr>
<td>Lebanon (G)</td>
<td>TOC = 6</td>
<td>0.22</td>
<td>0.50</td>
</tr>
<tr>
<td>Ewing-Lawrence (G)</td>
<td>TOC = 3–5</td>
<td>0.3</td>
<td>—</td>
</tr>
<tr>
<td>New Rochelle (G)</td>
<td>COD = 8</td>
<td>—</td>
<td>0.6</td>
</tr>
<tr>
<td>Owoosso (G)</td>
<td>COD = 25</td>
<td>—</td>
<td>0.65</td>
</tr>
<tr>
<td>Salt Lake City (P)</td>
<td>COD = 22</td>
<td>—</td>
<td>0.36</td>
</tr>
</tbody>
</table>

(Gr) - Granular Carbon  
(P) - Powdered Carbon

Filtration is presented as an optional step in Figure 8–25, but it would be wise to include it in the treatment scheme. Filtration acts as the safety factor in the solids removal step; therefore, its use can bring a high degree of reliability to the treatment scheme. In addition, it can be used to prevent an
overload of solids on the carbon which could foster excessive biological action. The positioning of the filter prior to or after the carbon contact is dictated by the method of carbon contact. Expanded bed and powdered carbon systems usually require filtration after the carbon step. Packed bed systems will benefit from filtration before the carbon.

Tables 8–11, 8–12 and Figure 8–26 provide information on the performance of three physical-chemical treatment pilot plants. These data illustrate removals at various stages of treatment for a variety of pollutants. The Blue Plains data include nitrogen removal by clinoptilolite.

![Graph showing removal efficiencies of physical-chemical treatment](Image)

**FIGURE 8–26** Removal Efficiencies of Physical-Chemical Treatment—Blue Plains Pilot.

**Table 8–11**

<table>
<thead>
<tr>
<th>PCWDERED CARBON PILOT PLANT OPERATING CONDITIONS</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Rate 50 gal/min</td>
<td></td>
</tr>
<tr>
<td>Chemical 425 mg/l Lime to pH 10.8</td>
<td></td>
</tr>
<tr>
<td>Carbon 150 mg/l + 0.4 mg/l polymer</td>
<td></td>
</tr>
</tbody>
</table>

Results

<table>
<thead>
<tr>
<th></th>
<th>COD</th>
<th>BOD</th>
<th>SS</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw Sewage</td>
<td>222</td>
<td>144</td>
<td>200</td>
<td>7.3</td>
</tr>
<tr>
<td>Clarified Effluent</td>
<td>65</td>
<td>47</td>
<td>28</td>
<td>1.4</td>
</tr>
<tr>
<td>Final Effluent</td>
<td>35</td>
<td>13</td>
<td>7</td>
<td>0.4</td>
</tr>
</tbody>
</table>

**Table 8–12**

| ROCKY RIVER WASTE TREATMENT PLANT                  |                  |
| CLARIFICATION-CARBON ADSORPTION                   |                  |

<table>
<thead>
<tr>
<th>Raw</th>
<th>Polymer</th>
<th>Carbon Contact</th>
<th>Time, minutes</th>
<th>% Removed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suspended Solids, mg/l</td>
<td>107</td>
<td>65</td>
<td>14</td>
<td>32.6</td>
</tr>
<tr>
<td>BOD, mg/l</td>
<td>118</td>
<td>57</td>
<td>13</td>
<td>21</td>
</tr>
<tr>
<td>COD, mg/l</td>
<td>235</td>
<td>177</td>
<td>67</td>
<td>50</td>
</tr>
</tbody>
</table>

Table 8–13 lists the features which physical-chemical treatment has with respect to conventional treatment systems. Because of these features and a favorable cost comparison a number of communities are planning physical-chemical treatment systems.

**Table 8–13**

| ADVANTAGES OF PHYSICAL-CHEMICAL TREATMENT          |                  |
| CONVENTIONAL PRIMARY + SECONDARY                  |                  |

1. Less Area Requirement—½ to ¼
2. Lower Sensitivity to Diurnal Variation
3. Not Affected By Toxic Substances
4. Potential For Significant Heavy Metal Removal
5. Superior Removal of P Compounds
6. Greater Flexibility In Design And Operation
7. Superior Organic Removal

**8.2.9.3 Ozone Oxidation.** In addition to carbon adsorption, another physical-chemical method of organics removal, ozonation, has been evaluated and found worthy of development. However, this procedure is several years behind activated carbon in development because so much required information has not yet been obtained.

Ozone is a powerful oxidant which is generated by passing a properly conditioned air stream between electrodes, one of which produces a corona discharge. It must be generated on site and used immediately. Ozone is only poorly soluble in water so that special contacting techniques must be used to insure optimum utilization.

**8.2.9.4 Pure Oxygen Activated Sludge.** The last improvement in procedures for removing organics from wastewater is the application of pure oxygen in the activated sludge process. The use of pure oxygen does not permit activated sludge to go beyond its inherent capability but allows a closer approach to this limit within an acceptable economic framework.
Early work in the biological treatment field pointed out the advantages of pure oxygen over air as the source of oxygen in activated sludge. However, the cost of the pure oxygen was quite high and a feasible system to achieve high degrees of utilization was not available, so the concept lay dormant. Over the last decade, the cost of pure oxygen has decreased and a system was devised which could achieve high utilization efficiency. Figure 8–27 illustrates the UNOX system developed by Union Carbide. The key features of this system are the use of covered reactors to prevent loss of oxygen; a baffled reactor to prevent short-circuiting and recompression and recycle of gas in each chamber. In all other respects, the system is virtually identical to a conventional activated sludge.

**FIGURE 8–27 Schematic Diagram of Multi-Stage Oxygenation System.**

8.2.10 Dissolved Inorganics Removal

In the process of its domestic or industrial use an increment of dissolved mineral matter is added to water. The magnitude varies from 100 milligrams per liter to 500 milligrams per liter per use depending generally on local conditions. Extensive reuse of wastewater even for industrial use will require at least partial demineralization of wastewater. Electrodialysis, ion exchange and reverse osmosis seem promising.

8.2.10.1 Electrodialysis. In sewage application, electrodialysis suffers severe limitations. The anion plates foul quite easily, drastically reducing treatment performance unless the feed has received extensive pretreatment. In addition, as the sewage is demineralized its electrical resistance increases. This limits the degree of economically feasible demineralization which can be obtained. It is difficult to reduce the TDS of water much below 300 to 400 milligrams per liter with electrodialysis. For these reasons electrodialysis is no longer considered a leading candidate process.

8.2.10.2 Ion Exchange. At present, ion exchange shows much promise for removal of dissolved inorganics. Cost of operation is lowest, especially at the salt levels anticipated in wastewater and the technology of ion exchange at full scale is proven.

Ion exchange systems require regeneration with solutions much higher in TDS than the original feed yielding a difficult problem of brine disposal.

8.2.10.3 Reverse Osmosis. At present, reverse osmosis is the most expensive of the three processes discussed for demineralization. However, its potential is considerably superior to that of the other two. This process is indeed in its infancy as the first working membranes were developed only fifteen years ago. In addition, membranes which will desalt water will remove virtually every other pollutant from wastewater. Thus reverse osmosis may become an all purpose pollution control system.

In this process water is forced to flow through a semi-permeable membrane (permeable to water but not to salts) by application of high pressure. Pressures of several hundred pounds per square inch are required to achieve fluxes of the order of ten gallons per square foot per day. The degree of demineralization can approach 99 percent and increases with the pressure applied.

The heart of this process is the membrane. At present, the best membranes are cast from a mix of cellulose acetate, acetone, formamide and magnesium perchlorate. Special techniques form a membrane with a thin (one micron) skin which provides the desalting surface and a thick (100 microns) porous sublayer which supplies a structural backing. This membrane is compressible, hydrolyzes at low and high pH and is easily damaged. Work is being conducted to develop new, tougher membrane materials.

Even if new membrane materials are found, the engineering problems of this process will remain unchanged. These problems are a structural support system to absorb the several hundred pounds per square inch pressure drop across the membrane and control of fouling on membrane surfaces. The support system must not only support the membrane but should provide high membrane area per unit volume.

Three configurations are in use, tubular, spiral-wound and hollow fine fiber. The tubular system has the membrane positioned along the inner wall of a porous one-half inch diameter tube. The spiral-wound module uses a stack of flat membranes separated by spacers and rolled into a jelly-roll form. The hollow fiber uses microscopic fibers of the membrane which are in essence thick-walled microcylinders. Here, pressure is applied outside the fiber and water permeates into the hollow core. As a rough guide, membrane area per unit volume is twenty square feet per cubic foot for tubular systems, 250 square feet per cubic foot for spiral wound systems and 2000 to 5000 square feet per cubic foot for hollow fine fibers. The systems are in reverse order on the basis of fouling resistance. In general, the tubular system is
preferred for dirty water application and the other two for clean water application.

Control of fouling can be achieved by pretreatment, establishment of turbulence at the membrane surface and chemical cleaning techniques. Application of these techniques is easiest in the tubular system. In addition, tubular systems afford the only opportunity for in-situ membrane replacement. At present none of the systems possess a marked advantage over the others.

The economics of wastewater demineralization are quite imprecise as life times of membranes and resins is not known, nor is degree of fouling and degree of demineralization required.

8.2.11 Summary

A review of many of the newest developments in wastewater treatment technology have been presented. In the interest of brevity, some areas such as microorganism removal and ultimate disposal have not been covered, and the coverage of others has been less than complete. It should be clear, however, that the wastewater treatment planner now has a whole range of treatment procedures. It is well within the realm of practicability to conceive of taking any wastewater and purifying it to any degree desired. All that is necessary is to hook together the various unit operations and processes required. A number of combinations which have already found some utility are summarized in the figures and tables below.

8.2.11.1 Biological-Physical Treatment. This system employs a filter to upgrade the performance of conventional treatment. It is illustrated in Figure 8–13 on page 299.

8.2.11.2 Biological-Chemical Treatment. In this system, illustrated in Figure 8–28, chemical coagulants can be added at various points for phosphorus removal and improved solids removal.

8.2.11.3 Physical-Chemical Treatment. In this system, illustrated in Figure 8–29, chemical clarification and activated carbon are combined to achieve wastewater purification.

8.2.11.4 Biological-Physical-Chemical Treatment. This system illustrated in Figure 8–27 combines a large number of processes to achieve a high degree of purification for a large variety of pollutants. Table 8–14 summarizes the performance to be expected from these treatment schemes.

8.2.12 Sprinkler Irrigation

Sprinkler irrigation of domestic, industrial and agricultural liquid wastes is appropriate when the disposal systems are adequately designed and conscientiously operated. The principles involved in adequately handling these liquid wastes can be discussed by use of a renovation-conservation cycle.
Three additional factors concerning the return phase of the cycle are important to effective operation. The liquid wastes must be distributed uniformly over the area. The rate of application must be sufficiently low so that all the water infiltrates through the soil surface. (It must move through the soil so that it can adequately be renovated.) The amount that is actually applied per week or the amount per year must be compatible with the soil type so that renovation can be assured.

8.2.12.3 Renovation. Under renovation, one usually considers mechanical, biological and chemical cleanup. Some industries are interested primarily in mechanical removal of organic material from the liquid wastes, followed by biological degradation. Municipal liquid waste disposal usually involves chemical renovation. That is, nutrients are removed from a solution through chemical reaction or transfer. This process is similar to that involved in agricultural, golf course or lawn fertilization.

In lawn fertilization, the fertilizer is placed on the surface of the soil and then irrigated or leached by rain. The fertilizer is taken into solution in the water. There is essentially no difference between this water and municipal sewage effluent. Each has nutrients in solution and as the water moves through the soil profile, these nutrients are chemically fixed in the soil or attached to the soil chemical complex where they stay until they are needed by the crop. Nutrients, such as nitrates, may be used directly by the crops or by soil microbes.

Farmers have been familiar with and have used this type of nutrient cycle for years. The nutrients are picked up by the crop and removed from the field with the crop. The same procedure is true when applying the various liquid wastes. Here, however, the nutrients are first applied to the soil and then a crop is grown in order to remove these nutrients. Harvesting of a crop makes room for the removal of the nutrients from the next application of wastewater.

8.2.12.4 Recharge. Before water is recharged, it must be adequately treated. Systems must be designed for each specific purpose and use intended for the irrigated liquid wastes.

8.2.12.5 Reuse. Potential reusers include agricultural, industrial and domestic. Agricultural reuse for crops is obvious. Distribution of wastewater over industrial and domestic water supply areas could frequently help alleviate waste disposal, as well as water supply problems.

8.2.13 Waterless Composting Toilet (Clivus Multrum)

Originally developed in Sweden, the composting toilet (or clivus multrum) is very similar in operation to the backyard "privy." Both toilets: and solid, organic kitchen wastes are discharged to and then composted in a specially designed bin in the home. Shower and sink wastewater are disposed of by different mechanisms such as a cesspool or septic tank system.

In the clivus multrum system, the bin is nine feet long, four feet high and five feet wide. The bin holds the organic wastes of a family for several
years. During that time, microbial decomposition takes place to digest the waste and the end product (after two to four years) is a humus that can be used in a garden. Gasses and other volatile material that are produced are vented through a stack. The temperature of the chamber is sufficient to create a positive ventilation pattern with air entering through two tubes and leaving by way of the vent stack. See Figure 8–31.

Since the clivus multrum type of system does not use water, it results in a significant water savings (as much as 40 percent in a typical household). It requires little maintenance. Bacterial populations in the final humus product were shown to be consistent with typical soil bacteria types and levels. In addition,ecal coliform bacteria have not been shown to be present in the mulch.

Manufacturers claim that 90 to 95 percent of the original volume of waste has been converted to waste gas which is vented to the atmosphere. Typical end-product generation rates are three to ten gallons of humus per person per year after the two to four year digestion period.

8.2.14 Individual Home, Aerobic Biological Treatment Units

Traditional on-lot wastewater disposal systems for individual homes include cesspools or septic tank/tile field systems. In each, waste materials are decomposed by bacteria which do not require oxygen for their life systems. Hence, the processes are anaerobic or septic; i.e., devoid of oxygen.

In recent years, equipment has been developed to provide for the aeration of home sewage in individual on-lot systems, thus achieving aerobic decomposition. Waste treatment utilizing aerobic microorganisms is more biologically efficient because the free oxygen dissolved in the wastewater allows the organisms to rapidly feed on and degrade both the suspended and dissolved organic matter. In a few hours, up to 90 percent of the organic matter is destroyed and a similar amount of suspended solids are removed.

A typical household aerobic treatment unit is depicted in Figure 8–32. It consists of two chambers, one for aeration and one for settling. Raw sewage (or septic tank effluent) enters the first chamber and is quickly mixed with the aerobic microorganisms by the air flow from the blower. The mixing brings the microbes into intimate contact with both the dissolved and undissolved waste matter. The nutrient material is rapidly absorbed by the organisms which utilize it for energy and cell growth, thus converting the majority of the organics in the waste to carbon dioxide, water and settleable sludge solids. In the second chamber, the sludge which contains the microorganisms settles out by gravity and is returned to the aeration chamber to continue the treatment process. The clarified effluent leaves the unit low in organic matter and suspended solids with partially reduced levels of fecal coliform bacteria. The aerobic home treatment units, however, do not remove nitrates from the wastewater.

While these units can produce a higher quality of effluent without odors, they require electrical energy, regular maintenance and servicing.

FIGURE 8–31 Clivus Multrum Toilet.
Because they may reduce the amount of solids carried over to the tile field, manufacturers claim a longer life of the tile field in tight soils. However, the treatment process is easily upset thus causing high discharges of solids to the disposal field.

Household aerobic treatment units are considerably more expensive to install and operate than a septic tank/tile field system. They require constant maintenance whereas a septic tank system requires essentially no maintenance. The aerobic system appears to offer advantages in areas of tight, impermeable soils or where other leaching problems may occur.

8.2.15 Meadow/ Marsh/Pond (M/M/P) System or Marsh/Pond (M/P) System

A meadow/marsh/pond system is an attempt to utilize aesthetically pleasing, natural systems for wastewater treatment. The approach depends upon long detention times for the slow, but effective, removal of suspended and dissovled pollutants. It utilizes physical removal of pollutants as well as biological reduction.

The process usually begins by screening the sewage to remove bulky objects. It may then pass through a degritter to remove sand and other non-degradable, coarse materials. The solid materials are then comminuted (shredded and chopped) to reduce degradables to fine, small particles. The waste stream then flows to basins where it is aerated and wherein biological decomposition begins.

Following aeration, the blend is normally pumped to a distribution box feeding two lined, sloped meadows. Only one meadow need be in operation at a given time while the other meadow is being dried out for harvesting of the crop. Meadows are alternated – the time intervals being determined by the sewage strength, the application rate, the condition of the crop and the season of the year. Meadowgrasses and crops are planted which assist in the removal of dissolved pollutants materials.

Gravity flow through the sloped meadow terminates in a lined marsh which merges with the meadow. The choice of plantings in the marsh depends on the geographic area and nature of the waste and generally will include harvestable crops which will flourish in the local climate.

In turn, the marsh terminates in a lined stabilization pond stocked with harvestable aquatic species which are indigenous to the area. A constant level is maintained in the marsh and pond by a fixed-invert overflow structure through which the treated waste can be discharged or disposed of in a different manner. Again, the biota of the pond are selected based upon the wastewater to be treated and the geographic area of the County.

Effluent levels of suspended solids, BOD and nutrients have been shown to be within regulatory agency standards. With high summer temperatures, bacteria populations may be higher than acceptable and a disinfection step would be necessary.

Since the entire M/M/P system has an underlying, impervious membrane liner there is no loss of contaminants to the groundwater environment. The entire process is thus controlled prior to discharge.

The marsh/pond design is the same as the meadow/marsh/pond—the marsh/pond system. The system requires less land and produces about the same quality effluent. It handles shock loadings fairly well but, like the full M/M/P system, has not been tested extensively in non-controlled conditions.

The M/M/P or M/P systems significantly reduce pollutants, including nitrites, if a system of cropping is instituted. Nitrogen in the wastewater is uptaken by the plants and becomes part of the biostroette. Unless that structure is removed, it will eventually die, decompose and release the nutrients back to the aquatic environment. See Figure 8–33.
8.2.16 Subsurface Denitrification of Septic Tank Effluents

In a granular type of soil, cesspool and septic tank systems, working in conjunction with the soil, are ideal for removal of organics and suspended solids. However, nitrates cannot be effectively removed without substantial modification.

The Suffolk County Department of Health Services is conducting a research project to demonstrate the feasibility of removing nitrogen in modified residential subsurface sewage disposal systems. An experimental disposal system has been constructed, consisting of a septic tank, leaching tile field system and denitrification system (see Figure 8–34). The disposal system is being fed with controlled quantities of sewage from an apartment complex. The wastewater first enters a conventional septic tank and tile field. An impermeable membrane deflector is located six feet below the bottom of the tile field and is sloped as to intercept and deflect the wastewater to a large impermeable pan.

Nitrogen compounds in the wastewater are first nitrified (converted from ammonia to nitrate) in the aerobic layer of native soil immediately below the tile field and above the membrane deflector. Once in the nitrate state, denitrification by certain types of bacteria (denitrifiers) will occur when anaerobic (devoid of oxygen) conditions are achieved in the pan. Methanol must be fed as a food source for the bacteria. This method of denitrification is commonly used in conventional waste treatment plants.

The denitrification process converts the nitrogen which is in the form of nitrate (NO₃⁻) to a gaseous state (N₂). A venting mechanism for releasing the nitrogen gas to the atmosphere is provided.

Investigation is currently underway to eliminate the necessity of feeding methanol, which has drawbacks when applied to individual sanitary disposal systems. The most promising alternate method would be the diversion of part of the septic tank effluent to the anaerobic zone.

![FIGURE 8–34 Denitrification of Septic Tank Effluents.](image-url)
9.0 INTRODUCTION

Population Estimates and Projections, 1975–1995 contains estimates of the present (1975) population and of the population at zoned capacity for each of the 108 municipalities, the 126 school districts and the 30 drainage basins in Nassau and Suffolk counties. It also contains the Nassau-Suffolk Regional Planning Board’s projection of the population of those same geographic areas for the years 1980, 1985, 1990 and 1995, together with an explanation of the methodology used to develop the projections. There is a brief discussion of the major changes in town and school district population indicated by the projections, and of the relation of the projected totals to the “saturation population”; that is, the population of the projection areas at zoned capacity. This is followed by a consideration of gross residential densities. A brief discussion of the changes in zoned capacity and of the differences between the Nassau-Suffolk 1966 and 1975 projections concludes the body of the text. A comparison of the Nassau-Suffolk projections with those of other agencies is included in the Appendix.

The intended use of the projection influenced the selection of the geographic areas for which projections were developed. The 208 Study requires the production of “baseline” or trend projections in order to determine the magnitude of future needs and to evaluate the effects of various wastewater management alternatives on future population. Since large-scale wastewater management planning generally deals with drainage basins or sizable sub-basins it was necessary to provide the engineers, hydrologists, soil scientists and other technical personnel with population projections for drainage basins. However, a number of considerations—among them the Board’s dependence upon readily available data, the potentially limited usefulness of projections for what were often extremely large, irregularly shaped drainage areas and the probable need for a subsequent breakdown of basin totals for sub-basins or other smaller areas for “208” or “201” (facilities planning) purposes, the desire for flexibility without loss of consistency, and the great number of requests for municipal and school district projections for general planning purposes—led the Planning Board to undertake the development of projections for the thirteen towns, the school districts or the portions thereof contained in each town, and the cities and villages or portions thereof in each town and school district. The availability of demographic and housing data and of zoning information for municipalities and school districts proved a most persuasive argument for the selection of these areas. The relative ease with which school districts or portions of school districts could be aggregated to drainage basins was an important consideration. The enhanced usefulness for general planning purposes and the simplicity of periodic revision, as necessary to reflect significant changes, were further reasons for the tabulation of data and the projection of data and the projection of population by municipality and school district.

9.1 CURRENT POPULATION ESTIMATES

The Nassau-Suffolk Regional Planning Board considers the Long Island Lighting Company’s population estimates for January 1, 1975 as the most accurate available for the Bi-County area and has adopted them as the base year numbers for its population projections.

The LILCO figures are part of a continuing series of estimates, produced on an annual basis. They are essentially an up-date of the latest decennial census and of special censuses, where available, adjusted to reflect changes in the population as indicated by increases or decreases in the
number of occupied housing units. The changes in the number of housing units are obtained from company records of active meters, new connections and discontinuances of service. The meter count for each area is multiplied by a factor for household size to produce an estimate of the area’s population. The household size multipliers are generally derived from the decennial census data and may be somewhat outdated and inaccurate by the latter half of the intercensal period. Nassau-Suffolk suspects that there may be areas in Nassau and in western Suffolk where the LILCO household size factors fail to reflect the full magnitude of recent changes in family size; however, such inaccuracies, if they do exist, are of little significance. The LILCO estimates for January 1, 1975 have been used as the base year figures for projection purposes. The relatively small increase in population that has been projected for the period 1975–1980 is expected to compensate for any possible overstatement of the 1975 population.

Table 9–1 indicates the estimated population of the Nassau-Suffolk 208 Planning Region, the two counties, and the fifteen major municipalities as of 1975. Comparable estimates for school districts, villages and drainage basins are shown in Appendix Table A-1.

Table 9–1

<table>
<thead>
<tr>
<th>County and Region</th>
<th>Municipality</th>
<th>Population 1/1/75</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Glen Cove City</td>
<td>26,880</td>
</tr>
<tr>
<td></td>
<td>Hempstead</td>
<td>814,050</td>
</tr>
<tr>
<td></td>
<td>Long Beach City</td>
<td>34,766</td>
</tr>
<tr>
<td></td>
<td>North Hempstead</td>
<td>238,559</td>
</tr>
<tr>
<td></td>
<td>Oyster Bay</td>
<td>341,692</td>
</tr>
<tr>
<td>Nassau County</td>
<td></td>
<td>1,455,947</td>
</tr>
<tr>
<td></td>
<td>Babylon</td>
<td>217,923</td>
</tr>
<tr>
<td></td>
<td>Brookhaven</td>
<td>317,489</td>
</tr>
<tr>
<td></td>
<td>East Hampton</td>
<td>13,063</td>
</tr>
<tr>
<td></td>
<td>Huntington</td>
<td>213,643</td>
</tr>
<tr>
<td></td>
<td>Islip</td>
<td>312,010</td>
</tr>
<tr>
<td></td>
<td>Riverhead</td>
<td>21,184</td>
</tr>
<tr>
<td></td>
<td>Shelter Island</td>
<td>1,918</td>
</tr>
<tr>
<td></td>
<td>Smithtown</td>
<td>122,498</td>
</tr>
<tr>
<td></td>
<td>Southampton</td>
<td>41,239</td>
</tr>
<tr>
<td></td>
<td>Southold</td>
<td>18,733</td>
</tr>
<tr>
<td>Suffolk County</td>
<td></td>
<td>1,279,690</td>
</tr>
<tr>
<td>Nassau-Suffolk S.M.S.A.</td>
<td></td>
<td>2,735,637</td>
</tr>
</tbody>
</table>

9.2 SATURATION POPULATION

The methodology used by Nassau-Suffolk to develop the area-wide and sub-area projections combines the concept of land capacity; that is, holding capacity under existing zoning, with trend analysis on a small area basis. The following paragraphs describe the step by step procedure that was followed.

Step 1. Identification and tabulation of all vacant land zoned for residential uses, including land in large estates. Aerial photographs, tax maps and assessment records were used to identify vacant acreage, zoning exempt lots (filed subdivisions) and scattered parcels in built-up areas. Zoning maps were consulted to determine the zoning category of vacant acreage and scattered lots in developed areas. Field reconnaissance was employed when necessary to supplement aerals and office records. The number of acres of vacant land in each zoning category, in each municipality or school district was recorded for use in calculating the total number of potential building sites. The number of zoning exempt lots, if any, was also noted.

Step 2. Calculation of existing vacant and potential building sites. The number of already subdivided vacant parcels, including zoning exempt lots was tabulated. (The category “Zoning Exempt Lots,” which is found most frequently in the eastern portion of Suffolk County, refers to lots on old filed maps that pre-date most of the local zoning ordinances. In areas characterized by considerable numbers of zoning exempt lots, the present zoning ordinance offers few clues as to the course of future development. For example, in some parts of the East End, the current zoning ordinance requires a minimum plot size of one acre or more; however, many parcels of 1/4 acre or less may constitute legal building sites because they were legal lots when the land was originally subdivided and have remained in single and separate ownership from that date to the present time. Wherever necessary an analysis of the tax map was undertaken in order to determine the magnitude of single and separate ownership and to permit evaluation of its impact on the ultimate utilization of the land. In built-up areas, any vacant parcels that appeared to be larger than necessary to conform to existing zoning were reviewed to determine whether they might yield more than a single building site. The computation of the potential yield of as yet unplatted acreage was accomplished through the application of a yield per acre factor for each zoning category to the total number of acres in that category. The yield per acre factors used are listed in Table 9–2. They represent average values based upon recent Nassau-Suffolk experience with conventionally designed subdivision plats.

The saturation estimate is based on existing zoning, so that if land were to be zoned for a higher or lower density, from or to non-residential use,
<table>
<thead>
<tr>
<th>Zoning Lot Size</th>
<th>Lots Per Acre</th>
<th>Zoning Lot Size</th>
<th>Lots Per Acre</th>
</tr>
</thead>
<tbody>
<tr>
<td>4,000</td>
<td>6.8</td>
<td>16,000</td>
<td>1.9</td>
</tr>
<tr>
<td>5,000</td>
<td>5.4</td>
<td>16,500</td>
<td>1.8</td>
</tr>
<tr>
<td>6,000</td>
<td>4.5</td>
<td>18,000</td>
<td>1.7</td>
</tr>
<tr>
<td>6,500</td>
<td>4.1</td>
<td>18,500</td>
<td>1.6</td>
</tr>
<tr>
<td>7,000</td>
<td>3.8</td>
<td>20,000</td>
<td>1.5</td>
</tr>
<tr>
<td>7,500</td>
<td>3.6</td>
<td>20,500</td>
<td>1.5</td>
</tr>
<tr>
<td>8,000</td>
<td>3.4</td>
<td>21,750</td>
<td>1.4</td>
</tr>
<tr>
<td>8,500</td>
<td>3.3</td>
<td>22,000</td>
<td>1.3</td>
</tr>
<tr>
<td>9,000</td>
<td>3.1</td>
<td>25,000</td>
<td>1.1</td>
</tr>
<tr>
<td>10,000</td>
<td>2.7</td>
<td>30,000</td>
<td>1.0</td>
</tr>
<tr>
<td>11,250</td>
<td>2.6</td>
<td>40,000</td>
<td>0.8</td>
</tr>
<tr>
<td>11,500</td>
<td>2.5</td>
<td>43,500</td>
<td>0.7</td>
</tr>
<tr>
<td>12,250</td>
<td>2.4</td>
<td>45,000</td>
<td>0.7</td>
</tr>
<tr>
<td>13,000</td>
<td>2.3</td>
<td>2 Acres</td>
<td>0.4</td>
</tr>
<tr>
<td>13,500</td>
<td>2.1</td>
<td>3 Acres</td>
<td>0.27</td>
</tr>
<tr>
<td>14,000</td>
<td>2.0</td>
<td>4 Acres</td>
<td>0.20</td>
</tr>
<tr>
<td>15,000</td>
<td>2.0</td>
<td>5 Acres</td>
<td>0.16</td>
</tr>
</tbody>
</table>

or acquired for public purposes such as recreation or farm preservation, the saturation figure would have to be adjusted. Recent observations suggest that the pattern of zoning on Long Island has become relatively stable. Changes to higher density are often offset by other changes to a lower density or by a land acquisition, thus minimizing the impact of any change in the ultimate saturation calculation.

**Step 3. Identification and tabulation of housing units by occupancy status.** The 1970 Census housing counts for all units and for vacant, seasonal, and occupied units were listed for each municipality and school district. Building permit data for the years 1970 through 1974 were employed to estimate the increase in the housing stock between 1970 and 1975. Where multi-family units constituted a sizeable percentage of the new housing, the number of single-family and multi-family units were noted separately. Long Island Lighting Company meter records were used in combination with 1970 census and 1970–74 building permit data to estimate the total number of dwellings and the number of occupied units in 1975.

Inasmuch as housing data in general and building permit data in particular are rarely tabulated for areas whose boundaries coincide with or even approximate those of school districts, it was necessary to employ allocation procedures utilizing aerial photographs, office records, and field verification in order to assign both pre-1970 and post-1970 housing to the appropriate school districts.

**Step 4. Calculation of the total number of housing units permitted under existing zoning.** The total number of housing units now in place and the total number of existing and potential building sites, each of which could be used to accommodate an additional unit, were summed to obtain an estimate of the number of potential housing units and households.

The saturation figure reflects the assumption that all existing housing units that are currently not occupied will be utilized and that seasonal and vacant year-round units will continue to exist in the five eastern towns of Suffolk County until after year 2000. Few seasonal and second homes are anticipated elsewhere on the island.

**Step 5. Estimation of household size at saturation.** Census counts of total population in households and occupied housing units were used to calculate the 1970 household size for each municipality and school district. Population and occupied housing unit estimates for 1975 were employed to calculate the significant changes, if any, that had occurred between the two dates. Following a review of the 1970 and 1975 figures and consideration of such relevant factors as the sharp decline in births, the age composition of the community, income, ethnicity, and of the increasing proportions of multifamily dwellings in a few localities, a probable average household size at saturation was selected for each of the projection areas. Appendix Tables A-2 through A-14 indicate the type of background information available for each of the towns and the format used for tabulations.

Increases have been projected for some communities presently characterized by a relatively small household size but having considerable potential for growth. Decreases have been projected for others now characterized by large households, little remaining land and declining school enrollments. Where the potential for apartment or condominium construction is greater than that for single family home construction, family size is expected to decrease. It is also expected to decrease in some Eastern Suffolk areas since these areas tend to attract small, elderly households despite the overwhelming predominance of single family homes.

The Nassau-Suffolk staff has made several assumptions that directly affect the selection of the household size factor for school districts and municipalities. It has been assumed that the overall trend towards smaller households that has been observed for more than a decade in much of Nassau and Suffolk will continue as both the national and local birth rate decline and as more and more young adults and the elderly choose to maintain their own households.

It is expected that an aging population will further reduce the number of births in many communities in Nassau and western Suffolk. As older households leave the area, the high cost of their former dwellings generally
precludes purchase by young families in the primary child-bearing age groups. The head of the household of the new family is likely to be between 35 and 49 years of age with a spouse of similar age and children in the 10 to 19 year age group. Some growth is expected to occur even in the absence of net natural increase. Although the new or in-migrant family may be somewhat smaller than that of the original occupants of the dwelling, it may be larger than the original family minus the grown children that have left to establish their own households. In communities where this type of change is occurring, a period of gradually decreasing household size is followed by a period of increasing household size that results in population gains.

**Step 6. Estimation of population at zoned capacity.** The number of existing and potential housing units, calculated in Step 2 was multiplied by the appropriate household size, selected in Step 5, to obtain the saturation population for each school district and municipality. The final line in Appendix Tables A-2 through A-14 indicates the estimated population at zoned capacity for each of the projection areas.

### 9.3 POPULATION PROJECTIONS

The twenty year projections are, in the first instance, town-wide projections. Figures for the school districts, cities, and villages were derived from the town projections by means of a step-down apportionment technique that permitted the allocation of anticipated growth to sub-areas. The following paragraphs summarize the procedures followed and describe some of the considerations that influenced the selection of growth rates and the assignment of shares.

As the initial step in the projection process, the growth patterns of the towns and sub-areas for the years 1970–1975 were subjected to detailed scrutiny and analysis. After in-depth study of the annual changes in population as estimated by Long Island Lighting Company, of building permit data, and of school enrollment figures, average annual growth rates were selected for each of the thirteen towns for the 1975–1980, 1980–1985, 1985–1990, and 1990–1995 projection periods.

The 1975–1980 rates were applied to the base year estimates to obtain figures for projected growth for the first five year period. Projected population increments and base year estimates for each town were summed to produce the 1980 projections. The rates for each of the three subsequent five year periods were applied to the projected population at the beginning of each period to obtain figures for anticipated growth and to permit calculation of the total population at the end of the respective five year projection period. Table 9–3 shows the base year estimates and the projected population for the thirteen towns, two cities, the counties and the Nassau-Suffolk Region.

<table>
<thead>
<tr>
<th>Nassau County</th>
<th>1,456,947</th>
<th>1,476,789</th>
<th>1,475,684</th>
<th>1,461,821</th>
<th>1,468,566</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glen Cove</td>
<td>26,880</td>
<td>27,878</td>
<td>28,339</td>
<td>28,699</td>
<td>28,981</td>
</tr>
<tr>
<td>Hempstead T.</td>
<td>814,050</td>
<td>817,353</td>
<td>819,568</td>
<td>821,390</td>
<td>823,268</td>
</tr>
<tr>
<td>Long Beach</td>
<td>34,768</td>
<td>35,716</td>
<td>36,491</td>
<td>37,241</td>
<td>37,941</td>
</tr>
<tr>
<td>North Hempstead T.</td>
<td>238,559</td>
<td>240,706</td>
<td>242,994</td>
<td>245,424</td>
<td>246,650</td>
</tr>
<tr>
<td>Oyster Bay T.</td>
<td>341,692</td>
<td>346,136</td>
<td>346,302</td>
<td>349,067</td>
<td>349,725</td>
</tr>
<tr>
<td>Babylon T.</td>
<td>217,923</td>
<td>220,175</td>
<td>229,530</td>
<td>243,144</td>
<td>244,362</td>
</tr>
<tr>
<td>Brookhaven T.</td>
<td>317,489</td>
<td>378,564</td>
<td>430,904</td>
<td>494,346</td>
<td>546,198</td>
</tr>
<tr>
<td>East Hampton T.</td>
<td>13,053</td>
<td>15,549</td>
<td>18,992</td>
<td>22,066</td>
<td>25,637</td>
</tr>
<tr>
<td>Huntington T.</td>
<td>213,643</td>
<td>226,546</td>
<td>234,761</td>
<td>241,089</td>
<td>244,759</td>
</tr>
<tr>
<td>Islip T.</td>
<td>312,010</td>
<td>339,044</td>
<td>367,156</td>
<td>388,290</td>
<td>399,895</td>
</tr>
<tr>
<td>Riverhead T.</td>
<td>21,184</td>
<td>23,412</td>
<td>26,529</td>
<td>30,363</td>
<td>34,752</td>
</tr>
<tr>
<td>Shelter Island T.</td>
<td>1,918</td>
<td>2,228</td>
<td>2,790</td>
<td>3,324</td>
<td>3,960</td>
</tr>
<tr>
<td>Smithtown T.</td>
<td>122,498</td>
<td>128,913</td>
<td>134,810</td>
<td>141,534</td>
<td>147,381</td>
</tr>
<tr>
<td>Southampton T.</td>
<td>41,239</td>
<td>47,200</td>
<td>54,837</td>
<td>64,352</td>
<td>75,518</td>
</tr>
<tr>
<td>Southold T.</td>
<td>18,733</td>
<td>21,016</td>
<td>23,814</td>
<td>26,716</td>
<td>29,972</td>
</tr>
<tr>
<td>Suffolk County</td>
<td>1,279,690</td>
<td>1,411,737</td>
<td>1,562,123</td>
<td>1,655,224</td>
<td>1,752,434</td>
</tr>
</tbody>
</table>

The apportionment of town growth to sub-areas was accomplished by the assignment of a share or percentage of the total growth to each school district or municipality or portion thereof located within the town. The prediction of long term change in small areas is difficult at best, and it was frequently necessary to develop not only a preliminary set but one or more revised sets of percentages before achieving an acceptable formula for the distribution of town growth. The shares, which varied considerably by area and projection period, were determined on the basis of staff assessment of relevant factors. The factors considered included, among others, the general development trends; the growth of the sub-area as compared with other sub-areas and the town as a whole; the availability of land; the desirability of sub-area, including accessibility; the type and cost of housing; and the presence or absence of impediments to development.

In some areas that are presently experiencing or are expected to experience population losses, the shares are characterized by negative values. The school districts or municipalities with negative shares were generally those in the older developed portions of the Region where the present population is close to or even exceeds the projected saturation population or where the losses resulting from a sharp decrease in household size are expected to more than offset the gains resulting from new development.

A comparison of the population estimates for 1975 and the figures for
the saturation population at zoned capacity indicates that many school districts and municipalities in Nassau County have already reached or exceeded the saturation figures. Several facts can be cited to explain this apparent contradiction. In most sub-areas the projected household size is smaller than the present household size and therefore the same, or possibly, even a slightly greater number of households can be expected to produce a smaller population at saturation than in 1975. In a few areas there are significant numbers of existing and potential illegal two or more family dwellings in single-family zones; however, in the absence of precise information as to their present number and as to local enforcement intentions, these had to be excluded from the saturation calculations. In every area having a sizeable institutional population, that population, or the population that could be accommodated at the one or more institutional facilities located in the school district or municipality was also excluded from the saturation calculation. The institutional population residing in such places as the state psychiatric centers, the Northport Veterans Hospital, and several universities was projected separately and then added to the household population to produce a combined total for both household and institutional population.

Appendix Tables A-15 through A-26 present the base year population, the five year change in the number of persons, and the total projected population for school districts and municipalities for the years 1980, 1985, 1990 and 1995.

Appendix Table A-27 summarizes the same data for the 21 school districts and 7 villages that include land in more than one town. Projections for drainage basins may be found in Appendix Table A-28. Appendix Tables A-30 through A-42 list the saturation population at zoned capacity and indicate the percent of saturation represented by the projected population for the school districts, the municipalities, and the districts and municipalities with land in more than one town.

A review of the school district and municipality projections reveals that in many of the areas already at or above saturation the population is expected to decrease somewhat but to remain above or near saturation throughout the twenty year period. It is anticipated that by 1995 the population of many other areas, particularly those in eastern Nassau and western Suffolk, will be at or near saturation. It appears unlikely that saturation at zoned capacity will be reached in most parts of central and eastern Suffolk until some time after the year 2000.

The reader should be aware that it is often even more difficult to project the precise timing of growth than it is to project its magnitude; and that consequently, the projection dates should be regarded as nothing more than the best possible approximations, given present uncertainties. For example, areas where lots in existing built-up neighborhoods or on mapped rapid change as large scale, well-financed developers proceed to convert estates, nurseries, clubs and preserves, and farmland to new communities.

Recent efforts to preserve farmland, shorefronts, wetlands and other areas of great environmental or recreational value have been taken into account in the projections and it has been assumed that should these efforts succeed, the growth rate in the affected areas will be slower than might otherwise be anticipated. It has also been assumed that as the population of a school district or municipality approaches saturation, the rate of growth will decline since the most desirable, moderately priced, easy to develop land will already have been utilized. The land that remains is frequently characterized by physical or legal impediments that deter its use for housing or, in other instances, is the subject of belated community efforts to preserve the last remaining local open space.

9.3.1 Projected Twenty Year Changes in Town and School District Populations.

According to the current series of projections, 93.1% of the Region's twenty year growth of approximately one half million persons will occur in streets constitute a large proportion of the vacant land can be expected to experience relatively slow, steady growth through most or all of the projection period; however, areas where large tracts of presently unsubdivided land constitute most of the vacant acreage can be expected to experience sudden, Suffolk County. Two towns, Brookhaven and Islip, each with a present population in excess of 300,000 persons, are expected to accommodate more than five-eighths of the additional residents. Table 9-4 indicates the number of persons and the share of the total Bi-County growth projected for the two counties and each of the fifteen major municipalities.

As might be expected, the population projections generally indicate little change in the number of residents in the Nassau County school districts; moderate change in the western Suffolk school districts; great change in the central Suffolk school districts; and little to moderate change, in the eastern Suffolk school districts.

Only four districts totally within Nassau County—Long Beach, Hempstead, Glen Cove, and Oyster Bay-East Norwich—and two Suffolk districts that include portions of Nassau-Cold Spring Harbor and Amityville—are projected to gain more than 2000 persons during the twenty year period. Another seven are projected to gain from 1000 to 2000 persons, while the remainder are projected to add fewer than 1000 persons or to lose population.

In contrast, four districts in Suffolk County—Connetquot in the Town of Islip, Sachem in the towns of Islip, Brookhaven and Smithtown, and Middle Island and William Floyd in the Town of Brookhaven—are expected to
add more than 20,000 new residents. Seven other districts—Smithtown in western Suffolk; Middle Country, Patchogue, Miller Place, Rocky Point and Shoreham-Wading River in central Suffolk; and Riverhead in eastern Suffolk—are expected to add between 10,000 and 20,000 new residents. Twenty districts are expected to add from 5000 to 10,000 persons. Three of these districts are in the Town of Huntington; two, in the Town of Babylon. One is in the Town of Smithtown; eight are in the Town of Islip; four, in the Town of Brookhaven; and two, in the Town of Southampton. Twenty-four districts, nine of them in the towns of Huntington and Babylon, are expected to add between 2000 and 5000 persons while the remaining sixteen districts—all but one of them located in eastern Suffolk—are expected to add fewer than 2000 persons.

Table 9-4

THE COUNTIES AND MAJOR MUNICIPALITIES:
AMOUNT AND DISTRIBUTION OF PROJECTED TWENTY YEAR GROWTH

<table>
<thead>
<tr>
<th>Projected Growth</th>
<th>Share of Regional Growth (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Town (No. of Persons)</td>
<td>1975–1995</td>
</tr>
<tr>
<td>Glen Cove</td>
<td>2,101</td>
</tr>
<tr>
<td>Hempstead T.</td>
<td>9,218</td>
</tr>
<tr>
<td>Long Beach</td>
<td>3,175</td>
</tr>
<tr>
<td>North Hempstead T.</td>
<td>8,091</td>
</tr>
<tr>
<td>Oyster Bay T.</td>
<td>8,033</td>
</tr>
<tr>
<td>Nassau County</td>
<td>30,618</td>
</tr>
<tr>
<td>Babylon T.</td>
<td>26,439</td>
</tr>
<tr>
<td>Brookhaven T.</td>
<td>228,709</td>
</tr>
<tr>
<td>East Hampton T.</td>
<td>12,584</td>
</tr>
<tr>
<td>Huntington T.</td>
<td>31,116</td>
</tr>
<tr>
<td>Islip T.</td>
<td>87,885</td>
</tr>
<tr>
<td>Riverhead T.</td>
<td>13,568</td>
</tr>
<tr>
<td>Shelter Island T.</td>
<td>2,042</td>
</tr>
<tr>
<td>Smithtown T.</td>
<td>24,883</td>
</tr>
<tr>
<td>Southampton T.</td>
<td>34,279</td>
</tr>
<tr>
<td>Southold T.</td>
<td>11,239</td>
</tr>
<tr>
<td>Suffolk County</td>
<td>472,744</td>
</tr>
<tr>
<td>Nassau-Suffolk S.M.S.A.</td>
<td>503,362</td>
</tr>
</tbody>
</table>

9.3.2 Gross Residential Density

Gross residential density, that is, the average number of persons per acre provides a crude indicator of probable need for the design and construction of local or area-wide wastewater collection and treatment facilities. Appendix Table A.30, which was originally prepared at the request of the 208 Study's consulting engineers, indicates gross densities for each of the thirteen towns and 126 school districts. Portions of school districts comprising land in more than one town are listed under their respective towns. Gross residential densities for the total area of the split districts can be obtained by dividing the sum of the population of the component sections by the sum of the acreage of those sections.

It is important to note the potentially critical limitations of a measure such as gross residential density. Its usefulness varies directly with the homogeneity of development patterns and inversely with the proportion of the total acreage in non-residential uses in any particular area. As an average, i; tends to mask extreme values and therefore fails to signal the presence of small sectors where existing or anticipated densities may be sufficient to warrant the provision of services not justified for the area as a whole. The presence of relatively large parcels devoted to industry, parks, airports or other public facilities often distorts the gross residential density calculation, making the area as a whole appear far less intensively utilized than is actually the case. In addition, for areas subject to seasonal variations in population, the figure for gross density, which is based on the number of year-round residents, will be unduly low; for areas characterized by large numbers of persons in hospitals or other group quarters, it will probably be somewhat high.

9.3.3 Changes in Projected Population at Zoned Capacity

Table 9-6 facilitates comparison of the most recent figures for population at zoned capacity, or saturation population, and those figures that were
developed a decade ago as part of the resource information for the Nassau Sufllolk Comprehensive Plan.

The new saturation population statistics for Nassau County show a potential increase of just under 15,000 persons, while those for Suffolk County show a decrease of over one half million. The small increase in Nassau is a result of a more accurate determination of available lots, rezonings to increase density, additional conversions to two family use and redevelopment of underutilized land. The reduction in the City of Long Beach is attributable to a density limit on major apartment buildings.

The significant decrease in Suffolk County has occurred largely as a result of municipal action by the eastern towns to implement plan recommendations calling for reductions in overall density in order to limit the demand for costly services, while maintaining the basically recreational-agricultural environment. The Town of Southold figures are a good example and show the result of a rezoning from lot sizes generally of 1/4 acre to lot sizes of one acre or more in most of the undeveloped parts of the Town. In contrast, the statistics for the Town of Islip reflect the acceptance and construction of higher density housing throughout the Town since the date of the earlier survey.

9.3.4 Comparison of Present and Earlier Series of Small Area Projections

The latest population projections reflect a slower growth rate than that anticipated in the projections developed in the previous decade. Table 9–7 lists the 1985 projections of both series and indicates the magnitude of the difference between them. It should be noted that 1985 represents the midpoint of the current projection period, only ten years removed from the present; whereas it represented the final date, some twenty years from the present when the earlier series was developed a decade ago.

The Bi-County area is expected to have just over three million people in 1985 or 300,000 fewer people than were projected earlier. The new projections are approximately 200,000 lower in Suffolk and 100,000 lower in Nassau County. The 1985 Suffolk County projected totals, contained in the earlier report, may very well be reached in the period between 1995 and 2000, while the earlier 1985 Nassau projected totals should not be reached in the foreseeable future. The high cost of new housing, which prices younger or poorer families out of the housing market, and an overall slowdown in the economy appear to be the major reasons for the slowed growth in Suffolk, where land is still available for extensive housing developments. The Nassau decrease is related to the same factors; however, less land is available and new housing on this land is very costly. The reduction in projected family size and revised plans drastically limiting housing at Mitchel Field also help to explain the markedly slower growth now projected for Nassau County.